Delaware Department of Transportation

Road Design Manual

(Archived)

2004 Edition

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
800 BAY ROAD
P.O. BOX 778
DOVER, DELAWARE 19903

NATHAN HAYWARD III
SECRETARY

MEMORANDUM

To: All Recipients of the January 2004 Road Design Manual

From: Kevin T. Canning
Quality Engineer KC

Date: December 13, 2004

Subject: **2004 Road Design Manual - Revision**

Attached you will find a revision to the Delaware Department of Transportation's 2004 Road Design Manual, this revision will be effective December 13, 2004. The changes are as follows:

- Sheet 3-5 – A line has been added for the signature of DelDOT's Assistant Director – Design.
- Sheet 3-7 – A misspelled word has been corrected.
- Sheet 10-24 – References to utility easements have been deleted.

Please replace the pages in your manual with the attached sheets.

Any comments or questions regarding the 2004 Road Design Manual should be addressed to Linda M. Osiecki.

KTC:los
Enclosure
cc: Carolann Wicks, Chief Engineer
Dennis M. O'Shea, Assistant Director – Design
Michael Simmons, Assistant Director, Project Development, South Region
Michael Angelo, Assistant Director, Project Development, North Region
Linda M. Osiecki, Consistency Engineer, Quality

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Figure 3-2
Design Exception Request

State Project No._____Federal-Aid Project No._____

Date: _____**Oversight Project: Yes _____No____**

Design Exception Abstract: (Provide a short summary detailing the nature of the exception, reasons for the request, etc.)

Note:
For all NHS projects, the thirteen controlling criteria to be met are design speed; through lane and auxiliary lane width; shoulder width; bridge width, structural capacity, horizontal alignment; vertical alignment; grades; stopping sight distance, cross-slope; superelevation; horizontal clearance; and vertical clearance.

RECOMMENDATION:

The purpose of this project is to----------.

The most effective method of addressing this is----------.

Based upon the conditions presented, it is recommended that a design exception be approved for the controlling substandard design element as justified.

Recommended By: _____
Squad Manager

Recommended By: _____
Group Engineer

Recommended By: _____
Assistant Director-Transportation Solutions

Recommended By: _____
Assistant Director-Design

Approved By: _____ **Date**: _____
Chief Engineer

Approved By: _____ **Date**: _____
Federal Highway Administration (NHS oversight projects only)

Enclosures: (Include design criteria, figures, calculations, etc. to document request.)

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-5, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smoother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages 156 to 161, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

10.5.2 EASEMENTS

Under certain conditions it is preferable to obtain an easement rather than to purchase right-of-way. There are two types of easements: (1) temporary and (2) permanent easements. The type of easement should be identified on the plans.

A temporary easement should be obtained where it is not necessary to obtain permanent possession of the land or permanent right of access to the land. Temporary easements are appropriate:

- For any areas where the Department will have no maintenance responsibility after the completion of the proposed project construction;
- Where relatively flat cut or fill slopes extend beyond the right-of-way line and the lateral clear zone or for grading purposes that may benefit the property;
- To obtain proper grade on private driveways and approaches;
- For channel changes and inlet and outlet ditches at drainage structures where future maintenance is not anticipated; and
- For construction working areas.

A permanent easement should be obtained where it is not necessary that the State own the land, but where perpetual interest is necessary. Examples are where the Department needs to access the property for future maintenance, repair, or replacement of the highway facility, its drainage systems or appurtenances and as provided for in a project's right of way or railroad agreement.

10.5.3 RIGHT-OF-WAY MONUMENTS

Right-of-way monuments may be placed to provide a permanent reference for re-establishing the centerline and right-of-way line. Right of way monuments should be located and punched so the center is on the right of way line. Details of a standard right-of-way monument are shown in the Department's Standard Construction Details.

10.6 FENCING

It is Department policy that installation of fences should normally be considered under one or more of the following conditions:

- For access restrictions on interstate or other designated controlled access highways,
- Replacement fence where an existing fence was removed because of right-of-way widening, or
- For locations where there is a documented need for fencing i.e. pedestrian and bicyclist safety or right-of-way negotiations.

The location of fencing depends on who will assume the ownership and maintenance responsibility, 1 ft [0.3 m] inside the right of way for DelDOT and 1 ft [0.3 m] outside if by others. The responsibility for installing fencing varies. Fencing required for DelDOT purposes will normally be shown on the contract plans and included as a bid item for the contractor. The type of fencing will depend on the characteristics and use of the adjacent property.

Installation of replacement fencing or new fencing as the result of negotiating easements or property takings are normally included in the right-of-way settlement agreement. This agreement provides for the affected property owner to be responsible for construction and maintenance of any new fence, with installation reimbursement

by the State. Occasionally, the right-of-way agreement will specify that this fencing be included as a contract item. Locations and quantities of fencing to be constructed in this manner are determined by Real Estate and coordinated with the designer.

10.7 UTILITY ADJUSTMENTS

This section is a general discussion of a project's relationship with utilities. The specific details are covered in Chapter 11, Plan Development. In addition, the Department has published the *Utilities Design Manual*, which clearly sets forth policies and procedures regarding the relationships among the Department, the public, and private utilities in Delaware. Much of the manual is related to the accommodation of utilities within the highway right-of-way and adjustments initiated by the utilities. Other sections define responsibilities and procedures related to needed utility adjustments resulting from proposed highway improvement projects. It is this second category that is of concern to designers.

In storm drain design, it is often possible to avoid conflicts with underground utilities by making minor adjustments in the line or grade of the storm drain. The designer should consider conflicts with any utility in making the final design to minimize relocations and conflicts. Relocations of utilities frequently delay the progress of construction and so should be avoided where possible. As further described in the manual, costs for relocating and adjusting utilities may either be the affected utility's or the Department's responsibility. The designer needs to recognize that no matter which party is responsible, the cost will be directly or indirectly passed on to the user.

The procedures and responsibilities for utilities adjustments set forth in the *Utilities Design Manual* are briefly summarized below in terms of the four phases of plan development.

10.7.1 SURVEY PLANS

The designer prepares survey plans showing the approximate project limits, existing detail, and project number(s) assigned to the project. Two sets of survey plans are provided for each utility and are transmitted by the Utilities Section.

The Utilities Section returns marked-up plans showing complete information on existing and abandoned facilities in the area. The information provided includes whether the facilities are aerial, surface or underground, sizes of pipes, numbers of conduits, approximate depths, and any private or commercial easements.

10.7.2 PRELIMINARY PLANS

The designer prepares preliminary plans showing the proposed alignment, profile, drainage, signal pole location, clear zones, right-of-way taking, existing utilities from survey plan data and other details. Two sets of plans are submitted by the Utilities Section to each potentially affected utility company, who reviews these plans and returns them showing their proposed work plan.

The designer, assigned utility coordinator and, in some cases, the utility company's representative review these plans to determine if the exact horizontal and vertical location of certain facilities are an important part of the design considerations. Where major conflicts with underground utilities appear possible, the Utilities Section arranges for determining the exact field location of the existing underground installations, typically with taking test pits. The Utilities Section submits this information to the designer. The designer and utility coordinator determine which underground or aerial conflicts cannot be avoided and discuss alternatives with the affected utility company. It is very important that the potential impact on scheduling, current project estimate and the responsibility for preparing the design plans

and quantities as well as the actual construction be identified at this time.

10.7.3 SEMI-FINAL PLANS

Semi-final plans are prepared showing the final centerline, profile, drainage, right of way and other details.

Two sets of semi-final plans for each utility are provided to the Utilities Section. These plans are delivered to each affected utility. A representative of the Utilities Section coordinates a review of the project's affect on each utility's facilities.

The utilities plot their proposed underground relocation work on the plans, and the information is returned to the designer through the Utilities Section. The Utilities Section arranges with the utility for any needed relocation work and formal agreements. The scope and schedule for the work is included in the project's utility statement that is furnished to the designer for preparation of the P. S. and E. package.

10.7.4 P. S. AND E. PLANS

When the final plans are completed and the project is advertised, one set of final plans is forwarded to each utility. If it has not previously been notified to start the adjustment, the utility is directed by letter from the Utilities Section to order materials and begin making the changes or alterations to their facilities.

10.8 SIDEWALKS

10.8.1. GOALS AND OBJECTIVES

Sidewalks are an integral part of the Department's transportation infrastructure program. They facilitate and encourage safe and convenient pedestrian travel within communities and among different land uses. They provide safe and reasonable access to public transportation and other alternative modes of transportation, thereby helping alleviate vehicular traffic and reduce emissions. They also reinforce the Americans with Disabilities Act (ADA) by increasing the access opportunity for mobility impaired individuals.

The incorporation of sidewalks and pedestrian amenities also provides better land use and transportation connections, thus encouraging more trips on foot, improve access to transit, improving access to adjacent land uses and communities, conserving energy, and enhancing the Department's vision for multi-modal and inter-modal transportation systems.

In addition, by providing this transportation option, the installation of sidewalks can be an effective means in reducing automobile dependence and use. It will assist Delaware's mission toward cleaner air under the requirements of the Clean Air Act Amendments of 1990.

With the addition and installation of proper sidewalks and pedestrian amenities, safety, accessibility, ridership, and more favorable perceptions of public transit service can be increased. Communities will also be able to safely link to other land uses and transportation modes, resulting in better and more desirable neighborhoods and land development practices.

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DOVER, DELAWARE 19903

NATHAN HAYWARD III
SECRETARY

MEMORANDUM

To: All Recipients of the January 2004 Road Design Manual

From: Kevin T. Canning
Quality Engineer KC

Date: March 10, 2005

Subject: **2004 Road Design Manual - Revision**

Attached you will find a revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective March 14, 2005. The changes are as follows:

- Sheet 9-14– In Figure 9-3, the maximum lift thickness for Type C surface mix has been corrected to 2 inches (50 mm).

Please replace the page in your manual with the attached sheet.

Any comments or questions regarding the 2004 Road Design Manual should be addressed to Linda M. Osiecki.

KTC:los
Enclosure
cc: Carolann Wicks, Chief Engineer
Dennis M. O'Shea, Assistant Director – Design
Michael Simmons, Assistant Director, Project Development, South Region
Michael Angelo, Assistant Director, Project Development, North Region
Linda M. Osiecki, Consistency Engineer, Quality

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and classes of flexible pavement, base course, and subbase.

Figure 9-2
Layer Coefficients

Material Type	Layer Coefficient
Type C Surface Mix	0.40
Type B Binder Course	0.40
Bituminous Concrete Base Course	0.32
Soil Cement	0.20
Graded Aggregate Base Course	0.14
Select Borrow	0.08

9.5.6 MINIMUM LIFT THICKNESS

Although the equations allow for a great number of thickness variations, there are the practicalities of constructing and maintaining a facility, which must be considered. Depending upon the material being placed, there are minimum and maximum limits in the placement depth that are practical for the available equipment to compact and are economical.

Minimum lift thickness for hot-mix is recommended to be three times the nominal aggregate size in the mix. The following are the practical maximum and minimum lift thickness (compacted) that are to be applied to the materials normally used in constructing a flexible pavement section.

9.5.7 TEMPORARY PAVEMENTS

It is not practical to attempt to follow the formalized AASHTO procedures for design of temporary pavements such as needed for detours during construction. Variations in speed and ease of placement as well as the anticipated required service life of the detour significantly affect the economic justification for the structural design.

When temporary pavements are needed designers should closely coordinate with the Materials and Research Section in the development of a practical pavement design based on knowledge of local conditions and engineering judgment.

Figure 9-3
Lift Thickness

Type of Material	Minimum Lift Thickness	Maximum Lift Thickness
Type C Surface Mix	1-¼ inches [30 mm]	2 inches [50 mm]
Type B Binder/Base	2-¼ inches [60 mm]	3 inches [75 mm]
Bituminous Concrete Base Course	3 inches [75 mm]	6 inches [150 mm]
Graded Aggregate Base Course	4 inches [100 mm]	8 inches [200 mm]
Soil Cement	4 inches [100 mm]	6 inches [150 mm]
Select Borrow	4 inches [100 mm]	8 inches [200 mm]
Open Graded	1 inch [25 mm]	1 inch [25 mm]

9.6 DESIGN FOR RIGID PAVEMENTS

Rigid pavements consist of a Portland cement concrete slab on a subbase course. The design procedure consists of developing an effective modulus of subgrade reaction based on subbase treatment and thickness, determine the slab thickness, allowing for any stage construction, adjusting for adverse environmental conditions, determining type of joints, joint sealant, and the required reinforcement.

and classes of flexible pavement, base course, and subbase.

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Graded Aggregate Base Course	4 inches [100 mm]	8 inches [200 mm]
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CAROLANN WICKS, P.E.
SECRETARY

TO: All Recipients of the January 2004 Road Design Manual

FROM: Kevin T. Canning AS for K.C
Quality Engineer

DATE: July 5, 2006

SUBJECT: **2004 Road Design Manual – Revision**

Attached you will find a revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective July 6, 2006. The change is as follows:

- Sheet 3-3 in the third full paragraph, the title of the publication " A Policy on Design Standards – Interstate System" was corrected.

Please replace the page in your manual with the attached sheet. Any comments or questions regarding the 2004 Road Design Manual should be addressed to Linda M. Osiecki.

KTC:los:l
Enclosure
cc: Robert Taylor, Chief Engineer
Dennis M. O'Shea, Assistant Director, Design
Michael Simmons, Assistant Director, Project Development South
Drew Boyce, Assistant Director, Project Development North
Linda M. Osiecki, Consistency Control Engineer
File (06094)

DelDOT

primarily because the motorist expects a higher standard and will drive the facility accordingly.

The design standard to be evaluated for an exception should be a major design element that will control the design. Major controlling design elements include:

- Design speed;
- Through lane and auxiliary lane widths;
- Shoulder widths;
- Stopping sight distance on vertical and horizontal curves;
- Horizontal alignment (radius of curve);
- Vertical alignment;
- Minimum and maximum grades;
- Cross slopes;
- Superelevation rate;
- Horizontal clearance;
- Vertical clearance;
- Bridge width; and
- Structural capacity.

The project scope, funding, functional classification and other factors influence the selection of appropriate design standards. Variances from these standards need some level of documentation for project files. New construction and reconstruction projects require greater detail on the rationale for departure from the established design standards and must be thoroughly documented.

New construction, reconstruction, and projects on the interstate and the NHS are expected to be in conformance with the appropriate standards and exceptions should be rare. For all projects on the NHS, except preventive maintenance, the standards are those in the current edition of the AASHTO publication *A Policy on Geometric Design of Highways and Streets.* For all projects, except preventive maintenance, on the interstate system, the design criteria to be met are contained in the current edition of *A Policy on Design Standards—Interstate System.*

Design exceptions for projects on the NHS and interstate having FHWA oversight as established in the current *Delaware Transportation/Federal Highway Administration Stewardship Agreement to Implement the Flexibility of Provisions of 23 United States Code Part 106* must have FHWA approval. The Chief Engineer approves design exceptions for state administered projects. Figure 3-2 is a guide format for developing a design exception request.

An exception for design speed should not be sought as this element establishes most if not all of the other parameters to be met. As discussed in this chapter and in several other sections in this manual, design speed is an achievable speed selected by the designer based on the various factors the designer must consider. Design elements that can not be met within that selected design speed should be supported by seeking a design exception.

The need for exceptions to the standards must be identified early in the project development phase in order that approvals or denials will not delay completion of the design or require extensive redesign. However, the need to evaluate a lower design value may arise at any time during the design process and needs to be addressed expeditiously. Thorough documentation is essential. The need for exceptions should not be viewed as normal or routine. Use the forms in this chapter to document decisions on design criteria and as a basis for developing and documenting requests for exceptions. The designer must provide the supporting rationale. The primary focus of the request should be highway safety. The designer's proposal should be the best practical alternative that considers whether or not other controlling design elements will be adversely affected.

Depending upon the significance of the request, the support information may include some or all of the following:

- Existing roadway characteristics,
- Required and proposed design criteria;
- Cross section or geometric figures comparing the existing and proposed conditions;
- Supporting calculations and cost analysis;
- Analysis of accident records;

primarily because the motorist expects a higher standard and will drive the facility accordingly.

The design standard to be evaluated for an exception should be a major design element that will control the design. Major controlling design elements include:

- Design speed;
- Through lane and auxiliary lane widths;
- Shoulder widths;
- Stopping sight distance on vertical and horizontal curves;
- Horizontal alignment (radius of curve);
- Vertical alignment;
- Minimum and maximum grades;
- Cross slopes;
- Superelevation rate;
- Horizontal clearance;
- Vertical clearance;
- Bridge width; and
- Structural capacity.

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- Analysis of accident records;

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
800 BAY ROAD
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DOVER, DELAWARE 19903

CAROLANN WICKS, P.E.
SECRETARY

December 18, 2006

Dear Road Design Manual Holders:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual – November 2006 Revision

This is the formal issuance of the November 2006 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective December 18, 2006. You can download this revision from DelDOT's web site: http://deldot.gov/static/pubs_forms/manuals/road_design/index.shtml. This web site also includes a file of all of the revised pages that can be used for printing, and a file with the changes highlighted on the previous version of the manual.

- Pages 1-2, 3-6, 3-7, 3-8, 3-13, 4-17, 5-3, 5-4, 5-5, 5-7, 5-8, 5-13, 5-17, 7-1, 7-3, 7-5, 7-7, 7-9, 7-14, 7-15, 7-16, 7-18, 7-19, 7-20, 7-28 and 10-64: Changes have been made for the 2004 AASHTO "A Policy on Geometric Design of Highways and Street", commonly referred to as the Green Book. This includes reference to this publication, page references and values, as well as some corrections to the text.
- Pages 2-2, 7-6, 7-10, 7-11, 7-24, 10-63, and 10-64: Corrections have been made to the text for formatting, missing labels and grammar.

Please replace the pages in your manual with the appropriate sheets. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely yours,

Dennis M. O'Shea
Assistant Director – Design:

cc:
Robert Taylor, Chief Engineer
Michael Simmons, Assistant Director, Project Development, South Region
Drew Boyce, Assistant Director, Project Development, North Region
Kevin T. Canning, Quality Engineer
Linda M. Osiecki, Program Manager, Quality

DelDOT

Factors to be recognized and addressed in the design process include:

- Selecting design guidelines that will provide for the safety of the user.
- Identifying the need for access to the facility, as well as mobility along the facility.
- Preservation or enhancement of historic sites and districts.
- Protection or enhancement of existing environmental assets.
- Identifying the economic needs of and impacts to the affected community or area.
- Developing an understanding of the social context of the community and area within which the facility exists.

Ensuring that a project design will have a balance of these factors is the result of a continuous and meaningful public involvement process. Throughout the public involvement process, the designer must make sure the purpose for the project as established in the project initiation is fulfilled while understanding and addressing the needs of the community. By doing this the introduction of new or additional issues during the final design phase that may result in delays and/or redesign will be minimized.

The basic design tools available to the designer are this manual and the 2004 American Association of State Highway Transportation Officials, (AASHTO) "*A Policy on Geometric Design of Highways and Streets*" commonly referred to as the "Green Book". In addition designers need to refer to other related publications and guidelines prepared by the Federal Highway Administration (FHWA), the Transportation Board (TRB), and other recognized experts in the transportation field. The principal publications are:

- AASHTO's *Roadside Design Guide*,
- AASHTO'S *Context Sensitive Design for Integrating Highway and Street Projects with the Community and the Environment*,
- Transportation Research Board's (TRB) *Highway Capacity Manual*,
- TRB's *Special Report 214 Designing Safer Roads*,
- FHWA's *Manual on Uniform Traffic Control Devices*,
- FHWA's *Flexibility in Highway Design*, and
- DelDOT's *Traffic Calming Manual.*

In addition, there are numerous other documents, particularly related to highway drainage and intersection design that must be referenced in designing a project.

Using these documents, the Road Design Manual was developed with emphasis on standards and practices that have proven to be successful in this state. The flexibility to design a project that will meet the expectations of the user, the community and Department exists in the Green Book and in the standards found in this manual. Published design standards have a measure of flexibility, usually stating a maximum and minimum value. Many of these values are empirically based using mathematical modeling techniques with assumed roadway surface conditions, driver reaction times and adverse weather conditions.

Designers need to recognize that there is a difference between the strict application of design standards found in the tables and charts versus providing consistency in design. The design should ensure there is consistency in application of the standards that allows the driver to react in a consistent and predictable manner when encountering similar roadway conditions. However, in responding to the many issues that arise on each project, there is a need for flexibility in

facility may result in increased expenditures that could be spent more effectively by improving additional road sections.

- Context. Design features should be selected that are in balance with the social context of the community and surrounding area. This is accomplished by gathering and including information from the public throughout the design process. A context sensitive design advances the objectives of safety, mobility, enhancement of the natural environment, and preservation of community values. Projects that improve the livability of the community or quality of the natural environment are considered context sensitive.

2.2 LEVELS OF SERVICE

In general terms, the level of service of a highway facility may be influenced by many factors, including surface condition and ride-ability. From the standpoint of design controls, the level of service is principally related to the ease and convenience with which the highway facility can serve the expected volumes of traffic.

The Transportation Research Board's (TRB) *Highway Capacity Manual* presents a thorough discussion of the level of service concept. Six levels of service are established from level A (the highest) through level F (the lowest).

The general characteristics of the various levels of service are:

- Level of Service A – free-flowing traffic; users virtually unaffected by other traffic, able to select desired speeds and maneuver unrestricted.
- Level of Service B – reasonably free traffic flow; users able to select desired speeds, but with a slight decline in freedom to maneuver.
- Level of Service C – stable flow, but operation of individual users is significantly affected by traffic; ability to select speeds is reduced and maneuvering requires substantial vigilance by the users.
- Level of Service D – high density approaching unstable flow. Speeds and freedom to maneuver are severely restricted. Small increases in traffic flow will generally cause operational problems.
- Level of Service E – operating conditions at or near capacity with unstable flow. All speeds at a low and relatively uniform value. Freedom to maneuver is extremely difficult.
- Level of Service F – forced or breakdown flow. Traffic exceeds capacity causing queues with stop-and-go waves, and operations are extremely unstable.

The traffic flow rates that can be served at each level are termed "service flow rates." Once a level of service has been identified as applicable for design, the accompanying service volume logically becomes the design service flow rate, implying that if the traffic volume using the facility exceeds that amount, operating conditions will fall below the level of service for which the facility was de-signed. A guide for selecting design levels of service is shown in Figure 2-1.

More detailed guidelines for selecting appropriate levels of service are given in AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the "Green Book") and TRB's *Highway Capacity Manual* (HCM).

Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page 384, Exhibit 5-5;
- Collector Roads and Streets (Rural) — page 425, Exhibit 6-5;
- Collector Roads and Streets (Urban) — page 433;
- Arterials (Rural) — page 448, Exhibit 7-3;
- Divided Arterials (Rural) — page 455;
- Urban Arterials — page 472;
- Freeways — page 504.

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-5, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smoother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages 167 to 174, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

Vertical curvature, horizontal curvature, roadside obstructions, or any combination of these elements can restrict sight distance. Procedures for checking available sight distances are described in the Green Book, pages 127-131.

3.2.4 PASSING SIGHT DISTANCE

Consideration of passing sight distance is limited to two-lane, two-way highways on which vehicles frequently overtake slower-moving vehicles and the passing operation must be accomplished on a lane used by opposing traffic.

Passing sight distance for design is determined on the basis of the length needed to accomplish the passing maneuver. Derivation of the required distance is described in the Green Book, pages 118-126. AASHTO recommends that, “In designing a highway these distances should be exceeded as much as practical ...”

These distances for design should not be confused with other distances used as warrants for placing no-passing pavement markings on completed highways. Values shown in the MUTCD are substantially less than the design distances and are derived from traffic operation control needs based on assumptions different from those for design.

Because of vertical and horizontal sight limitations, nearly all two-lane highways have some no-passing restrictions. In rolling terrain, the proportionate amount of no-passing sections usually becomes greater. Normally it is impracticable to attempt to provide passing sight distance throughout the entire length of a project. The principal design consideration is to try to provide adequate passing opportunities as frequently as possible.

There are no fixed values for the frequency of passing sections. Experience shows that highway capacity is measurably reduced when a significant percentage of a section of highway is restricted to sight distances less than 1500 ft [500 m]. Highways with high traffic volumes will require a higher proportion of passing opportunities than those with low traffic volumes. Where an analysis shows that a lack of passing sight distances has reduced capacity to near or below the expected traffic volumes, it is necessary to consider adjustments in the alignment and grade, or to provide additional lanes.

3.3 STANDARDS BASED ON TRAFFIC VOLUMES

Standards not directly related to design speed are influenced primarily by traffic volumes. Tables for these standards shown in the tables at the end of this chapter reflect variations by traffic volume ranges.

3.3.1 NUMBER OF LANES

The number of lanes required for any highway is directly related to the facility's traffic volume and desired level of service. But there are no simple, fixed criteria for these relationships. The recommended number of lanes is normally obtained through the project development process.

The *Highway Capacity Manual* gives two very general guidelines for determining the need for additional lanes. These numbers are based on long sections of roadway with uninterrupted traffic flow having the highest standards for design controls (horizontal and vertical geometrics and cross-sectional elements), ideal weather conditions, daylight, etc.

1. Under ideal conditions, a two-lane rural highway can accommodate about 900 passenger vehicles (two-way) per hour with a reasonably high level of service if there are adequate passing opportunities and no long, steep grades. Considerably more vehicles can be accommodated if motorists are willing to accept a lower level of service, a greater degree of congestion and lower operating speeds.

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the safest available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-4), the Design Criteria Form (Figure 3-5) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there are steep grades accompanied by considerable no-passing distances.

More detailed guidelines and criteria for the design of grades, including critical lengths of grades and minimum and maximum grades are presented in Chapter Five - Alignment and Superelevation and the Green Book, pages 231-250. The maximum grades should be used infrequently, only as dictated by severe terrain conditions. When it is necessary to use maximum grades, the designer should check other design criteria and roadside features that may be improved to minimize the impact of using the higher design grade.

3.3.8 BRIDGES

The designer should coordinate with the Bridge Design Section when determining vertical clearances. A minimum vertical clearance for roads over interstate, U.S. and state routes is 16.5 ft [5 m]. Pedestrian bridges and overhead sign structures must have an extra 1 foot [0.3 m] of clearance, a total of 17.5 ft [5.3 m]. These clearances allow for a 4 in. [100 mm] future resurfacing.

3.3.9 MEDIANS

Geometric criteria for medians on multilane divided highways are discussed in Chapter Four.

A width of 40 ft [12.0 m] or more for depressed medians permits adequate drainage design with flat slopes. A median width of at least 50 ft [15.0 m] can safely store a school bus. Wider medians are desirable where right-of-way permits allowing for the placement of a median bridge pier or overhead sign structure without the need for barrier protection. Wider medians should also be considered where there is a potential for adding travel lanes in the median to meet future traffic demand. Also see the Green Book pages 456 and 457 for further discussion on this subject.

Where flat longitudinal slopes on the roadway are encountered, the cross slopes of the median may be varied to increase the longitudinal slope of the median ditch. For example, the cross slope may be kept very flat (10:1 or flatter) at the upper end of the drainage area and steeper (6:1) at the lower end.

4.3.4 MEDIAN BARRIERS

For divided highways with large traffic volumes and high operating speeds, a wide, depressed median is the best choice. Under some conditions this is not practicable, and a flush or raised median must be provided. But in this case, some type of physical barrier must be placed in the median to prevent out-of-control vehicles from crossing into opposing traffic lanes.

Several types of physical median barriers can be designed. Criteria for median barriers are discussed in Chapter Ten-Miscellaneous Design and the *Roadside Design Guide*.

4.3.5 MEDIAN OPENINGS

The design of median openings and channelization for left turns is included with the discussion on intersection design in Chapter Seven.

Figure 5-1
Minimum Radius for Open Highway Conditions and Superelevation Rate of 4%

US Customary		Metric	
Design Speed (mph)	Radius (ft)	Design Speed [km/h]	Radius [m]
15	42	20	8
20	86	30	22
25	154	40	47
30	250	50	86
35	371	60	135
40	533	70	203
45	711	80	280
50	926	90	375
55	1190	100	492
60	1500		

Figure 5-2
Minimum Radius for Open Highway Conditions and Superelevation Rate of 6%

US Customary		Metric	
Design Speed (mph)	Radius (ft)	Design Speed [km/h]	Radius [m]
15	39	20	8
20	81	30	21
25	144	40	43
30	231	50	79
35	340	60	123
40	485	70	184
45	643	80	252
50	833	90	336
55	1060	100	437
60	1330	110	560
65	1660	120	756
70	2040	130	951

3. For multi-lane divided highways with independent roadways or relatively wide medians, independent horizontal and vertical controls are established at the centerline of each roadway.

The relationships between these control line locations and the pivot points for superelevation of horizontal curvature are described in Section 5.3.

5.1.3 TYPES OF CURVES

The types of curves used in designing horizontal curvature may be simple circular curves, spiral transition curves or compound curves. Circular curves use a uniform radius for the entire distance between adjacent tangent sections. Spiral transition sections more closely replicate the vehicle and driver's behavior when entering a curve. They are introduced at each end of the circular curve to gradually ease the driver into and out of curves without a sharp break at the tangent sections. This is particularly noticeable with relatively sharp curves and higher vehicle operating speeds. Compound curves are most commonly used for turning roadways where it is necessary to fit the curve to the inside edge of the design vehicle's swept path. When the design speed of a turning roadway is 45 mph [70 km/h] or less, compound curvature can be used to form the entire alignment of the turning roadway. However, the exclusive use of compound curves can increase the right-of-way impacts.

Although circular curves are normally used in the design of Delaware roadways, using spiral transitions may be considered as described in the Green Book, pages 184 to 192.

Where spiral transition curves are to be used, right-of-way lines should not be defined as a spiral curve paralleling the centerline. Instead, the right-of-way should be described with a circular curve or compound circular curve of a similar shape. A practical guide for the length of a spiral is the length required for superelevation runoff.

5.1.4 SIGHT DISTANCE ON HORIZONTAL CURVES

An important element in ensuring driver safety and maintaining a roadway's operational efficiency is providing adequate sight distance—the length of roadway ahead visible to the driver. Sight distance applies to four conditions that may arise when setting a project's horizontal alignment:

(1) Is adequate distance available to stop?

(2) Is there adequate opportunity and length available for passing on two-lane roadways?

(3) Is there adequate distance for drivers to react when approaching complex decision points?

(4) Has the selected criteria for measuring these distances been applied to the selected design?

Providing adequate sight distance is also important in the design of intersections, in particular, those in rural areas. These locations tend to be less safe than urban ones, primarily because of higher speeds and lack of driver awareness. Providing at least the minimum sight distance will play an important role in reducing these occurrences.

5.1.4.1 STOPPING SIGHT DISTANCE

The designer must check sight distance across the inside of horizontal curves. Sight obstructions such as walls, concrete safety barriers, bridge parapets, cut slopes, vegetation and buildings may limit sight distance on curves. Where these obstructions cannot be removed or permanently controlled, adjustment in the normal cross section or a change in alignment may be required to provide and assure continuation of adequate sight distance. For areas within a project that may cause confusion or delay a driver's reaction time i.e. multiple decision points, it may be necessary to check the decision sight distance also.

Minimum stopping sight distance for each design speed is shown in Chapter Three-Design Standards. The sight line is a chord of the curve. The applicable stopping sight distance is checked by measuring along the centerline of the inside lane around the curve. See the Green Book, pages 224-228 for the design and evaluation of stopping sight distances on horizontal curves. Horizontal sight distance is based on the formula:

$$HSO = R\left[\left(1 - \cos\frac{28.65\,S}{R}\right)\right]$$

Where:

S = Stopping sight distance, ft [m]
R = Radius of curve, ft [m]
HSO = Horizontal sightline offset, ft [m]

Where the obstruction is a cut slope on the inside of the curve, it is necessary to know the critical height of vegetation on the slope for measuring the middle ordinate distance. Because the height criteria for stopping sight distance are 3.5 ft [1,080 mm] for the eye and 2 ft [600 mm] for the object, a height of 2.75 ft [840 mm] may be assumed as the midpoint of the line of sight where the cut slope usually obstructs sight. In some cases, retaining walls, concrete median safety barriers, and other similar features constructed on the inside of curves may be sight obstructions and need to be checked for stopping sight distance.

Solutions to sight distance problems on horizontal curves might be removal of obstructions, flattening the curves and flattening or benching cut slopes. It should be kept in mind that stopping sight distances greater than the minimum should be used for design. Minimum stopping sight distance values may be used only if greater values cannot be obtained without undue costs. On new construction, the stopping sight distance at any loca-

tion shall never be less than the minimum standard for stopping sight distance for the selected design speed. Designs for new construction and reconstruction projects that do not meet these standards must have a design exception approved by the Chief Engineer.

5.1.4.2 PASSING SIGHT DISTANCE

The minimum passing sight distance for a two-lane road is about four times greater than the minimum stopping sight distance at the same design speed. To provide the greater passing sight distance, clear sight areas on the insides of curves must be considerably wider. Often this is not practicable. It is necessary to acknowledge and accept no-passing zones.

Passing sight distance depends on the eye height of 3.5 ft [1,080 mm] and object height of 3.5 ft [1,080 mm]. The sight line to the center of the area inside a curve is about 0.75 ft [240 mm] higher than the stopping sight distance.

Perhaps the simplest way to measure passing sight distance is directly from the plans, using a straightedge. Potential obstructions are plotted on the plans. In the case of cut slopes, a dotted line is plotted for the horizontal distance from the centerline of the inside lane to a point on the cut slope 4 ft [1.2 m] above the traffic lane. Because vegetation also blocks vision, its anticipated height must be included in the 4 ft [1.2 m]. The straight edge is placed along the edge of the obstruction (or dotted line), and the intersection with the centerline identifies the sight distance.

Where horizontal curves and vertical curves occur at the same general location, the sight distances for each must be considered together. At least the minimum stopping sight distance must be provided for each. Efforts to provide passing sight distance for one might be completely negated by a no-passing zone situation for the other.

For more information see the Green Book, pages 118 to 131.

5.1.4.3 DECISION SIGHT DISTANCE

Drivers frequently are called on to make decisions concerning vehicle operations. Occasionally, the characteristics of the horizontal alignment can adversely affect the ability to make these decisions. Examples of this include:

- ***Proximity to a Curve***. It is important that the driver has a complete or partial view of the curve ahead to indicate the direction of curvature. With some combinations of vertical and horizontal curvature, the curve may come as a surprise and the driver may have difficulty reacting properly.
- ***Curve Signing***. To be effective, curve signing must be located a considerable distance ahead of the curve. The use of short tangents between curves results in inadequate length for proper signing. Where the design speed of the curve is equal to or greater than the legal posted speed, the length of the tangent should be at least 300 ft [90 m] plus the required distance for superelevation transition.
- ***Route Continuity***. When a driver approaches a diverging roadway situation, such as a Y intersection, an exit ramp on a curve, or a flat-angle intersection, the main route should be distinctly emphasized with sufficient sight distance to eliminate any uncertainty on the part of the driver.

The Green Book, pages 115-117, provides more details and tables of calculated values for checking decision sight distance.

5.1.5 COORDINATION WITH VERTICAL ALIGNMENT

Curvature and grades should be in proper balance. Emphasis on a tangent alignment is not desirable when it results in extremely steep or long grades. An emphasis on flat grades is not desirable when it results in excessive curvature. A compromise between the two extremes is the best approach.

Refer to the Green Book for the maximum grades permitted for various combinations of functional classification, traffic volume and terrain. The maximum grades should be used only where absolutely necessary. Grades much flatter than maximum normally should be used.

For short grades less than about 500 ft [150 m] in length, the maximum gradient may be one percent steeper than the values shown in the tables.

5.2.3 MINIMUM GRADES

Minimum grades are primarily related to the need for adequate drainage. For uncurbed pavements that are adequately crowned to drain laterally, relatively flat or even level profile grades may be used. With curbed pavement, the minimum longitudinal grade in usual cases should be 0.5 percent. With a high-type pavement accurately crowned on a firm subgrade, a longitudinal grade of about 0.35 percent may be used. Even on uncurbed pavements, it is desirable to provide a minimum of about 0.35 percent longitudinal grade because the lateral crown slope originally constructed may subsequently be reduced as a result of irregular swell, pavement structure consolidation, maintenance operations or resurfacing. Use of flatter grades may be justified in special cases.

5.2.4 MINIMUM DITCH GRADES

Special attention should be directed to minimum ditch grades. Any ponding of water in the side ditches, particularly on expansive soils, has a very detrimental effect on the subgrade. To ensure continuing flow, ditch grades should be sloped at least 0.5 percent–preferably steeper. This may require some special warping of ditch grades where the roadway profile cannot be adjusted accordingly. A minimum depth of ditch has been established at 2.5 ft [800 mm] below the elevation of the hinge point between the shoulder and frontslope to assure proper drainage of pavement base and subgrade. In superelevated sections both the ditch grade and bottom width may have to be adjusted in order to prevent water ponding onto the shoulder or traveled way.

5.2.5 CRITICAL LENGTH OF GRADE

From the standpoint of vehicle operating characteristics and the effect on highway capacity, the steepness of the grade is not the only factor to be considered. The length of the grade can become a critical factor and must also be considered.

The term “critical length of grade” is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For a given grade, lengths less than critical ones result in acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than critical ones, design adjustments such as change in location to reduce grades or addition of extra lanes should be made. It is recommended that a 10 mph [15 km/h] speed reduction be used as the general guide for determining critical lengths of grades. The Green Book, pages 242 and 243, provides curves showing the critical lengths of grade resulting from various combinations of percent upgrade and designated speed reductions.

On roads with moderate to heavy traffic volumes, where critical lengths are approached or exceeded, and passing opportunities are limited, long lines of smaller vehicles will accumulate behind the slower vehicles. This reduces both the operating speed and highway capacity and, consequently, the level of service. Consideration should be given to providing climbing lanes. A capacity analysis should be conducted to determine whether the addition of a climbing lane is warranted. Procedures for such an analysis are shown in Chapter Ten of the *Highway Capacity Manual*. Factors considered in the analysis include:

- Desired level of service,
- Lane widths and lateral clearance,
- Percent of trucks and buses,
- Passing sight distance,

- Steepness and length of grades,
- Volume/capacity ratio, and
- Service volume.

5.2.6 CLIMBING LANE CRITERIA

The need for climbing lanes in Delaware is seldom warranted. The Green Book pages 241 to 250 gives a through explanation for the design of these lanes.

5.2.7 VERTICAL CURVES

Vertical curves are used to effect gradual changes between tangent grades at their point of intersection. They have the properties of a simple parabolic curve. The vertical offsets from the tangent grade vary with the square of the horizontal distance from the curve end (point of tangency).

Vertical curves that are offset below the tangent are termed crest vertical curves. Those that are offset above the tangent are termed sag vertical curves. Examples of each curve type are shown in Figure 5-4.

The minimum lengths of crest vertical curves are determined mainly by sight distance requirements. These lengths generally are satisfactory from the standpoint of safety, comfort and convenience. An exception may be at decision areas, such as intersections and approaches to ramp exit gores, where adequate sight distance requires longer lengths.

Passing sight distance seldom can be attained on a crest vertical curve simply by lengthening the curve. Excessively long vertical curves often reduce the length of passing opportunities on the adjacent tangent sections on either side of the crest. They also can adversely impact roadway and roadside ditch drainage systems.

Sag vertical curves use four different criteria for determining their lengths:

(1) headlight sight distance,

(2) passenger comfort,

(3) drainage control, and

(4) general appearance.

The primary control used in design is headlight sight distance.

5.2.8 VERTICAL CURVE DESIGN

The principal concern in designing vertical curves is to ensure that at least the minimum stopping sight distance is provided. The values set forth in the design standards for stopping sight distance are also applied to vertical curves. Refer to the Green Book pages 265 to 280 for more design detail.

For crest vertical curves, the design eye height is 3.5 feet [1,080 mm] and the object height is 2.0 ft [600 mm]. The crest of the curve should not obstruct the line of sight.

Nighttime driving conditions govern sag vertical curves. The sight distance control is the height of headlight and the distance illuminated to an object rather than driver eye height. The distance illuminated is that of a headlight beam with an assumed upward divergence of 1 degree and headlight mounting height of 2 ft [600 mm]. Equations found in the Green Book are used to determine these values for various design speeds. For overall safety, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. The values in Figure 5-6 were developed using the design stopping sight distance as the light beam distance.

For passing sight distance, the controls are different than for stopping sight distance. The design height of the eye remains at 3.5 ft [1,080 mm], but the height of the object (oncoming car) is increased to 3.5 ft [1,080 mm].

By analyzing the requirements relating to sight distances and the characteristics of the curve, determinations can be made as to the minimum permissible length of curve for particular situations. A ride control criterion for vertical curve length of not less than three times the design speed in mph [0.6 times the design speed in km/h] is recommended for comfort.

The minimum length of a vertical curve is computed by the following formula:

on street sections usually are considerably shorter than for comparable locations on rural roads.

Where controlling factors are not severe, the normal practice of carrying the profile grade on the centerline or on the median edges of pavement will work satisfactorily. Where outside controls are significant, it may be necessary to supplement the main profile with other elevation controls, such as gutter-line profiles or top-of-curb profiles. Where this is necessary, the supplemental controls should be clearly shown on the typical sections, profiles, and grades and geometrics sheets.

Special attention must be given to existing features when designing urban grades. This is particularly true in the case of private driveways when a street is being widened. With even moderate driveway grades, up or down, angular breaks must be kept flat enough with adequate clearance so that the undercarriage or bumpers of vehicles will not drag. Reference should be made to the Department's publication *DelDOT Entrance Manual*.

Where roadside development is extensive and the general elevation is higher on one side than on the other, an unsymmetrical section may be required. The crown point (and profile grade) may be offset from the centerline so the total drop from the crown line to the gutter line will be more than normal on one side and less than normal on the other. However the location of the crown point must be at the edge of the travel lane.

5.3 SUPERELEVATION

The transitional rate of applying superelevation into and out of curves is influenced by design speed, degree of curvature and number of lanes. Introducing superelevation permits a vehicle to travel through a curve more safely and at a higher speed than would be possible with a normal crown section. For a given degree of curvature, a steeper superelevation is required for a higher design speed than is needed for a lower design speed. For a given design speed more superelevation is needed through sharp curves than for relatively flat curves.

The maximum rates of superelevation used on roadways are controlled by four factors:

(1) Climate conditions (i.e. frequency of ice and snow);

(2) Terrain conditions (i.e. flat or rolling);

(3) Type of area (i.e. rural or urban); and

(4) Frequency of slow-moving vehicles.

Basic design controls for superelevation are presented in Chapter Three. Rural roadways are usually designed with a maximum superelevation rate of 6 percent but it may be appropriate to use a rate of 8 percent. Urban roadways are normally to be designed with a superelevation rate of 4 percent. Superelevation may be omitted on low-speed urban streets subjected to severe constraints. The selected superelevation rate establishes the minimum permissible radius of curve based on a project's design speed.

This section discusses practical application of superelevation criteria, with particular attention to:

- The rates of superelevation to be used for various combinations of design speed and curve radius,
- The manner of transition of slope between normal tangent sections and superelevated sections on curves, and
- Special criteria for superelevation of shoulders and auxiliary lanes.

5.3.1 RATES OF SUPERELEVATION

The Green Book, pages 165 to 174, sets forth the basic design criteria based on design speeds for the normal design superelevation rates of e_{max} = 4 and 6 percent as well as other values ranging up to 12 percent. The criteria shown includes the minimum radius of curvature, crown treatment and superelevation runoff lengths (L), all of which are related to the number of lanes to be rotated. The minimum rate of cross slope for a traveled lane is determined by drainage requirements.

Figure 5-9*
Minimum Superelevation Runoff and Tangent Runout Lengths (US Customary)

Minimum runoff and runout length (ft)				
Design speed (mph)	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
25	34	69	103	34
30	36	73	109	36
35	39	77	116	39
40	41	83	124	41
45	44	89	133	44
50	48	96	144	48
55	51	102	153	51
60	53	107	160	53
65	56	112	167	56
70	60	120	180	60
Two lanes rotated				
25	51	103	154	51
30	55	109	164	55
35	58	116	174	58
40	62	124	186	62
45	67	133	200	67
50	72	144	216	72
55	77	153	230	77
60	80	160	240	80
65	84	167	251	84
70	90	180	270	90

Figure 5-10*
Minimum Superelevation Runoff and Tangent Runout Lengths [Metric]

Minimum runoff and runout length [m]				
Design speed [km/h]	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
20	9	18	27	9
30	10	19	29	10
40	10	21	31	10
50	11	22	33	11
60	12	24	36	12
70	13	26	39	13
80	14	29	43	14
90	15	31	46	15
100	16	33	49	16
110	18	35	53	18
Two lanes rotated				
20	14	27	41	14
30	14	29	43	14
40	15	31	46	15
50	17	33	50	17
60	18	36	54	18
70	20	39	59	20
80	22	43	65	22
90	23	46	69	23
100	25	49	74	25
110	26	53	79	26

*Note: Figures 5-9 and 5-10 are based on 12-ft [3.6 m] lanes and 2.0% normal cross slope

Chapter Seven

Intersections

The intersection of two or more roads presents an opportunity for conflict among vehicles. For freeways, the potential for conflict is significantly reduced through the use of interchanges. But interchanges usually are not feasible for the vast majority of intersections on arterials and collectors. This chapter is a general discussion of intersection design with those elements of particular application to this state. The details on intersection design are found in Chapter 9 of AASHTO's Green Book.

The principal objectives in the design of at-grade intersections are:

- To minimize the potential for and severity of conflicts,
- To provide adequate capacity, and
- To assure the convenience and ease of drivers in making the necessary maneuvers.

In the design of intersections there are three elements to consider:

(1) Perception-reaction distance,

(2) Maneuver distance, and

(3) Queue-storage distance.

The distance traveled during the perception-reaction time varies with vehicle speed, driver alertness, and driver familiarity with the location. Where left-turn lanes are introduced, this distance includes that to brake and change lanes. Where no turn lanes are provided, the distance needed is for the driver to brake comfortably. The storage length should be sufficient to accommodate the longest queue most commonly experienced.

An important consideration in the design of intersections is the treatment of right-turn lanes. Right turns can be free flowing, yield or stop controlled. In order to operate properly, free flowing right-turn lanes need to have an adequate acceleration distance free of access points for drivers to safely merge into the through traffic. Some drivers, particularly older drivers, are apprehensive when entering another leg of an intersection and may stop or slow down in the merge lane until the lane is clear of traffic. However, when properly designed, the majority of drivers will use the lane as proposed.

7.1 GENERAL CONSIDERATIONS

This section describes the various types of intersections and the general criteria that must be considered during design. Project intersection design configurations are developed during the project development phase based upon capacity analysis, accident studies, pedestrian use, bicycle use and transit options. In addition, design-hour turning movements, size and operating characteristics of the predominant vehicles, types of movements that must be provided, vehicle speeds, and existing and proposed adjacent land-use are considered.

Intersection designs range from a simple residential driveway to a complicated convergence of several high-volume multi-lane roadways. They all have the same fundamental design elements: (1) level of service, (2) alignment, (3) profile, (4) roadway cross section(s), and (5) sight distance. However, other

signals, or where signals may be warranted in the near future, it may be desirable to warp the crowns of both roads to avoid a pronounced hump or dip in the grade line of the minor highway. Intersections in superelevation areas are difficult to provide smooth grades or adequate drainage for and should be avoided.

7.1.5 FRONTAGE ROAD INTERSECTIONS

When a divided arterial highway is flanked by a frontage road, the problems of design and traffic control are more complex. Four separate intersections actually exist at each cross street.

The problem becomes more severe when the distance between the arterial and frontage road is relatively small. Generally, the outer separation between the two roadways should be 150 ft [50 m] or more.

Quite often, right-of-way considerations make it impractical to provide the full desired outer separation width. The alternative is to accept a narrow outer separation between cross roads and design a bulb-shaped separation in the immediate vicinity of each cross road.

7.1.6 DISTANCE BETWEEN INTERSECTIONS

Criteria for location, frequency and layout of private entrances and driveways are documented in DelDOT's *Standards and Regulations for Access to State Highways*. Illustrative sketches are shown for typical entrance and driveway designs for various conditions. For other types of public intersections, there are no fixed criteria as to frequency or distance between intersections. However, intersection spacing should provide sufficient distance to allow the proper development of all necessary turning lanes, bypass lanes, and, if signalized, proper signal coordination. Ideally this distance should be at least 350 ft [110 m] or more. Where intersections are closely spaced, several considerations should be kept in mind. It may be necessary to impose turn restrictions at some locations, prohibit pedestrian crossings, or provide frontage roads for access to intersecting roads. Where crossroads are widely spaced each at-grade intersection must necessarily accommodate all cross, turning and pedestrian movements.

7.2 TURNING MOVEMENTS

All intersections involve some degree of vehicular turning movements. There are various factors that influence the geometric design of turning lanes. The design controls for turning roadways are the traffic volume and types of vehicles making the turning movement. The roadway of primary concern is that used by right-turning traffic but may also be used for other roadways within the intersection. Figure 7-1 shows the terminology used when designing turning movements. The outer trace of the front bumper overhang and the path of the inner rear wheel establish the boundaries of the turning paths of a design vehicle.

The three typical types of designs for right-turning roadways in intersections are:

(1) A minimum edge-of-traveled-way design (Green Book, pages 583 to 621),

(2) A design with a corner triangular island (Green Book, pages 634 to 639), and

(3) A free-flow design using simple radius or compound radii (Green Book, pages 639 to 649). The turning radii and pavement cross slopes for free-flow right turns are functions of design speed and type of vehicle.

general classes of vehicles: (1) passenger cars, (2) buses, (3) trucks, and (4) recreational vehicles. The passenger car class includes passenger cars of all size, sport/utility vehicles, minivans, vans and pickup trucks. The bus class includes inter-city buses (motor coaches), city transit, school, and articulated buses. The truck class includes single-unit trucks, truck tractor-semitrailer combinations, and truck tractors with semitrailers in combination with full trailers. The recreational vehicle class includes motor homes, passenger cars with campers, cars with boat trailers, motor homes with boat trailers, and motor homes pulling cars. In addition, the bicycle should also be considered a design vehicle where applicable.

The dimensions for the design vehicles within these classes are listed in the Green Book. In the design process, the largest design vehicle likely to use that facility and its turning roadways with considerable frequency, or a design vehicle with special characteristics, is used to determine the design of such critical features as radii at intersections and radii of turning roadways.

A general guide to selecting a design vehicle is as follows:

- **P design vehicle** would be used for residential driveways, roadways with restricted truck use, local road intersections with a major roadway where use is infrequent, and low volume-minor road intersections. In most cases the selection of the SU design vehicle is preferred in all of these cases. The radii and widths permitted for the P design vehicle are very awkward for a single-unit delivery or service truck to safely maneuver.
- **SU design vehicle** provides the most economical minimum edge-of-traveled way design for rural roadways and other light truck use intersections. However, current and projected truck use on these roadways needs to be considered before a final design vehicle selection, particularly if any channelization is proposed.
- **Semitrailer combinations design vehicles** should be used where truck combinations will turn repeatedly, particularly heavily industrialized or commercial areas. Providing for these vehicles increases the paved areas, radii and other design parameters. Even in rural areas the local economy may be based on frequent semitrailer usage. Project development and scoping should identify these areas.

A project may have several design vehicles depending upon the predominant type of vehicle using the turning roadways being designed. A residential driveway would only need to consider a passenger car with an occasional single unit delivery truck. Industrial entrances would consider the predominant semi-trailer unit as the design vehicle. Other turning roadways and intersections would have to be similarly analyzed and an appropriate design vehicle selected. The purpose of this analysis is to assure the physical features are placed in positions that allow for the movement without making unsafe maneuvers, particularly within the through travel lanes, or destroying roadway features (curbs, signs, light poles, etc.).

Figure 7-2 shows selected minimum radii for the more commonly used design vehicles. Figure 7-3 (also see the Green Book, pages 216 to 224) shows the minimum design radii at the inner edge of the traveled way for roadway curves within an intersection based on a superelevation rate of 8.0 percent and free flow. For design speeds above 45 mph [70 km/h], the values are based on open road conditions. (See the Green Book, page 147.) At intersections controlled by stop signs, lower rates of superelevation are usually more appropriate. See the Green Book, pages 150 to 151, for urban streets.

Figure 7-2
Minimum Turning Radii for Selected Design Vehicles

Design Vehicle Type	Symbol	US Customary		Metric	
		Minimum Design Radius (ft)	Minimum Inside Radius (ft)	Minimum Design Radius [m]	Minimum Inside Radius [m]
Passenger Car	P	24	14.4	7.3	4.4
Single Unit Truck	SU	42	28.3	12.8	8.6
Intercity Bus	BUS-40 [BUS-12]	45	27.6	13.7	8.4
City Transit Bus	City-Bus	42	24.5	12.8	7.5
Conventional School Bus (65 pass.)	S-BUS36 [S-BUS11]	38.9	23.8	11.9	7.3
Large School Bus (84 pass.)	S-BUS40 [S-BUS12]	39.4	25.4	12.0	7.7
Intermediate Semi-trailer	WB-40 [WB-12]	40	19.3	12.2	5.9
Intermediate Semi-trailer	WB-50 [WB-15]	45	17.0	13.7	5.2

Figure 7-3
Minimum Radii at Inner Edge of Traveled Way for Intersection Curves–Free Flow

US Customary	
Design Speed (turning) (mph)	Minimum Radius (ft)
10	25
15	50
20	90
25	150
30	230
35	310
40	430
45	540
Metric	
Design Speed (turning) [km/h]	Minimum Radius [m]
10	7
20	10
30	25
40	50
50	80
60	115
70	160

7.2.2 EDGE-OF-TRAVELED-WAY DESIGNS

In the design of the edge of pavement for the minimum path of a given design vehicle, it is assumed that the vehicle is properly positioned within the traffic lane at the beginning and end of the turn. This position is 2 ft [0.6 m] from the edge of 12 ft [3.6 m] wide pavements on the tangents approaching and leaving the intersection curve.

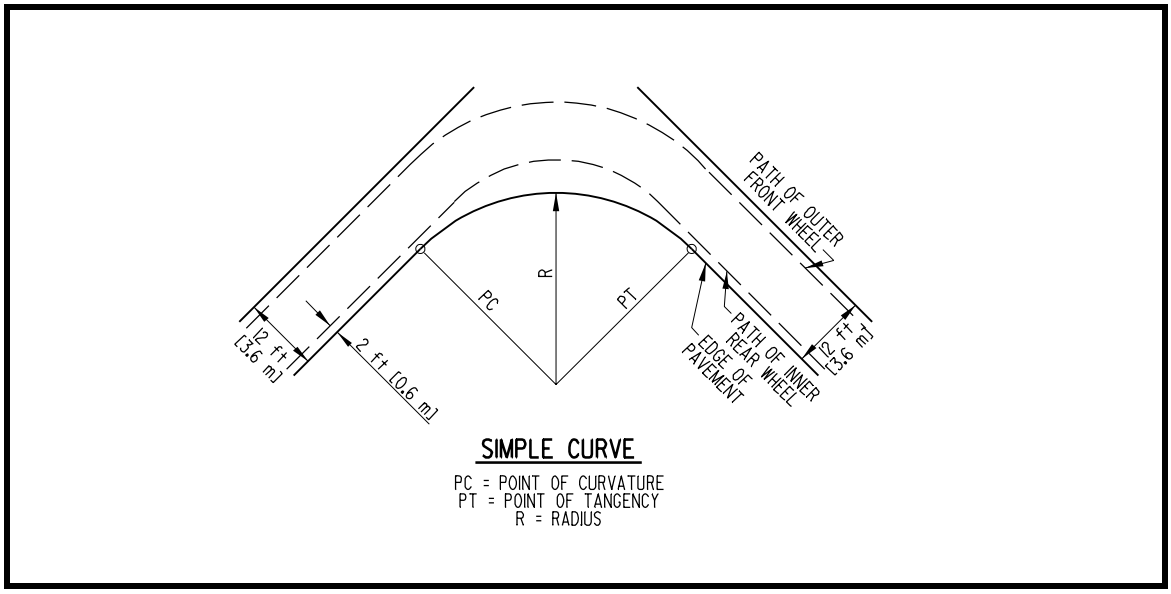
Three types of curves commonly are used for the design of pavement edges at intersections:

- simple curve,
- 3-centered symmetric compound curve, and
- 3-centered asymmetric compound curve.

Use of the simple curve usually is limited to residential driveways and low traffic volume intersections where there is little truck traffic. The 3-centered curve should be used

for edge-of-traveled-way design at all major intersections. Figures 7-4 and 7-5 illustrate the three types of edge-of-traveled-way designs. The Green Book, Exhibits 9-19 and 9-20 on pages 584 through 591 tabulate values for application of simple and three-centered compound curve applications for various angles of turning roadways. The angle of turn that is the next highest to the angle of turn of the intersection being designed should be selected.

Figure 7-4
Intersection Edge-of-Traveled-Way Design Layout
Using Simple Curves



7.2.3 PAVEMENT WIDTHS FOR TURNING ROADWAYS

The pavement and roadway widths of turning roadways at intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated, and may be designed for one-way or two-way operation, depending on the geometry of the intersection. Widths determined for turning roadways may also apply on through roadways within an intersection, such as channelizing islands.

Pavement widths for turning roadways are classified for the following types of operations:

Case I – one-lane, one-way operation with no provision for passing a stalled vehicle;

Case II – one-lane, one-way operation with provision for passing a stalled vehicle; and

Case III – two-lane operation, either one-way or two-way.

Case I widths are normally used for minor turning movements and for moderate turning volumes where the connecting roadway is relatively short. The chance of vehicle breakdown is remote under these conditions, but one of the edges of pavement should be available for passing a stalled vehicle, i.e. mountable curb and clear of obstructions.

Case II widths are determined to allow operation at low speed and with restricted clearance past a stalled vehicle. These widths are applicable to all turning movements of moderate to heavy volumes that do not exceed the capacity of a single-lane connection. In the event of a breakdown, traffic flow can be maintained at somewhat reduced speed. Many ramps and connections at channelized intersections are in this category.

Case III widths apply where operation is two-way, or one-way with two lanes needed to handle the traffic volume. In the latter case, downstream lanes must be able to accommodate the two-lane volume. In each category the required pavement width depends jointly on the size of the design vehicle and the curvature of the turning roadway. Selection of the design vehicle is based on the size and frequency of vehicle types. The pavement width increases with both the size of the design vehicle and the sharpness of curvature. See Figures 7-6 and 7-7 for the recommended design widths of pavements for turning roadways at intersections for three types of operations and for three conditions of traffic mixes.

The designer should refer to the Green Book, pages 199 to 229, for further details on designing turning roadways within intersections.

7.3 CHANNELIZATION

Channelization is the separation or regulation of conflicting-traffic movements into definite paths of travel by traffic islands or pavement markings to facilitate the safe and orderly movement of both vehicles and pedestrians. Proper channelization increases capacity, improves safety, provides maximum convenience, and instills driver confidence. Improper channelization has the opposite effect and may be worse than none at all. Over channelization should be avoided because it could create confusion and deteriorate operations.

7.3.1 PURPOSE

Channelization of at-grade intersections is generally warranted for one or more of the following factors:

- The paths of vehicles are confined by channelization so that not more than two paths cross at any one point.
- The angle and location at which vehicles merge, diverge or cross are controlled.
- The paved area is reduced, thereby narrowing the area of conflict between vehicles and decreasing the tendency of drivers to wander.
- Clearer indications are provided for the proper path in which movements are to be made.
- The predominant movements are given priority.
- Areas provide for pedestrian refuge.
- Separate storage lanes permit turning vehicles to wait clear of through-traffic lanes.
- Space is provided for traffic control devices so they can be more readily perceived.
- Prohibited turns are controlled.

7.3.2 DESIGN PRINCIPLES

Design of a channelized intersection usually involves the following significant controls—the type of design vehicle, the cross sections on the crossroads, the projected traffic volumes in relation to capacity, the number of pedestrians, the speed of vehicles, and the type and location of traffic control devices. Furthermore, physical controls such as right-of-way and terrain have an effect on the extent of channelization that is economically feasible.

Figure 7-6
Design Widths for Turning Roadways (US Customary)

Radius on Inner Edge R (ft)	Pavement Width (ft)								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	14	17	18	20	24	26	26
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 1 ft Add 2 ft			None Add 1 ft			Add 1 ft Add 2 ft		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 12 ft where shoulder is 4 ft or wider			Deduct shoulder width; minimum width as under Case I			Deduct 2 ft where shoulder is 4 ft or wider		

Note:

Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination vehicles.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Figure 7-7
Design Widths for Turning Roadways [Metric]

Radius on Inner Edge R [m]	Pavement width [m]								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 0.3 m Add 0.6 m			None Add 0.3 m			Add 0.3 m Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width; minimum width as under Case I			Deduct 0.6 m where shoulder is 1.2 m or wider		

Note:

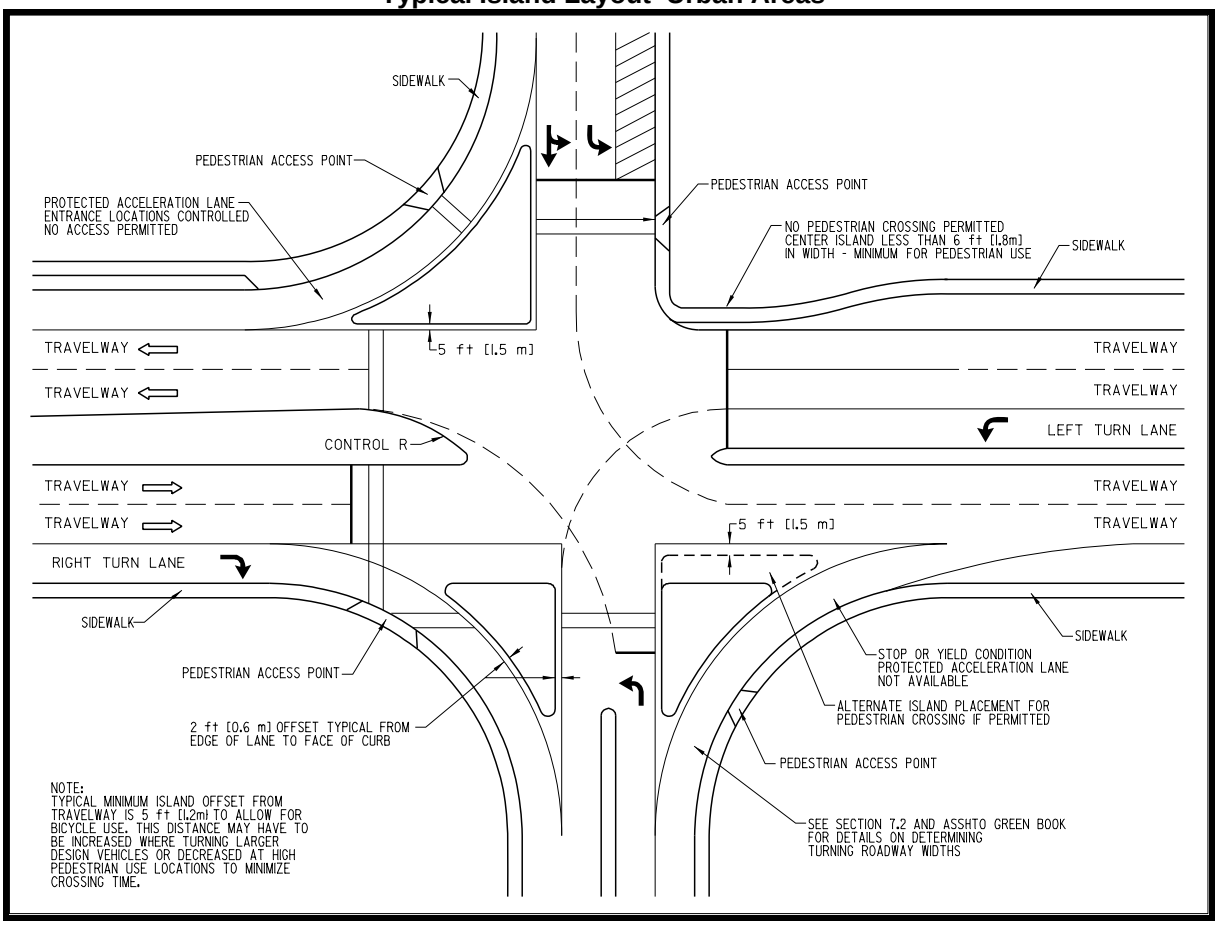
Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination trucks.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Figure 7-9
Typical Island Layout–Urban Areas



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7.4 SIGHT DISTANCE

The operator of a vehicle approaching an at-grade intersection should have an unobstructed view of the whole intersection and of a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions which is termed "approach sight distance". (See Figure 7-10.) The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and the resultant distances traversed during perception, reaction time, and braking. In addition to approach sight distance, sight distance is also provided to allow stopped vehicles sufficient view of the intersecting roadway to decide when to enter the intersecting roadway or to cross it, which is "departure sight distance." Both sight distances must be checked on all intersection designs based on the procedures set forth in the Green Book, pages 650 to 679.

7.4.1 MINIMUM SIGHT DISTANCE TRIANGLE

Sight triangles are areas of unobstructed sight along both roads at an intersection and across their included corner for a distance sufficient to allow the operators of vehicles approaching simultaneously to see each other in time to prevent collision at the intersection.

The length of the legs may vary based upon traffic volumes, design speeds, operating speeds and type of intersection traffic control.

Normally in less densely populated areas the minimum sight distance of any leg would be that required to meet the design stopping sight distance for the major road. At high volume intersections the need for large sight triangles is diminished and is a function of the types of traffic control devices and the presence or absence of other vehicles. The Green Book, pages 654 to 679, provides details for determining sight triangles for several different conditions that may occur at intersections, primarily based on the type of traffic control.

In each case, assumptions are made about the physical layout and the actions of vehicle operators on both intersecting roads. For each case, the space-time-velocity relations indicate the minimum sight triangle that is required to be free of obstructions. Any object within the sight triangle high enough above the elevation of the adjacent obstruction should be removed or lowered. Such objects include cut slopes, trees, hedges, bushes, or tall crops. There should be no parking within the sight triangle.

The minimum stopping sight distance in the Green Book for open highway conditions are also valid for turning roadway intersections of the same design speed. Figure 7-11 includes stopping sight distance for lower turning speeds than commonly used under open roadway conditions. These values should be available at all points along a turning roadway, and should be increased wherever practical. They apply to both vertical and horizontal alignment.

Figure 7-10
Sight Distance Triangles—Elements for At-Grade Intersections

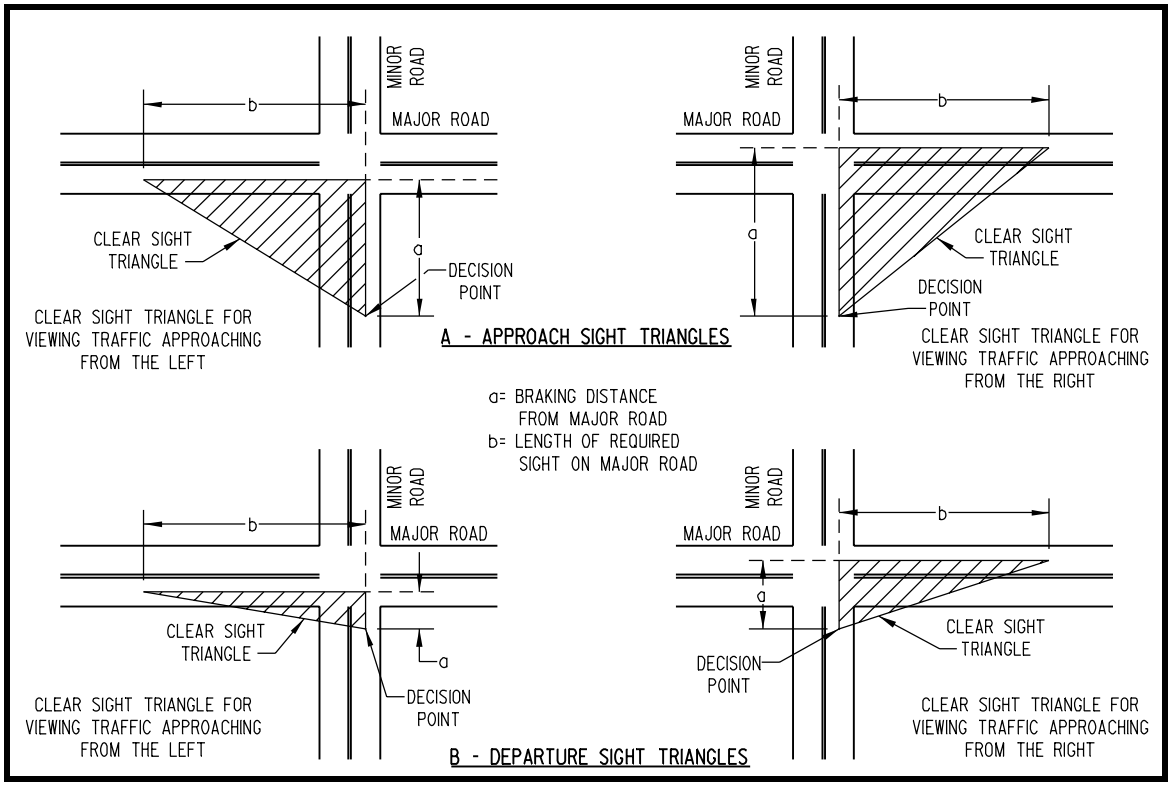


Figure 7-11
Minimum Stopping Sight Distance for Turning Roadways at Intersections

US Customary	
Design (Turning)Speed (mph)	**Stopping Sight Distance (ft)**
10	50
15	80
20	115
25	155
30	200
35	250
40	305
45	360
Metric	
Design (Turning)Speed [km/h]	**Stopping Sight Distance [m]**
15	15
20	20
30	35
40	50
50	65
60	85
70	105

7.4.2 INTERSECTION MANEUVERS

When traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position. There are three basic maneuvers that occur at the average intersection. These maneuvers are:

1. To travel across the intersecting roadway by clearing traffic from both the left and the right of the crossing vehicle,
2. To turn left into the crossing roadway by first clearing traffic on the left and then entering the traffic stream with vehicles from the right, and
3. To turn right into the intersecting roadway by entering the traffic stream with vehicles from the left.

The stop condition criterion is applicable to two-lane, two-directional roadways through multi-lane divided highways. Where the principal roadway is either undivided or divided with a narrow median (the median is too narrow to store the design vehicle), the departure maneuvers are treated as a single operation. Where the major roadway is divided and has a median wide enough to safely store the design vehicle, the departure maneuvers are considered as two operations. The first operation concerns the traffic approaching from the left for all three maneuvers; that is, crossing the entire roadway, crossing part of the roadway and turning left into the crossroad or turning right into the crossroad. The second phase concerns traffic from the right for the first two operations; i.e., continuing to cross the major roadway or turning left and merging with traffic from the right. The Green Book, pages 650 to 679, provides details on analyzing the departure sight triangles for these maneuvers.

7.5 AUXILIARY TURNING LANES

Auxiliary turning lanes may be introduced at intersections under a variety of conditions including rural or urban locations and free flowing, signalized or stop controlled traffic designs. Using auxiliary lanes to handle turning movements at high volume intersections can reduce congestion, improve safety and provide better traffic control. Auxiliary lanes are also used on four-lane divided roadways and high volume two lane roadways under open road conditions. They improve safety and traffic flow when introducing median openings, intersections at minor crossroads or U-turn locations.

Straight tapers at least 100 ft [30 m] long should be used for deceleration lanes.

The designer has at least four methods available (listed from the preferred to the least acceptable) for determining deceleration lane lengths: (1) design the intersection in accordance with the HCM based on detailed existing and projected traffic data, (2) provide the desirable lengths as discussed in the Green Book (3) design left turn lanes based on the methodology shown on Figure 7-17 or (4) provide the minimum lengths as discussed in this section and shown in Figure 7-18. The use of each of these approaches is also dependent upon the roadway classification, type of facility, the location of the intersection within the facility, project scope and funding.

The Green Book proposes that for arterials with a selected design speed of 30, 40, 45, 50 and 55 mph [50, 60, 70, 80 and 90 km/h], the desirable deceleration lengths of the auxiliary lanes, where practical, are 170, 275, 340, 410 and 485 ft [50, 70, 95, 120 and 150 m], respectively. These lengths allow the driver to comfortably decelerate to a full stop from the full design speed with grades of 3 percent or less. These values do not include taper or required storage length.

The Green Book further discusses the fact that on many urban facilities providing the full length is not practical, physically possible or economically reasonable to provide the suggested desirable lengths needed for decelerating from design speed or operating speed to a full stop condition. On urban facilities in densely developed areas, the need for storage length may override the desirable deceleration length. The Green Book concludes that on urban and collectors the designer may assume that a portion of the deceleration speed is accomplished in the through lane and/or on the taper before entering the full width auxiliary lane. The Green Book further states that: “Therefore, the lengths given above should be accepted as a desirable goal and should be provided where practical.”

Figure 7-17 illustrates a design methodology for determining a reasonable minimum length for an auxiliary turning lane under open highway conditions when complete traffic data is not available. In this figure, the typical average running speed on the main facility is used and some deceleration for the left-turn movement is assumed to occur prior to entering the turning lane. Based on assumed vehicle approach speeds, the desirable deceleration lengths are shown in Figure 7-18. See the Green Book, page 851, for lengths applicable to other exit curve design speeds.The lengths shown do not include any taper lengths or required storage lengths. These lengths are for open highway conditions. It should be recognized that operating speeds, traffic volumes, traffic mix, type of facility, project intent, roadside development, and intersection frequency and spacing all influence a designer’s ability to provide the lengths shown in Figure 7-18.

To reiterate, in the use of Figure 7-17, DelDOT has adopted the recognition in the Green Book, page 714, that a degree of deceleration can safely take place in the through lane depending upon posted speed, type of facility and traffic volumes. The suggested design approach for arterial and other high volume roadways assumes a reduction of 10 mph [15 km/h] below posted speed occurs in the through lane. For collectors and other medium volume roadways, an assumed reduction of 15 mph [20 km/h] is practical. For low volume collectors and local streets, a reduction of 20 mph [30 km/h] below the posted speed may be assumed in the design of auxiliary lanes. The deceleration lengths shown in the figures are applicable to both left and right-turn lanes.

Figure 7-17 is a general guide for use when the designer does not have existing or projected traffic volumes or turning counts. When this data is available the length and design of auxiliary lanes should be analyzed in accordance with the HCM.

Figure 7-12
Guide for Need for Left-Turn Lanes on Two Lane Highways

Opposing volume (vph)	Advancing volume (vph)			
	5% Left turns	10% Left turns	20% Left turns	30% Left turns
40-mph [60 km/h] operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph [80 km/h] operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph [100 km/h] operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

For signalized intersections when there are no current or projected traffic counts or studies available to indicate the needed storage length, then the following method suggested by AASHTO can be applied.

Storage length is based on the number of vehicles likely to arrive in an average cycle time period within the peak hour in accordance with the following formula:

$$S.L. = (N \ / \ C) \times V.L. \times 1.5$$

Where:

S.L. = Storage Length,
V.L. = Vehicle length—use 20 ft [6.0 m] for passenger cars,
N = Number of left-turn vehicles in peak hour, and
C = Number of cycles per hour.

At unsignalized intersections, the average cycle time is assumed to be 2 minutes, so:

$$C = \frac{60 \text{ minutes per hour}}{2 \text{ minutes}} = 30$$

At signalized intersections, "C" is computed using the actual cycle time, so:

$$C = \frac{60 \text{ minutes per hour}}{\text{Actual cycle time in minutes}}$$

Where there is a demonstrated need due to turning volumes versus available gaps in the opposing traffic, the recommended minimum storage length for median auxiliary lanes is 50 ft [15 m]; for separate left turn facilities where no median exists, the minimum recommended storage length is 100 ft [30 m]. These lengths will allow for storing one P and one SU design vehicle or an occasional WB-50 [WB-15]. The greater length where there is no median provides allowance for a decrease in available turning paths.

Acceleration lanes for right-turning vehicles entering a traveled way may need to be considered when turning volumes exceed 100 vph. However, as discussed earlier in this chapter acceleration lanes are not always desirable where entering drivers can wait for an opportunity to merge without disrupting through traffic, such as at a signalized intersection. The use of acceleration lanes should generally be restricted to rural, free-flow, or

controlled access situations. The length of the acceleration lane is a function of the pooled highway speed and speed of the turning vehicle as shown in Figure 7-19. The Green Book, page 847, gives additional lengths for other selected entrance curve design speeds.

7.5.1 MINIMUM TURN LANE LENGTHS

A project's intent or funding may not allow for providing the desirable left-turn lengths as suggested in the Green Book, or be designed in accordance with the HCM or the method shown on Figure 7-17. In this situation and for uniform application, the suggested minimum lengths for left turn lanes are as described in this section. The parameters are:

- Suggested minimum lengths apply to divided roadways at unsignalized locations.
- No previously identified history of problems with accidents, operation or safety.
- No established warrants based on traffic volume.
- Locations with observed or anticipated high truck use need more storage length.

The lengths are determined by general class of roadway. It should be recognized that each location is unique and has to be analyzed based on its characteristics, including traffic control devices and the selected length may be different than those that follow.

Divided rural arterials and collectors with moderate to heavy through traffic with a posted speed of 50 mph [80 km/h] or greater use a taper length of 100 ft [30 m], deceleration length of 250 ft [75 m], and storage length 100 ft [30 m].

Divided rural arterials and collectors with light through traffic and a posted speed of 50 mph [80 km/h] or greater use a taper length of 100 ft [30 m], deceleration length of 150 ft [45 m], and storage length of 50 ft [15 m].

Divided urban arterials and collectors with heavy to moderate through traffic and a posted speed of 50 mph [80 km/h] or less use a taper length of 100 ft [30 m], deceleration length of 200 ft [60 m], and storage length of 100 ft [30 m].

Divided urban arterials and collectors with light through traffic and a posted speed of 50 mph [80 km/h] or less use a taper length of 100 ft [30 m], deceleration length of 200 ft [60 m], and storage length of 50 ft [15 m].

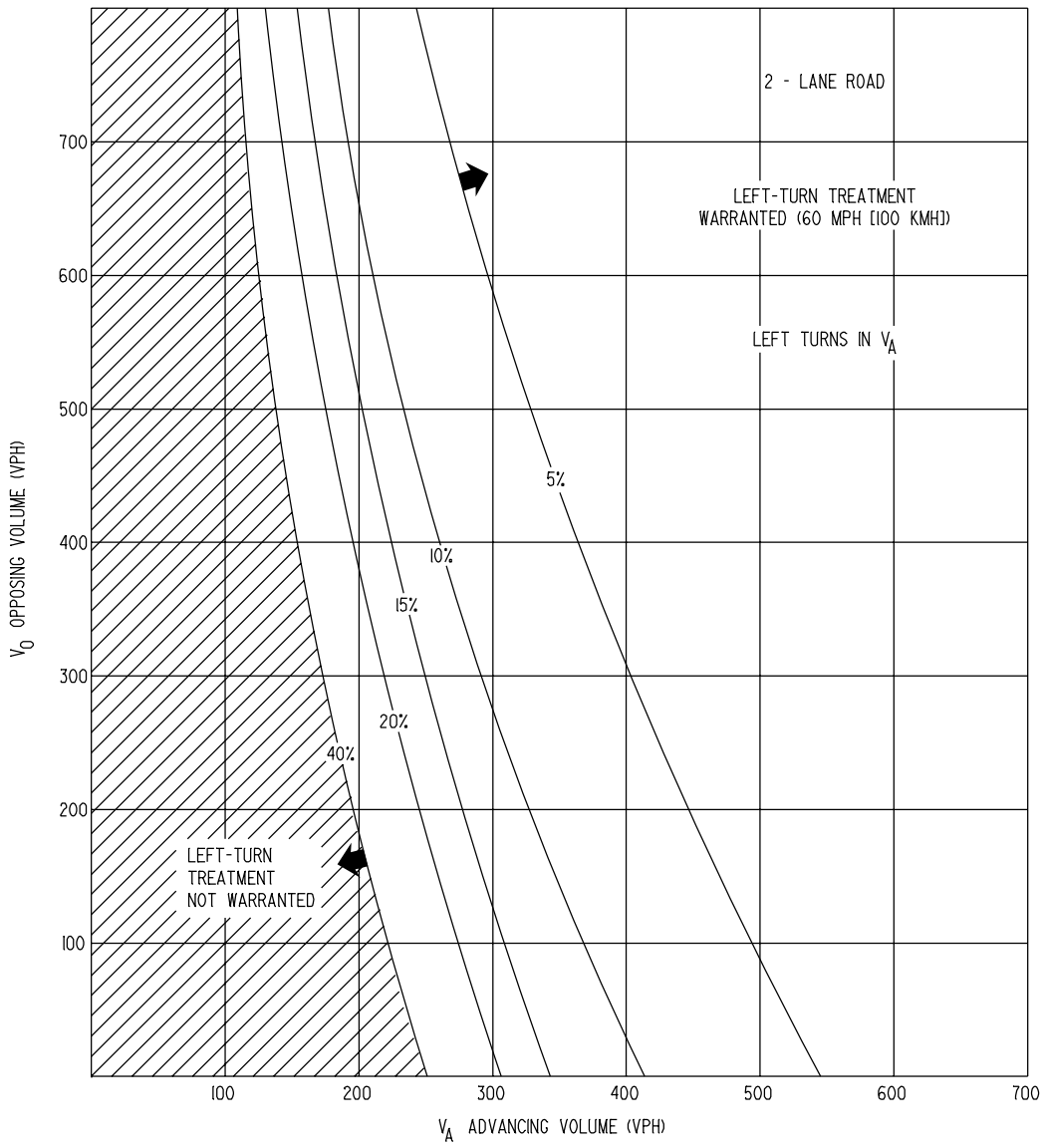
The designer should refer to the Green Book and the HCM for further discussion concerning these guidelines.

7.6 MEDIAN OPENINGS

The following is a general discussion of median opening design. The Green Book, pages 689 to 728, presents a comprehensive discussion on the concepts and design of median openings.

Median opening designs range from designing for simple U-turn movements to the more complex unsignalized and signalized rural and urban intersections that may include traffic from minor crossroads and streets or major roadways and commercial entrances. The design of median openings and median end treatments is based on traffic volumes, operating speeds, predominant types of turning vehicles and median width. Crossing and turning traffic must operate in conjunction with the through traffic on a divided highway. This requirement makes it necessary to know the volume and composition of all movements occurring simultaneously during the design hour. The discussion in this section is primarily directed to rural, unsignalized, divided roadways.

Figure 7-16
Graphical Guide for Left-Turn Lane for
60 mph [100 km/h] Operating Speed



the trailing edge for 20 ft [6.0 m].The bullet nose design can be designed to conform to the traffic movements permitted in the intersection. A wide median normally would have a portion of the nose flattened to be parallel to the median opening centerline and, depending upon the channelization design, a semicircular design may be more appropriate.

Figure 7-20
Preferred Median End Shapes
Based on Median Width

Controlling Median Width	Median End Shape
4 ft [1.2 m] or less	*Semicircular
4 to 66 ft [1.2 to 20 m]	Bullet Nose or Modified Bullet Nose
Over 66 ft [20 m]	Treated as separate intersection

*At locations with left turning cross road traffic, use a controlling radius of at least 40 ft [12 m].

7.6.3 LENGTHS OF MEDIAN OPENINGS

For any intersection on a divided highway, the length of the median opening should be as great as the width of the crossroad roadway pavement plus shoulders. The width and type of crossroad combined with the median width and selected control radius affect the median opening. The design should minimize any unsafe tracking encroachment into oncoming traffic from crossroads. AASHTO recommends that in no case should the opening be less than 40 ft [12 m] for a 90-degree intersection or less than the width of the crossroad pavement plus shoulders or plus 8 ft [2.4 m] for a crossroad without shoulders. Where the crossroad is a divided highway, the length of the opening should be at least equal to the width of the crossroad roadways, median and shoulders or 8 ft [2.4 m] if there are no shoulders.

Median openings are a function of median width and the selected control radius. Use of a 40 ft [12 m] minimum opening without regard to these two items should only be considered for minor, rural, unsignalized crossroads. Median openings of 50 to 64 ft [15 to 20 m] are more typical. The 40 ft [12 m] minimum length of opening does not apply to openings for U-turns where, depending upon the predominant vehicle, larger openings may be needed to ensure the vehicle can turn into the desired lane. As median widths become greater than 50 ft [15 m] the increased pavement area may create confusion as to proper vehicle paths and movements. These wider openings may need additional traffic control devices. ASSHTO recommends avoiding using median openings greater than 80 ft [25 m].

7.6.4 DESIRABLE MEDIAN OPENING DESIGNS FOR LEFT TURNS

Median openings that enable vehicles to turn on minimum paths, and at very low speeds, are adequate for intersections where traffic for the most part proceeds straight through the intersection. Where through-traffic volumes and speeds are high and left-turning movements are important, undue interference with through traffic should be avoided by providing median openings that permit turns without encroachment on adjacent lanes. This arrangement would enable turns to be made at speeds above those for the minimum vehicle paths and provide space for vehicle protection while turning or stopping.

For median openings having control radii greater than the minimum for the selected design vehicle, see the Green Book, pages 690 to 696. The three radii R, R1 and R2 control bullet-nose end designs. Figure 7-21 shows the layout. Radius R is the control radius for the sharpest portion of the turn. R_1 defines the turnoff curve at the median edge. R_2 is the radius of the tip.

When a sufficiently large R_1 is used, an acceptable turning speed for vehicles leaving the major road is assured, and a sizable area inside the inner edge of the through-traffic lane between points 1 and 2 on Figure 7-21 may be

- Simple, clear signing.

10.11.5 BUSES

As previously discussed, where buses load and unload within the parking lot, traffic flow should be such that buses and automobiles do not conflict. Buses require adequate room for decelerating, proper turning radii, maneuvering into and out of an adequate loading area, and returning to the mainline traffic flow. Refer to Chapter 7 of this manual and the Green Book for design criteria such as required turning radii for bus operation.

10.11.6 KISS-AND-RIDE FACILITIES

A kiss-and-ride facility is located so that transit or commuter passengers can easily and safely access the terminal or loading zone with minimum conflicts with other vehicles; see Figure 10-19. To accomplish this, circulation in the kiss-and-ride facility should be one-way and flank the terminal or loading area. Parking should be at 45 degrees to allow for pull through and face the terminal or loading zone. To operate properly it is usually necessary to enforce kiss-and-ride restrictions.

10.11.7 PEDESTRIANS

Two pedestrian movements must be provided for park-and-ride lots that serve bus routes: a direct and safe approach from adjacent streets to the bus stop and pedestrian access from the parking area. Pedestrian circulation in parking lots is provided by aisles and crosswalks or, in larger lots, by walkways. The pedestrian path from any parking stall to the bus stop should be as direct as possible.

10.11.8 BICYCLES AND MOTORCYCLES

It is important to provide adequate bicycle storage racks at park-and-ride lots where large concentrations of bicycle traffic are expected. Similarly, a special parking area for motorcycles will improve utilization of space. Motorcycle storage should be on Portland cement concrete to prevent stands from sinking into hot asphalt pavement. Provisions for locking both bicycles and motorcycles to prevent theft are needed. This includes bicycle racks and lockers.

10.11.9 DISABLED

At lots for transfer to buses, the design should consider provisions for safe and convenient access for the elderly and disabled. Design requirements and provisions for disabled parking shall be in conformance with the *Americans with Disabilities Act of 1990* and the State of Delaware *Architectural Accessibility Standards.*

Facilities for the disabled should also be designed in accordance with the following:

- Disabled persons should reach the bus loading zone without crossing any access roads;
- Loading areas must meet ADA space requirements;
- Disabled persons must never be forced to travel behind parked cars; and
- Suitable ramps must be provided.

10.11.10 PARKING DIMENSIONS AND LOT LAYOUT

Parking areas and roadway layout for park-and-rides can be designed in much the same manner as other parking facilities. Facilities that interact with transit, where DTC does not specify a bus size, should use a 40 foot [12 m] transit vehicle as the design vehicle. Standard dimensions for car parking stalls are shown in Figure 10-20.

For design purposes only two size stalls should be considered—standard and intermediate. If compact car parking is to be provided, it should be in a prime location or the driver will select more convenient

available stalls. Combining several different types of stalls also creates an undesirable and more complicated signing layout. If there is adequate room, limiting the design to accommodate the standard stall size, 9 by 18.5 ft [2.7 by 5.6 m] is preferred. The minimum bus-parking stall should be 13 by 32 ft [4 by 15.25 m].

Figure 10-18
Typical Car Parking Dimensions

Size	Stall Width ft [m]	Stall Length ft [m]	Aisle Width ft [m]
Standard	8.5-9.5 [2.6-2.9]	18-20 [5.5-6.0]	24-26 [7.5-8.0]
Inter-mediate	8.0-9.0 [2.4-2.7]	16-18 [4.9-5.5]	22-24 [6.7-7.3]
Compact	7.5-8.5 [2.3-2.6]	15-17 [4.6-5.2]	20-22 [6.0-6.7]

Substandard stall and aisle widths are false economy. Although they permit marking more stalls in a given area, vehicles tend to encroach upon adjacent stalls such that one or more spaces are unavailable for use. The end result is no gain in actual space usage.

Vehicles and other objects should be excluded from corners or parking spots where it is necessary to provide adequate intersection sight distances. Islands at the end of rows should be considered when laying out the lot. For pedestrian safety, the parking stalls and aisles should be parallel to the direction of the desired pedestrian flow. For efficient land area use, a row of parking on each side of the aisle is preferred.

Aisle width is a function of the parking angle and stall width. One-way aisles are generally used with angle parking and two-way circulation is generally used with 90-degree parking. Aisle lengths should not exceed 400 ft [120 m] to limit pedestrian walking distance. One-way aisles should favor counterclockwise circulation with head-in parking only. Due to lower vehicle undercarriage heights, a 6 in [150 mm] curb is recommended where head-in parking is being considered. Sidewalks should be a minimum of 5 ft [1.5 m] and loading areas should be 12 ft [3.6 m] wide. Pedestrian paths from parking spaces to loading areas should be as direct as possible. All sidewalks and curb areas are to be in conformance with ADA standards.

Figures 10-21 and 10-22 provide data for planning stall layouts for standard stall sizes of 9 by 18.5 ft [2.0 by 5.6 m]. Layouts for intermediate and compact stall sizes are available in the AASHTO Green Book.

The parking area should be sloped to provide positive drainage. Ponding water in a lot is undesirable for both vehicle and pedestrian movement, particularly where freezing may create icy spots. The recommended minimum grade is 1%, the desirable is 2%, and the maximum is 5%. The designer should provide adequate access and areas for snow removal and/or storage. The pavement selection needs to recognize that a variety of traffic loads, particularly when transit is expected, may be applied to the lot and the pavement type and strength designed accordingly.

Factors to be recognized and addressed in the design process include:

- Selecting design guidelines that will provide for the safety of the user.
- Identifying the need for access to the facility, as well as mobility along the facility.
- Preservation or enhancement of historic sites and districts.
- Protection or enhancement of existing environmental assets.
- Identifying the economic needs of and impacts to the affected community or area.
- Developing an understanding of the social context of the community and area within which the facility exists.

Ensuring that a project design will have a balance of these factors is the result of a continuous and meaningful public involvement process. Throughout the public involvement process, the designer must make sure the purpose for the project as established in the project initiation is fulfilled while understanding and addressing the needs of the community. By doing this the introduction of new or additional issues 2004 ing the final design phase that may result in delays and/or redesign will be minimized.

The basic design tools available to the designer are this manual and the American Association of State Highway Transportation Officials, (AASHTO) "*A Policy on Geometric Design of Highways and Streets*" commonly referred to as the "Green Book". In addition designers need to refer to other related publications and guidelines prepared by the Federal Highway Administration (FHWA), the Transportation Board (TRB), and other recognized experts in the transportation field. The principal publications are:

- AASHTO's *Roadside Design Guide*,
- AASHTO'S *Context Sensitive Design for Integrating Highway and Street Projects with the Community and the Environment*,
- Transportation Research Board's (TRB) *Highway Capacity Manual*,
- TRB's *Special Report 214 Designing Safer Roads*,
- FHWA's *Manual on Uniform Traffic Control Devices*,
- FHWA's *Flexibility in Highway Design*, and
- DelDOT's *Traffic Calming Manual.*

In addition, there are numerous other documents, particularly related to highway drainage and intersection design that must be referenced in designing a project.

Using these documents, the Road Design Manual was developed with emphasis on standards and practices that have proven to be successful in this state. The flexibility to design a project that will meet the expectations of the user, the community and Department exists in the Green Book and in the standards found in this manual. Published design standards have a measure of flexibility, usually stating a maximum and minimum value. Many of these values are empirically based using mathematical modeling techniques with assumed roadway surface conditions, driver reaction times and adverse weather conditions.

Designers need to recognize that there is a difference between the strict application of design standards found in the tables and charts versus providing consistency in design. The design should ensure there is consistency in application of the standards that allows the driver to react in a consistent and predictable manner when encountering similar roadway conditions. However, in responding to the many issues that arise on each project, there is a need for flexibility in

expenditures that could be spent more effectively by improving additional road sections.

- Context. Design features should be selected that are in balance with the social context of the community and surrounding area. This is accomplished by gathering and including information from the public throughout the design process. A context sensitive design advances the objectives of safety, mobility, enhancement of the natural environment, and preservation of community values. Projects that improve the livability of the community or quality of the natural environment are considered context sensitive.

2.2 LEVELS OF SERVICE

In general terms, the level of service of a highway facility may be influenced by many factors, including surface condition and ride-ability. From the standpoint of design controls, the level of service is principally related to the ease and convenience with which the highway facility can serve the expected volumes of traffic.

The Transportation Research Board's (TRB) *Highway Capacity Manual* presents a thorough discussion of the level of service concept. Six levels of service are established from level A (the highest) through level F (the lowest).

The general characteristics of the various levels of service are:

- Level of Service A – free-flowing traffic; users virtually unaffected by other traffic, able to select desired speeds and maneuver unrestricted.
- Level of Service B – reasonably free traffic flow; users able to select desired speeds, but with a slight decline in freedom to maneuver.
- Level of Service C – stable flow, but operation of individual users is significantly affected by traffic; ability to select speeds is reduced and maneuvering requires substantial vigilance by the users.
- Level of Service D – high density approaching unstable flow. Speeds and freedom to maneuver are severely restricted. Small increases in traffic flow will generally cause operational problems.
- Level of Service E – operating conditions at or near capacity with unstable flow. All speeds at a low and relatively uniform value. Freedom to maneuver is extremely difficult.
- Level of Service F – forced or breakdown flow. Traffic exceeds capacity causing queues with stop-and-go waves, and operations are extremely unstable.

The traffic flow rates that can be served at each level are termed "service flow rates." Once a level of service has been identified as applicable for design, the accompanying service volume logically becomes the design service flow rate, implying that if the traffic volume using the facility exceeds that amount, operating conditions will fall below the level of service for which the facility was de-signed. A guide for selecting design levels of service is shown in Figure 2-1.

More detailed guidelines for selecting appropriate levels of service are given in AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the "Green Book") and TRB's *Highway Capacity Manual* (HCM).

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Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page ~~388~~, Exhibit 5-5; 384
- Collector Roads and Streets (Rural) — page ~~429~~, Exhibit 6-5; 425
- Collector Roads and Streets (Urban) — page ~~437~~; 433
- Arterials (Rural) — page ~~452,~~ Exhibit 7-3; 448
- Divided Arterials (Rural) — page ~~459~~; 455
- Urban Arterials — page ~~476~~; 472
- Freeways — page ~~508~~. 504

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-5, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages ~~156 to 161~~ 167 to 174, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

Vertical curvature, horizontal curvature, roadside obstructions, or any combination of these elements can restrict sight distance. Procedures for checking available sight distances are described in the Green Book, pages 127-131.

3.2.4 PASSING SIGHT DISTANCE

Consideration of passing sight distance is limited to two-lane, two-way highways on which vehicles frequently overtake slower-moving vehicles and the passing operation must be accomplished on a lane used by opposing traffic.

Passing sight distance for design is determined on the basis of the length needed to accomplish the passing maneuver. Derivation of the required distance is described in the Green Book, pages 118-126. AASHTO recommends that, "In designing ~~highways~~ these distances should be exceeded as much as practical ..."
a highway

These distances for design should not be confused with other distances used as warrants for placing no-passing pavement markings on completed highways. Values shown in the MUTCD are substantially less than the design distances and are derived from traffic operation control needs based on assumptions different from those for design.

Because of vertical and horizontal sight limitations, nearly all two-lane highways have some no-passing restrictions. In rolling terrain, the proportionate amount of no-passing sections usually becomes greater. Normally it is impracticable to attempt to provide passing sight distance throughout the entire length of a project. The principal design consideration is to try to provide adequate passing opportunities as frequently as possible.

There are no fixed values for the frequency of passing sections. Experience shows that highway capacity is measurably reduced when a significant percentage of a section of highway is restricted to sight distances less than 1500 ft [500 m]. Highways with high traffic volumes will require a higher proportion of passing opportunities than those with low traffic volumes. Where an analysis shows that a lack of passing sight distances has reduced capacity to near or below the expected traffic volumes, it is necessary to consider adjustments in the alignment and grade, or to provide additional lanes.

3.3 STANDARDS BASED ON TRAFFIC VOLUMES

Standards not directly related to design speed are influenced primarily by traffic volumes. Tables for these standards shown in the tables at the end of this chapter reflect variations by traffic volume ranges.

3.3.1 NUMBER OF LANES

The number of lanes required for any highway is directly related to the facility's traffic volume and desired level of service. But there are no simple, fixed criteria for these relationships. The recommended number of lanes is normally obtained through the project development process.

The *Highway Capacity Manual* gives two very general guidelines for determining the need for additional lanes. These numbers are based on long sections of roadway with uninterrupted traffic flow having the highest standards for design controls (horizontal and vertical geometrics and cross-sectional elements), ideal weather conditions, daylight, etc.

1. Under ideal conditions, a two-lane rural highway can accommodate about 900 passenger vehicles (two-way) per hour with a reasonably high level of service if there are adequate passing opportunities and no long, steep grades. Considerably more vehicles can be accommodated if motorists are willing to accept a lower level of service, a

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the s 231-250 est available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-4), the Design Criteria Form (Figure 3-5) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there are steep grades accompanied by considerable no-passing distances.

More detailed guidelines and criteria for the design of grades, including critical lengths of grades and minimum and maximum grades are presented in Chapter Five - Alignment and Superelevation and the Green Book, pages ~~235-254.~~ The maximum grades should be used infrequently, only as dictated by severe terrain conditions. When it is necessary to use maximum grades, the designer should check other design criteria and roadside features that may be improved to minimize the impact of using the higher design grade.

3.3.8 BRIDGES

The designer should coordinate with the Bridge Design Section when determining vertical clearances. A minimum vertical clearance for roads over interstate, U.S. and state routes is 16.5 ft [5 m]. Pedestrian bridges and overhead sign structures must have an extra 1 foot [0.3 m] of clearance, a total of 17.5 ft [5.3 m]. These clearances allow for a 4 in. [100 mm] future resurfacing.

3.3.9 MEDIANS

Geometric criteria for medians on multilane divided highways are discussed in Chapter Four.

A width of 40 ft [12.0 m] or more for depressed medians permits adequate drainage design with flat slopes. A median width of at least 50 ft [15.0 m] can safely store a school bus. Wider medians are desirable where right-of-way permits allowing for the placement of a median bridge pier or overhead sign structure without the need for barrier protection. Wider medians should also be considered where there is a potential for adding travel lanes in the median to meet future traffic demand. Also see the Green Book pages ~~460 and 461~~ 456 and 457 for further discussion on this subject.

Where flat longitudinal slopes on the roadway are encountered, the cross slopes of the median may be varied to increase the longitudinal slope of the median ditch. For example, the cross slope may be kept very flat (10:1 or flatter) at the upper end of the drainage area and steeper (6:1) at the lower end.

4.3.4 MEDIAN BARRIERS

For divided highways with large traffic volumes and high operating speeds, a wide, depressed median is the best choice. Under some conditions this is not practicable, and a flush or raised median must be provided. But in this case, some type of physical barrier must be placed in the median to prevent out-of-control vehicles from crossing into opposing traffic lanes.

Several types of physical median barriers can be designed. Criteria for median barriers are discussed in Chapter Ten-Miscellaneous Design and the *Roadside Design Guide*.

4.3.5 MEDIAN OPENINGS

The design of median openings and channelization for left turns is included with the discussion on intersection design in Chapter Seven.

pages 167-174, GB

Figure 5-1
Minimum Radius for Open Highway Conditions and Superelevation Rate of 4%

US Customary		Metric	
Design Speed (mph)	Radius ~~ft~~ (ft)	Design Speed [km/h]	Radius ~~m~~ [m]
15	~~70~~ 42	20	~~15~~ 8
20	~~125~~ 86	30	~~35~~ 22
25	~~205~~ 154	40	~~60~~ 47
30	~~300~~ 250	50	~~100~~ 86
35	~~420~~ 371	60	~~150~~ 135
40	~~565~~ 533	70	~~215~~ 203
45	~~730~~ 711	80	280
50	~~930~~ 926	90	375
55	1190	100	~~490~~ 492
60	~~1505~~ 1500		

Figure 5-2
Minimum Radius for Open Highway Conditions and Superelevation Rate of 6%

US Customary		Metric	
Design Speed (mph)	Radius ~~ft~~ (ft)	Design Speed [km/h]	Radius ~~m~~ [m]
15	~~65~~ 39	20	~~15~~ 8
20	~~115~~ 81	30	~~30~~ 21
25	~~185~~ 144	40	~~55~~ 43
30	~~275~~ 231	50	~~90~~ 79
35	~~380~~ 340	60	~~135~~ 123
40	~~510~~ 485	70	~~195~~ 184
45	~~660~~ 643	80	~~250~~ 252
50	~~835~~ 833	90	~~335~~ 336
55	~~1065~~ 1060	100	~~435~~ 437
60	~~1340~~ 1330	110	560
65	1660	120	~~755~~ 756
70	~~2050~~ 2040	130	~~950~~ 951

3. For multi-lane divided highways with independent roadways or relatively wide medians, independent horizontal and vertical controls are established at the centerline of each roadway.

The relationships between these control line locations and the pivot points for superelevation of horizontal curvature are described in Section 5.3.

5.1.3 TYPES OF CURVES

The types of curves used in designing horizontal curvature may be simple circular curves, spiral transition curves or compound curves. Circular curves use a uniform radius for the entire distance between adjacent tangent sections. Spiral transition sections more closely replicate the vehicle and driver's behavior when entering a curve. They are introduced at each end of the circular curve to gradually ease the driver into and out of curves without a sharp break at the tangent sections. This is particularly noticeable with relatively sharp curves and higher vehicle operating speeds. Compound curves are most commonly used for turning roadways where it is necessary to fit the curve to the inside edge of the design vehicle's swept path. When the design speed of a turning roadway is 45 mph [70 km/h] or less, compound curvature can be used to form the entire alignment of the turning roadway. However, the exclusive use of compound curves can increase the right-of-way impacts.

Although circular curves are normally used in the design of Delaware roadways, using spiral transitions may be considered as described in the Green Book, pages ~~176 to 183.~~ 184 to 192

Where spiral transition curves are to be used, right-of-way lines should not be defined as a spiral curve paralleling the centerline. Instead, the right-of-way should be described with a circular curve or compound circular curve of a similar shape. A practical guide for the length of a spiral is the length required for superelevation runoff.

5.1.4 SIGHT DISTANCE ON HORIZONTAL CURVES

An important element in ensuring driver safety and maintaining a roadway's operational efficiency is providing adequate sight distance—the length of roadway ahead visible to the driver. Sight distance applies to four conditions that may arise when setting a project's horizontal alignment:

(1) Is adequate distance available to stop?

(2) Is there adequate opportunity and length available for passing on two-lane roadways?

(3) Is there adequate distance for drivers to react when approaching complex decision points?

(4) Has the selected criteria for measuring these distances been applied to the selected design?

Providing adequate sight distance is also important in the design of intersections, in particular, those in rural areas. These locations tend to be less safe than urban ones, primarily because of higher speeds and lack of driver awareness. Providing at least the minimum sight distance will play an important role in reducing these occurrences.

5.1.4.1 STOPPING SIGHT DISTANCE

The designer must check sight distance across the inside of horizontal curves. Sight obstructions such as walls, concrete safety barriers, bridge parapets, cut slopes, vegetation and buildings may limit sight distance on curves. Where these obstructions cannot be removed or permanently controlled, adjustment in the normal cross section or a change in alignment may be required to provide and assure continuation of adequate sight distance. For areas within a project that may cause confusion or delay a driver's reaction time i.e. multiple decision points, it may be necessary to check the decision sight distance also.

Minimum stopping sight distance for each design speed is shown in Chapter Three-Design Standards. The sight line is a chord of the curve. The applicable stopping sight distance is checked by measuring along the centerline of the inside lane around the curve. See the Green Book, pages ~~228-232~~ 224-228 for the design and evaluation of stopping sight distances on horizontal curves. Horizontal sight distance is based on the formula:

HSO

$$\cancel{M} = R\left[\left(1 - \cos\frac{28.65\,S}{R}\right)\right]$$

Where:

S = Stopping sight distance, ft [m]
R = radius of curve, ft [m]
~~M = middle ordinate,~~ ft [m]
HSO=Horizontal sightline offset

Where the obstruction is a cut slope on the inside of the curve, it is necessary to know the critical height of vegetation on the slope for measuring the middle ordinate distance. Because the height criteria for stopping sight distance are 3.5 ft [1,080 mm] for the eye and 2 ft [600 mm] for the object, a height of 2.75 ft [840 mm] may be assumed as the midpoint of the line of sight where the cut slope usually obstructs sight. In some cases, retaining walls, concrete median safety barriers, and other similar features constructed on the inside of curves may be sight obstructions and need to be checked for stopping sight distance.

Solutions to sight distance problems on horizontal curves might be removal of obstructions, flattening the curves and flattening or benching cut slopes. It should be kept in mind that stopping sight distances greater than the minimum should be used for design. Minimum stopping sight distance values may be used only if greater values cannot be obtained without undue costs. On new construction, the stopping sight distance at any loca-

tion shall never be less than the minimum standard for stopping sight distance for the selected design speed. Designs for new construction and reconstruction projects that do not meet these standards must have a design exception approved by the Chief Engineer.

5.1.4.2 PASSING SIGHT DISTANCE

The minimum passing sight distance for a two-lane road is about four times greater than the minimum stopping sight distance at the same design speed. To provide the greater passing sight distance, clear sight areas on the insides of curves must be considerably wider. Often this is not practicable. It is necessary to acknowledge and accept no-passing zones.

Passing sight distance depends on the eye height of 3.5 ft [1,080 mm] and object height of 3.5 ft [1,080 mm]. The sight line to the center of the area inside a curve is about 0.75 ft [240 mm] higher than the stopping sight distance.

Perhaps the simplest way to measure passing sight distance is directly from the plans, using a straightedge. Potential obstructions are plotted on the plans. In the case of cut slopes, a dotted line is plotted for the horizontal distance from the centerline of the inside lane to a point on the cut slope 4 ft [1.2 m] above the traffic lane. Because vegetation also blocks vision, its anticipated height must be included in the 4 ft [1.2 m]. The straight edge is placed along the edge of the obstruction (or dotted line), and the intersection with the centerline identifies the sight distance.

Where horizontal curves and vertical curves occur at the same general location, the sight distances for each must be considered together. At least the minimum stopping sight distance must be provided for each. Efforts to provide passing sight distance for one might be completely negated by a no-passing zone situation for the other.

For more information see ~~AASHTO's 2001~~ the Green Book, pages 118 to 131.

5.1.4.3 DECISION SIGHT DISTANCE

Drivers frequently are called on to make decisions concerning vehicle operations. Occasionally, the characteristics of the horizontal alignment can adversely affect the ability to make these decisions. Examples of this include:

- ***Proximity to a Curve***. It is important that the driver has a complete or partial view of the curve ahead to indicate the direction of curvature. With some combinations of vertical and horizontal curvature, the curve may come as a surprise and the driver may have difficulty reacting properly.
- ***Curve Signing***. To be effective, curve signing must be located a considerable distance ahead of the curve. The use of short tangents between curves results in inadequate length for proper signing. Where the design speed of the curve is equal to or greater than the legal posted speed, the length of the tangent should be at least 300 ft [90 m] plus the required distance for superelevation transition.
- ***Route Continuity***. When a driver approaches a diverging roadway situation, such as a Y intersection, an exit ramp on a curve, or a flat-angle intersection, the main route should be distinctly emphasized with sufficient sight distance to eliminate any uncertainty on the part of the driver.

The Green Book, pages 115-117, provides more details and tables of calculated values for checking decision sight distance.

5.1.5 COORDINATION WITH VERTICAL ALIGNMENT

Curvature and grades should be in proper balance. Emphasis on a tangent alignment is not desirable when it results in extremely steep or long grades. An emphasis on flat grades is not desirable when it results in excessive curvature. A compromise between the two extremes is the best approach.

for

Refer to the Green Book ~~show~~ the maximum grades permitted for various combinations of functional classification, traffic volume and terrain. The maximum grades should be used only where absolutely necessary. Grades much flatter than maximum normally should be used.

For short grades less than about 500 ft [150 m] in length, the maximum gradient may be one percent steeper than the values shown in the tables.

5.2.3 MINIMUM GRADES

Minimum grades are primarily related to the need for adequate drainage. For uncurbed pavements that are adequately crowned to drain laterally, relatively flat or even level profile grades may be used. With curbed pavement, the minimum longitudinal grade in usual cases should be 0.5 percent. With a high-type pavement accurately crowned on a firm subgrade, a longitudinal grade of about 0.35 percent may be used. Even on uncurbed pavements, it is desirable to provide a minimum of about 0.35 percent longitudinal grade because the lateral crown slope originally constructed may subsequently be reduced as a result of irregular swell, pavement structure consolidation, maintenance operations or resurfacing. Use of flatter grades may be justified in special cases.

5.2.4 MINIMUM DITCH GRADES

Special attention should be directed to minimum ditch grades. Any ponding of water in the side ditches, particularly on expansive soils, has a very detrimental effect on the subgrade. To ensure continuing flow, ditch grades should be sloped at least 0.5 percent–preferably steeper. This may require some special warping of ditch grades where the roadway profile cannot be adjusted accordingly. A minimum depth of ditch has been established at 2.5 ft [800 mm] below the elevation of the hinge point between the shoulder and frontslope to assure proper drainage of pavement base and subgrade. In superelevated sections both the ditch grade and bottom width may have to be adjusted in order to prevent water ponding onto the shoulder or traveled way.

5.2.5 CRITICAL LENGTH OF GRADE

From the standpoint of vehicle operating characteristics and the effect on highway capacity, the steepness of the grade is not the only factor to be considered. The length of the grade can become a critical factor and must also be considered.

The term “critical length of grade” is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For a given grade, lengths less than critical ones result in acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than critical ones, design adjustments such as change in location to reduce grades or addition of extra lanes should be made. It is recommended that a 10 mph [15 km/h] speed reduction be used as the general guide for determining critical lengths of grades. The Green Book, pages ~~245 and 246,~~ 242 and 243 provides curves showing the critical lengths of grade resulting from various combinations of percent upgrade and designated speed reductions.

On roads with moderate to heavy traffic volumes, where critical lengths are approached or exceeded, and passing opportunities are limited, long lines of smaller vehicles will accumulate behind the slower vehicles. This reduces both the operating speed and highway capacity and, consequently, the level of service. Consideration should be given to providing climbing lanes. A capacity analysis should be conducted to determine whether the addition of a climbing lane is warranted. Procedures for such an analysis are shown in Chapter Ten of the *Highway Capacity Manual*. Factors considered in the analysis include:

- Desired level of service,
- Lane widths and lateral clearance,
- Percent of trucks and buses,
- Passing sight distance,

- Steepness and length of grades,
- Volume/capacity ratio, and
- Service volume.

5.2.6 CLIMBING LANE CRITERIA

The need for climbing lanes in Delaware is seldom warranted. The Green Book pages ~~246 to 254~~ gives a through explanation for the design of these lanes. 241 to 250

5.2.7 VERTICAL CURVES

Vertical curves are used to effect gradual changes between tangent grades at their point of intersection. They have the properties of a simple parabolic curve. The vertical offsets from the tangent grade vary with the square of the horizontal distance from the curve end (point of tangency).

Vertical curves that are offset below the tangent are termed crest vertical curves. Those that are offset above the tangent are termed sag vertical curves. Examples of each curve type are shown in Figure 5-4.

The minimum lengths of crest vertical curves are determined mainly by sight distance requirements. These lengths generally are satisfactory from the standpoint of safety, comfort and convenience. An exception may be at decision areas, such as intersections and approaches to ramp exit gores, where adequate sight distance requires longer lengths.

Passing sight distance seldom can be attained on a crest vertical curve simply by lengthening the curve. Excessively long vertical curves often reduce the length of passing opportunities on the adjacent tangent sections on either side of the crest. They also can adversely impact roadway and roadside ditch drainage systems.

Sag vertical curves use four different criteria for determining their lengths:

(1) headlight sight distance,

(2) passenger comfort,

(3) drainage control, and

(4) general appearance.

The primary control used in design is headlight sight distance.

5.2.8 VERTICAL CURVE DESIGN

The principal concern in designing vertical curves is to ensure that at least the minimum stopping sight distance is provided. The values set forth in the design standards for stopping sight distance are also applied to vertical curves. Refer to the Green Book pages ~~269 to 281~~for more design detail. 265 to 280

For crest vertical curves, the design eye height is 3.5 feet [1,080 mm] and the object height is 2.0 ft [600 mm]. The crest of the curve should not obstruct the line of sight.

Nighttime driving conditions govern sag vertical curves. The sight distance control is the height of headlight and the distance illuminated to an object rather than driver eye height. The distance illuminated is that of a headlight beam with an assumed upward divergence of 1 degree and headlight mounting height of 2 ft [600 mm]. Equations found in the Green Book are used to determine these values for various design speeds. For overall safety, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. The values in Figure 5-6 were developed using the design stopping sight distance as the light beam distance.

For passing sight distance, the controls are different than for stopping sight distance. The design height of the eye remains at 3.5 ft [1,080 mm], but the height of the object (oncoming car) is increased to 3.5 ft [1,080 mm].

By analyzing the requirements relating to sight distances and the characteristics of the curve, determinations can be made as to the minimum permissible length of curve for particular situations. A ride control criterion for vertical curve length of not less than three times the design speed in mph [0.6 times the design speed in km/h] is recommended for comfort.

The minimum length of a vertical curve is computed by the following formula:

on street sections usually are considerably shorter than for comparable locations on rural roads.

Where controlling factors are not severe, the normal practice of carrying the profile grade on the centerline or on the median edges of pavement will work satisfactorily. Where outside controls are significant, it may be necessary to supplement the main profile with other elevation controls, such as gutter-line profiles or top-of-curb profiles. Where this is necessary, the supplemental controls should be clearly shown on the typical sections, profiles, and grades and geometrics sheets.

Special attention must be given to existing features when designing urban grades. This is particularly true in the case of private driveways when a street is being widened. With even moderate driveway grades, up or down, angular breaks must be kept flat enough with adequate clearance so that the undercarriage or bumpers of vehicles will not drag. Reference should be made to the Department's publication *DelDOT Entrance Manual*.

Where roadside development is extensive and the general elevation is higher on one side than on the other, an unsymmetrical section may be required. The crown point (and profile grade) may be offset from the centerline so the total drop from the crown line to the gutter line will be more than normal on one side and less than normal on the other. However the location of the crown point must be at the edge of the travel lane.

5.3 SUPERELEVATION

The transitional rate of applying superelevation into and out of curves is influenced by design speed, degree of curvature and number of lanes. Introducing superelevation permits a vehicle to travel through a curve more safely and at a higher speed than would be possible with a normal crown section. For a given degree of curvature, a steeper superelevation is required for a higher design speed than is needed for a lower design speed. For a given design speed more superelevation is needed through sharp curves than for relatively flat curves.

The maximum rates of superelevation used on roadways are controlled by four factors:

(1) Climate conditions (i.e. frequency of ice and snow);

(2) Terrain conditions (i.e. flat or rolling);

(3) Type of area (i.e. rural or urban); and

(4) Frequency of slow-moving vehicles.

Basic design controls for superelevation are presented in Chapter Three. Rural roadways are usually designed with a maximum superelevation rate of 6 percent but it may be appropriate to use a rate of 8 percent. Urban roadways are normally to be designed with a superelevation rate of 4 percent. Superelevation may be omitted on low-speed urban streets subjected to severe constraints. The selected superelevation rate establishes the minimum permissible radius of curve based on a project's design speed.

This section discusses practical application of superelevation criteria, with particular attention to:

- The rates of superelevation to be used for various combinations of design speed and curve radius,
- The manner of transition of slope between normal tangent sections and superelevated sections on curves, and
- Special criteria for superelevation of shoulders and auxiliary lanes.

5.3.1 RATES OF SUPERELEVATION 165 to 174

The Green Book, pages ~~155 to 159~~, sets forth the basic design criteria based on design speeds for the normal design superelevation rates of e_{max} = 4 and 6 percent as well as other values ranging up to 12 percent. The criteria shown includes the minimum radius of curvature, crown treatment and superelevation runoff lengths (L), all of which are related to the number of lanes to be rotated. The minimum rate of cross slope for a traveled lane is determined by drainage requirements.

Figure 5-9*
Minimum Superelevation Runoff and Tangent Runout Lengths (US Customary)

Minimum runoff and runout length (ft)				
Design speed (mph)	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
25	34	69	103	34
30	36	73	109	36
35	39	77	116	39
40	41	83	124	41
45	44	89	133	44
50	48	96	144	48
55	51	102	153	51
60	53	107	160	53
65	56	112	~~168~~ 167	56
70	60	120	180	60
Two lanes rotated				
25	51	103	154	51
30	55	109	164	55
35	58	116	174	58
40	62	124	186	62
45	67	133	200	67
50	72	144	216	72
55	77	153	230	77
60	80	160	240	80
65	84	~~168~~ 167	~~252~~ 251	84
70	90	~~190~~ 180	270	90

Figure 5-10*
Minimum Superelevation Runoff and Tangent Runout Lengths [Metric]

Minimum runoff and runout length [m]				
Design speed [km/h]	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
20	9	18	27	9
30	10	19	29	10
40	10	21	31	10
50	11	22	~~32~~ 33	11
60	12	24	36	12
70	13	26	39	13
80	14	29	43	14
90	15	31	46	15
100	16	33	49	16
110	18	35	53	18
Two lanes rotated				
20	14	27	41	14
30	14	29	43	14
40	15	31	46	15
50	~~16~~ 17	~~32~~ 33	~~49~~ 50	~~16~~ 17
60	18	36	54	18
70	20	39	59	20
80	22	43	65	22
90	23	46	69	23
100	25	49	74	25
110	26	53	79	26

*Note: Figures 5-9 and 5-10 are based on 12-ft [3.6 m] lanes and 2.0% normal cross slope

Chapter Seven

Intersections

The intersection of two or more roads presents an opportunity for conflict among vehicles. For freeways, the potential for conflict is significantly reduced through the use of interchanges. But interchanges usually are not feasible for the vast majority of intersections on arterials and collectors. This chapter is a general discussion of intersection design with those elements of particular application to this state. The details on intersection design are found in Chapter 9 of AASHTO's ~~2001~~ Green Book.

The principal objectives in the design of at-grade intersections are:

- To minimize the potential for and severity of conflicts,
- To provide adequate capacity, and
- To assure the convenience and ease of drivers in making the necessary maneuvers.

In the design of intersections there are three elements to consider:

(1) Perception-reaction distance,

(2) Maneuver distance, and

(3) Queue-storage distance.

The distance traveled during the perception-reaction time varies with vehicle speed, driver alertness, and driver familiarity with the location. Where left-turn lanes are introduced, this distance includes that to brake and change lanes. Where no turn lanes are provided, the distance needed is for the driver to brake comfortably. The storage length should be sufficient to accommodate the longest queue most commonly experienced.

An important consideration in the design of intersections is the treatment of right-turn lanes. Right turns can be free flowing, yield or stop controlled. In order to operate properly, free flowing right-turn lanes need to have an adequate acceleration distance free of access points for drivers to safely merge into the through traffic. Some drivers, particularly older drivers, are apprehensive when entering another leg of an intersection and may stop or slow down in the merge lane until the lane is clear of traffic. However, when properly designed, the majority of drivers will use the lane as proposed.

7.1 GENERAL CONSIDERATIONS

This section describes the various types of intersections and the general criteria that must be considered during design. Project intersection design configurations are developed during the project development phase based upon capacity analysis, accident studies, pedestrian use, bicycle use and transit options. In addition, design-hour turning movements, size and operating characteristics of the predominant vehicles, types of movements that must be provided, vehicle speeds, and existing and proposed adjacent land-use are considered.

Intersection designs range from a simple residential driveway to a complicated convergence of several high-volume multi-lane roadways. They all have the same fundamental design elements: (1) level of service, (2) alignment, (3) profile, (4) roadway cross section(s), and (5) sight distance. However, other

signals, or where signals may be warranted in the near future, it may be desirable to warp the crowns of both roads to avoid a pronounced hump or dip in the grade line of the minor highway. Intersections in superelevation areas are difficult to provide smooth grades or adequate drainage for and should be avoided.

7.1.5 FRONTAGE ROAD INTERSECTIONS

When a divided arterial highway is flanked by a frontage road, the problems of design and traffic control are more complex. Four separate intersections actually exist at each cross street.

The problem becomes more severe when the distance between the arterial and frontage road is relatively small. Generally, the outer separation between the two roadways should be 150 ft [50 m] or more.

Quite often, right-of-way considerations make it impractical to provide the full desired outer separation width. The alternative is to accept a narrow outer separation between cross roads and design a bulb-shaped separation in the immediate vicinity of each cross road.

7.1.6 DISTANCE BETWEEN INTERSECTIONS

Criteria for location, frequency and layout of private entrances and driveways are documented in DelDOT's *Standards and Regulations for Access to State Highways*. Illustrative sketches are shown for typical entrance and driveway designs for various conditions. For other types of public intersections, there are no fixed criteria as to frequency or distance between intersections. However, intersection spacing should provide sufficient distance to allow the proper development of all necessary turning lanes, bypass lanes, and, if signalized, proper signal coordination. Ideally this distance should be at least 350 ft [110 m] or more. Where intersections are closely spaced, several considerations should be kept in mind. It may be necessary to impose turn restrictions at some locations, prohibit pedestrian crossings, or provide frontage roads for access to intersecting roads. Where crossroads are widely spaced each at-grade intersection must necessarily accommodate all cross, turning and pedestrian movements.

7.2 TURNING MOVEMENTS

All intersections involve some degree of vehicular turning movements. There are various factors that influence the geometric design of turning lanes. The design controls for turning roadways are the traffic volume and types of vehicles making the turning movement. The roadway of primary concern is that used by right-turning traffic but may also be used for other roadways within the intersection. Figure 7-1 shows the terminology used when designing turning movements. The outer trace of the front bumper overhang and the path of the inner rear wheel establish the boundaries of the turning paths of a design vehicle.

The three typical types of designs for right-turning roadways in intersections are:

(1) A minimum edge-of-traveled-way design (Green Book, pages ~~587 to 623),~~ 583 to 621

(2) A design with a corner triangular island (Green Book, pages ~~638 to 643),~~ and 634 to 639

(3) A free-flow design using simple radius or compound radii (Green Book, pages ~~643 to 653~~ 639 to 649). The turning radii and pavement cross slopes for free-flow right turns are functions of design speed and type of vehicle.

general classes of vehicles: (1) passenger cars, (2) buses, (3) trucks, and (4) recreational vehicles. The passenger car class includes passenger cars of all size, sport/utility vehicles, minivans, vans and pickup trucks. The bus class includes inter-city buses (motor coaches), city transit, school, and articulated buses. The truck class includes single-unit trucks, truck tractor-semitrailer combinations, and truck tractors with semitrailers in combination with full trailers. The recreational vehicle class includes motor homes, passenger cars with campers, cars with boat trailers, motor homes with boat trailers, and motor homes pulling cars. In addition, the bicycle should also be considered a design vehicle where applicable.

The dimensions for the design vehicles within these classes are listed in the Green Book. In the design process, the largest design vehicle likely to use that facility and its turning roadways with considerable frequency, or a design vehicle with special characteristics, is used to determine the design of such critical features as radii at intersections and radii of turning roadways.

A general guide to selecting a design vehicle is as follows:

- **P design vehicle** would be used for residential driveways, roadways with restricted truck use, local road intersections with a major roadway where use is infrequent, and low volume-minor road intersections. In most cases the selection of the SU design vehicle is preferred in all of these cases. The radii and widths permitted for the P design vehicle are very awkward for a single-unit delivery or service truck to safely maneuver.
- **SU design vehicle** provides the most economical minimum edge-of-traveled way design for rural roadways and other light truck use intersections. However, current and projected truck use on these roadways needs to be considered before a final design vehicle selection, particularly if any channelization is proposed.
- **Semitrailer combinations design vehicles** should be used where truck combinations will turn repeatedly, particularly heavily industrialized or commercial areas. Providing for these vehicles increases the paved areas, radii and other design parameters. Even in rural areas the local economy may be based on frequent semitrailer usage. Project development and scoping should identify these areas.

A project may have several design vehicles depending upon the predominant type of vehicle using the turning roadways being designed. A residential driveway would only need to consider a passenger car with an occasional single unit delivery truck. Industrial entrances would consider the predominant semi-trailer unit as the design vehicle. Other turning roadways and intersections would have to be similarly analyzed and an appropriate design vehicle selected. The purpose of this analysis is to assure the physical features are placed in positions that allow for the movement without making unsafe maneuvers, particularly within the through travel lanes, or destroying roadway features (curbs, signs, light poles, etc.).

Figure 7-2 shows selected minimum radii for the more commonly used design vehicles. Figure 7-3 (also see the Green Book, pages ~~198 to 203)~~ 216 to 224 shows the minimum design radii at the inner edge of the traveled way for roadway curves within an intersection based on a superelevation rate of 8.0 percent and free flow. For design speeds above 45 mph [70 km/h], the values are based on open road conditions. (See the Green Book, page ~~145.~~ 147) At intersections controlled by stop signs, lower rates of superelevation are usually more appropriate. See the Green Book, ~~page 197~~ pages 150 to 151, for urban streets.

Figure 7-2
Minimum Turning Radii for Selected Design Vehicles

Design Vehicle Type	Symbol	US Customary		Metric	
		Minimum Design Radius (ft)	Minimum Inside Radius (ft)	Minimum Design Radius [m]	Minimum Inside Radius [m]
Passenger Car	P	24	14.4	7.3	4.4
Single Unit Truck	SU	42	28.3	12.8	8.6
Intercity Bus	BUS-40 [BUS-12]	45	27.6	13.7	8.4
City Transit Bus	City-Bus	42	24.5	12.8	7.5
Conventional School Bus (65 pass.)	S-BUS36 [S-BUS11]	38.9	23.8	11.9	7.3
Large School Bus (84 pass.)	S-BUS40 [S-BUS12]	39.4	25.4	12.0	7.7
Intermediate Semi-trailer	WB-40 [WB-12]	40	19.3	12.2	5.9
Intermediate Semi-trailer	WB-50 [WB-15]	45	17.0	13.7	5.2

Figure 7-3 Inner Edge
Minimum Radii at ~~inner edge~~ of Traveled Way for Intersection Curves–Free Flow

US Customary	
Design Speed (turning) (mph)	Minimum Radius (ft)
10	25
15	50
20	90
25	150
30	230
35	310
40	430
45	540
Metric	
Design Speed (turning) [km/h]	Minimum Radius [m]
10	7
20	10
30	25
40	50
50	80
60	115
70	160

7.2.2 EDGE-OF-TRAVELED-WAY DESIGNS

In the design of the edge of pavement for the minimum path of a given design vehicle, it is assumed that the vehicle is properly positioned within the traffic lane at the beginning and end of the turn. This position is 2 ft [0.6 m] from the edge of 12 ft [3.6 m] wide pavements on the tangents approaching and leaving the intersection curve.

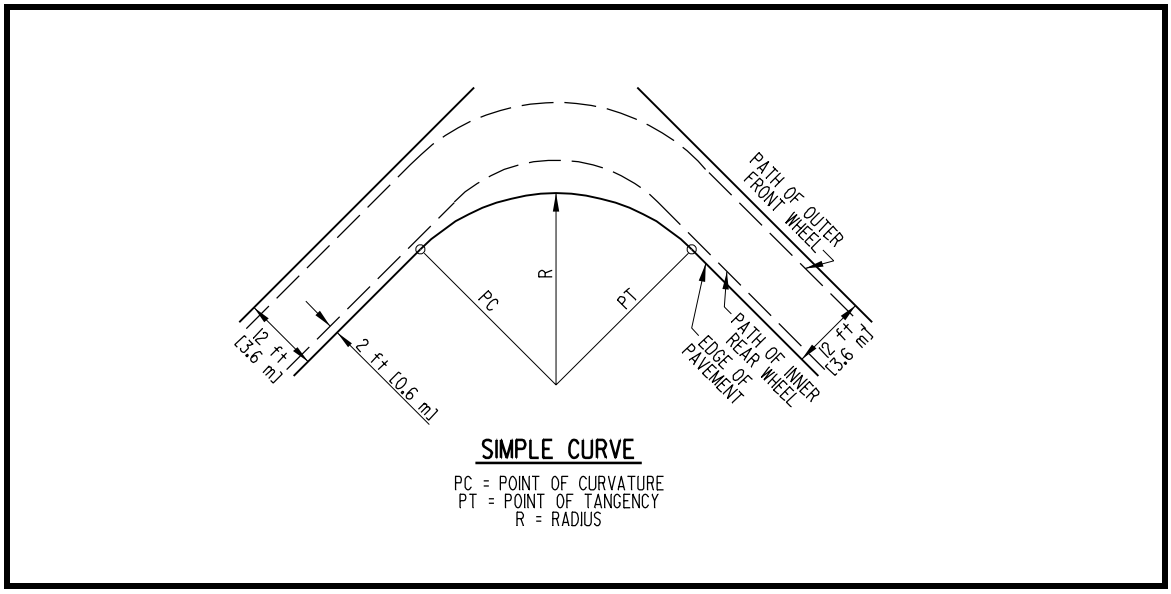
Three types of curves commonly are used for the design of pavement edges at intersections:

- simple curve,
- 3-centered symmetric compound curve, and
- 3-centered asymmetric compound curve.

Use of the simple curve usually is limited to residential driveways and low traffic volume intersections where there is little truck traffic. The 3-centered curve should be used

for edge-of-traveled-way design at all major intersections. Figures 7-4 and 7-5 illustrate the three types of edge-of-traveled-way designs. The ~~AASHTO's 2001~~ Green Book, Exhibits 9-19 and 9-20 on pages ~~588 through 595~~ 584 through 591 tabulate values for application of simple and three-centered compound curve applications for various angles of turning roadways. The angle of turn that is the next highest to the angle of turn of the intersection being designed should be selected.

Figure 7-4
Intersection Edge-of-Traveled-Way Design Layout
Using Simple Curves



7.2.3 PAVEMENT WIDTHS FOR TURNING ROADWAYS

The pavement and roadway widths of turning roadways at intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated, and may be designed for one-way or two-way operation, depending on the geometry of the intersection. Widths determined for turning roadways may also apply on through roadways within an intersection, such as channelizing islands.

Pavement widths for turning roadways are classified for the following types of operations:

Case I – one-lane, one-way operation with no provision for passing a stalled vehicle;

Case II – one-lane, one-way operation with provision for passing a stalled vehicle; and

Case III – two-lane operation, either one-way or two-way.

Case I widths are normally used for minor turning movements and for moderate turning volumes where the connecting roadway is relatively short. The chance of vehicle breakdown is remote under these conditions, but one of the edges of pavement should be available for passing a stalled vehicle, i.e. mountable curb and clear of obstructions.

Case II widths are determined to allow operation at low speed and with restricted clearance past a stalled vehicle. These widths are applicable to all turning movements of moderate to heavy volumes that do not exceed the capacity of a single-lane connection. In the event of a breakdown, traffic flow can be maintained at somewhat reduced speed. Many ramps and connections at channelized intersections are in this category.

Case III widths apply where operation is two-way, or one-way with two lanes needed to handle the traffic volume. In the latter case, downstream lanes must be able to accommodate the two-lane volume. In each category the required pavement width depends jointly on the size of the design vehicle and the curvature of the turning roadway. Selection of the design vehicle is based on the size and frequency of vehicle types. The pavement width increases with both the size of the design vehicle and the sharpness of curvature. See Figures 7-6 and 7-7 for the recommended design widths of pavements for turning roadways at intersections for three types of operations and for three conditions of traffic mixes.

The designer should refer to the ~~2001~~ Green Book, pages ~~191 to 228,~~ 199 to 229 for further details on designing roadways within intersections.

7.3 CHANNELIZATION

Channelization is the separation or regulation of conflicting-traffic movements into definite paths of travel by traffic islands or pavement markings to facilitate the safe and orderly movement of both vehicles and pedestrians. Proper channelization increases capacity, improves safety, provides maximum convenience, and instills driver confidence. Improper channelization has the opposite effect and may be worse than none at all. Over channelization should be avoided because it could create confusion and deteriorate operations.

7.3.1 PURPOSE

Channelization of at-grade intersections is generally warranted for one or more of the following factors:

- The paths of vehicles are confined by channelization so that not more than two paths cross at any one point.
- The angle and location at which vehicles merge, diverge or cross are controlled.
- The paved area is reduced, thereby narrowing the area of conflict between vehicles and decreasing the tendency of drivers to wander.
- Clearer indications are provided for the proper path in which movements are to be made.
- The predominant movements are given priority.
- Areas provide for pedestrian refuge.
- Separate storage lanes permit turning vehicles to wait clear of through-traffic lanes.
- Space is provided for traffic control devices so they can be more readily perceived.
- Prohibited turns are controlled.

7.3.2 DESIGN PRINCIPLES

Design of a channelized intersection usually involves the following significant controls—the type of design vehicle, the cross sections on the crossroads, the projected traffic volumes in relation to capacity, the number of pedestrians, the speed of vehicles, and the type and location of traffic control devices. Furthermore, physical controls such as right-of-way and terrain have an effect on the extent of channelization that is economically feasible.

Figure 7-6
Design Widths for Turning Roadways (US Customary)

Radius on Inner Edge R (ft)	Pavement Width (ft)								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	14	17	18	20	24	26	26
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 1 ft Add 2 ft			None Add 1 ft			Add 1 ft Add 2 ft		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 12 ft where shoulder is 4 ft or wider			Deduct shoulder width; minimum width as under Case I			Deduct 2 ft where shoulder is 4 ft or wider		

Note:

Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination vehicles.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Figure 7-7
Design Widths for Turning Roadways [Metric]

Radius on Inner Edge R [m]	Pavement width [m]								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 0.3 m Add 0.6 m			None Add 0.3 m			Add 0.3 m Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width; minimum width as under Case I			Deduct 0.6 m where shoulder is 1.2 m or wider		

Note:

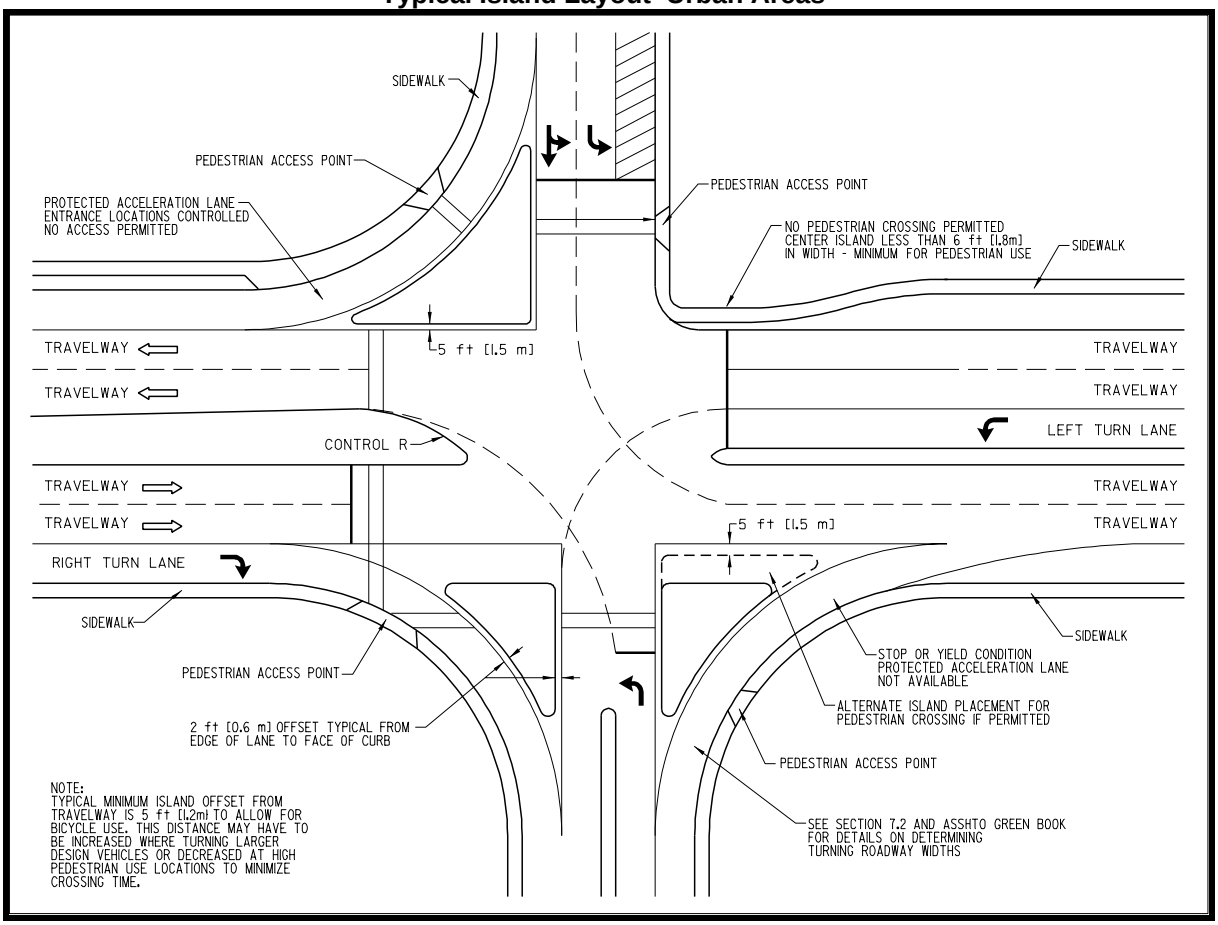
Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination trucks.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Figure 7-9
Typical Island Layout–Urban Areas



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7.4 SIGHT DISTANCE

The operator of a vehicle approaching an at-grade intersection should have an unobstructed view of the whole intersection and of a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions which is termed "approach sight distance". (See Figure 7-10.) The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and the resultant distances traversed during perception, reaction time, and braking. In addition to approach sight distance, sight distance is also provided to allow stopped vehicles sufficient view of the intersecting roadway to decide when to enter the intersecting roadway or to cross it, which is "departure sight distance." Both sight distances must be checked on all intersection designs based on the procedures set forth in the Green Book, pages ~~654 to 682.~~ 650 to 679

7.4.1 MINIMUM SIGHT DISTANCE TRIANGLE

Sight triangles are areas of unobstructed sight along both roads at an intersection and across their included corner for a distance sufficient to allow the operators of vehicles approaching simultaneously to see each other in time to prevent collision at the intersection.

The length of the legs may vary based upon traffic volumes, design speeds, operating speeds and type of intersection traffic control.

Normally in less densely populated areas the minimum sight distance of any leg would be that required to meet the design stopping sight distance for the major road. At high volume intersections the need for large sight triangles is diminished and is a function of the types of traffic control devices and the presence or absence of other vehicles. The Green Book, pages ~~658 to 682,~~ 654 to 679 provides details for determining sight triangles for several different conditions that may occur at intersections, primarily based on the type of traffic control.

In each case, assumptions are made about the physical layout and the actions of vehicle operators on both intersecting roads. For each case, the space-time-velocity relations indicate the minimum sight triangle that is required to be free of obstructions. Any object within the sight triangle high enough above the elevation of the adjacent obstruction should be removed or lowered. Such objects include cut slopes, trees, hedges, bushes, or tall crops. There should be no parking within the sight triangle.

The minimum stopping sight distance in the Green Book for open highway conditions are also valid for turning roadway intersections of the same design speed. Figure 7-11 includes stopping sight distance for lower turning speeds than commonly used under open roadway conditions. These values should be available at all points along a turning roadway, and should be increased wherever practical. They apply to both vertical and horizontal alignment.

Figure 7-10
Sight Distance Triangles—Elements for At-Grade Intersections

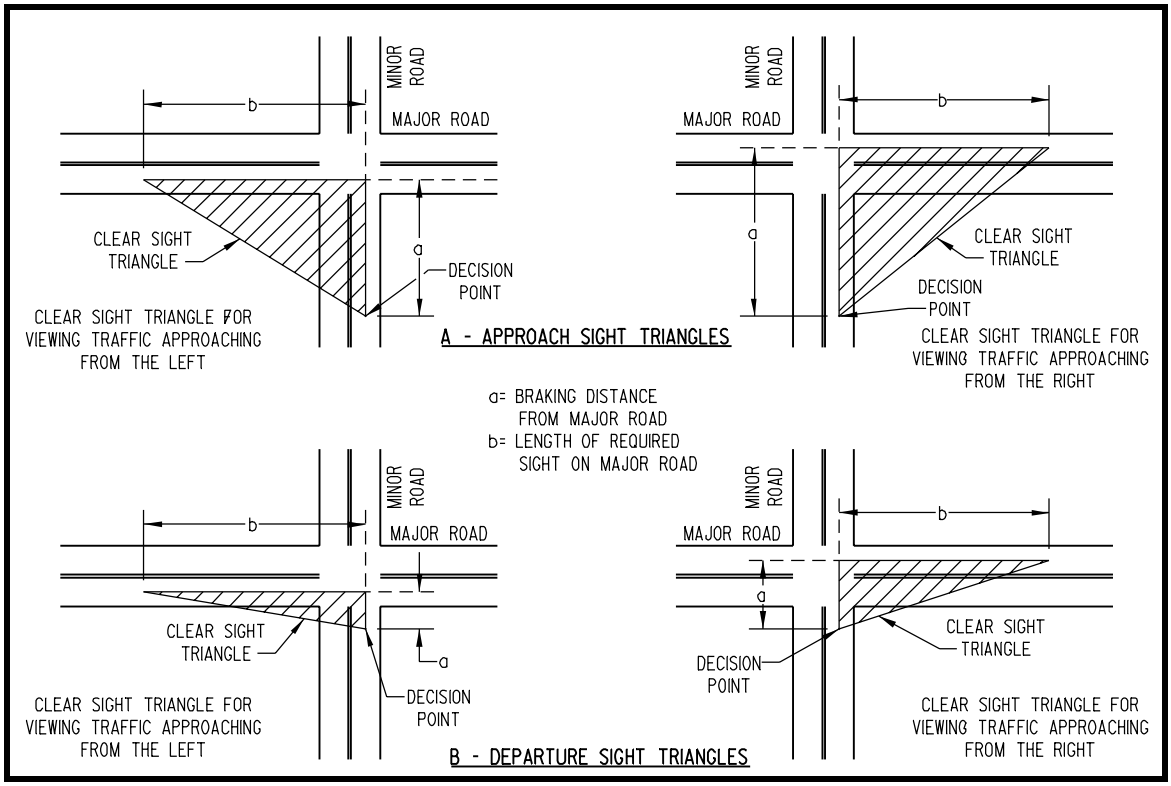


Figure 7-11
Minimum Stopping Sight Distance for Turning Roadways at Intersections

US Customary	
Design (Turning)Speed (mph)	**Stopping Sight Distance (ft)**
10	50
15	80
20	115
25	155
30	200
35	250
40	305
45	360
Metric	
Design (Turning)Speed [km/h]	**Stopping Sight Distance [m]**
15 ~~10~~	15
20	20
30	35
40	50
50	65
60	85
70	105

7.4.2 INTERSECTION MANEUVERS

When traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position. There are three basic maneuvers that occur at the average intersection. These maneuvers are:

1. To travel across the intersecting roadway by clearing traffic from both the left and the right of the crossing vehicle,
2. To turn left into the crossing roadway by first clearing traffic on the left and then entering the traffic stream with vehicles from the right, and
3. To turn right into the intersecting roadway by entering the traffic stream with vehicles from the left.

The stop condition criterion is applicable to two-lane, two-directional roadways through multi-lane divided highways. Where the principal roadway is either undivided or divided with a narrow median (the median is too narrow to store the design vehicle), the departure maneuvers are treated as a single operation. Where the major roadway is divided and has a median wide enough to safely store the design vehicle, the departure maneuvers are considered as two operations. The first operation concerns the traffic approaching from the left for all three maneuvers; that is, crossing the entire roadway, crossing part of the roadway and turning left into the crossroad or turning right into the crossroad. The second phase concerns traffic from the right for the first two operations; i.e., continuing to cross the major roadway or turning left and merging with traffic from the right. The Green Book, pages ~~654 to 682,~~ 650 to 679 provides details on analyzing the departure sight triangles for these maneuvers.

7.5 AUXILIARY TURNING LANES

Auxiliary turning lanes may be introduced at intersections under a variety of conditions including rural or urban locations and free flowing, signalized or stop controlled traffic designs. Using auxiliary lanes to handle turning movements at high volume intersections can reduce congestion, improve safety and provide better traffic control. Auxiliary lanes are also used on four-lane divided roadways and high volume two lane roadways under open road conditions. They improve safety and traffic flow when introducing median openings, intersections at minor crossroads or U-turn locations.

Straight tapers at least 100 ft [30 m] long should be used for deceleration lanes.

The designer has at least four methods available (listed from the preferred to the least acceptable) for determining deceleration lane lengths: (1) design the intersection in accordance with the HCM based on detailed existing and projected traffic data, (2) provide the desirable lengths as discussed in the Green Book (3) design left turn lanes based on the methodology shown on Figure 7-17 or (4) provide the minimum lengths as discussed in this section and shown in Figure 7-18. The use of each of these approaches is also dependent upon the roadway classification, type of facility, the location of the intersection within the facility, project scope and funding.

The Green Book proposes that for arterials with a selected design speed of 30, 40, 45, 50 and 55 mph [50, 60, 70, 80 and 90 km/h], the desirable deceleration lengths of the auxiliary lanes, where practical, are ~~230, 330, 430, 550 and 680 ft [70, 100, 130, 165, and 205 m],~~ 170, 275, 340, 410 and 485 ft [50,70, 95, 120 and 150m] driver to from the full design speed with grades of 3 percent or less. These values do not include taper or required storage length.

The Green Book further discusses the fact that on many urban facilities providing the full length is not practical, physically possible or economically reasonable to provide the suggested desirable lengths needed for decelerating from design speed or operating speed to a full stop condition. On urban facilities in densely developed areas, the need for storage length may override the desirable deceleration length. The Green Book concludes that on urban and collectors the designer may assume that a portion of the deceleration speed is accomplished in the through lane and/or on the taper before entering the full width auxiliary lane. The Green Book further states that: “Therefore, the lengths given above should be accepted as a desirable goal and should be provided where practical.”

Figure 7-17 illustrates a design methodology for determining a reasonable minimum length for an auxiliary turning lane under open highway conditions when complete traffic data is not available. In this figure, the typical average running speed on the main facility is used and some deceleration for the left-turn movement is assumed to occur prior to entering the turning lane. Based on assumed vehicle approach speeds, the desirable deceleration lengths are shown in Figure 7-18. See the Green Book, page ~~855,~~ 851 for lengths applicable to other exit curve design speeds.The lengths shown do not include any taper lengths or required storage lengths. These lengths are for open highway conditions. It should be recognized that operating speeds, traffic volumes, traffic mix, type of facility, project intent, roadside development, and intersection frequency and spacing all influence a designer’s ability to provide the lengths shown in Figure 7-18.

To reiterate, in the use of Figure 7-17, DelDOT has adopted the recognition in the Green Book, page ~~718~~ 714, that degree of deceleration can safely take place in the through lane depending upon posted speed, type of facility and traffic volumes. The suggested design approach for arterial and other high volume roadways assumes a reduction of 10 mph [15 km/h] below posted speed occurs in the through lane. For collectors and other medium volume roadways, an assumed reduction of 15 mph [20 km/h] is practical. For low volume collectors and local streets, a reduction of 20 mph [30 km/h] below the posted speed may be assumed in the design of auxiliary lanes. The deceleration lengths shown in the figures are applicable to both left and right-turn lanes.

Figure 7-17 is a general guide for use when the designer does not have existing or projected traffic volumes or turning counts. When this data is available the length and design of auxiliary lanes should be analyzed in accordance with the HCM.

Figure 7-12
Guide for Need for Left-Turn Lanes on Two Lane Highways

Opposing volume (vph)	Advancing volume (vph)			
	5% Left turns	10% Left turns	20% Left turns	30% Left turns
40-mph [60 km/h] operating speed				
800	330	240	180	160
600	~~420~~ 410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph [80 km/h] operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph [100 km/h] operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

For signalized intersections when there are no current or projected traffic counts or studies available to indicate the needed storage length, then the following method suggested by AASHTO can be applied.

Storage length is based on the number of vehicles likely to arrive in an average cycle time period within the peak hour in accordance with the following formula:

$$S.L. = (N \ / \ C) \times V.L. \times 1.5$$

Where:

S.L. = Storage Length,
V.L. = Vehicle length—use 20 ft [6.0 m] for passenger cars,
N = Number of left-turn vehicles in peak hour, and
C = Number of cycles per hour.

At unsignalized intersections, the average cycle time is assumed to be 2 minutes, so:

$$C = \frac{60 \text{ minutes per hour}}{2 \text{ minutes}} = 30$$

At signalized intersections, "C" is computed using the actual cycle time, so:

$$C = \frac{60 \text{ minutes per hour}}{\text{Actual cycle time in minutes}}$$

Where there is a demonstrated need due to turning volumes versus available gaps in the opposing traffic, the recommended minimum storage length for median auxiliary lanes is 50 ft [15 m]; for separate left turn facilities where no median exists, the minimum recommended storage length is 100 ft [30 m]. These lengths will allow for storing one P and one SU design vehicle or an occasional WB-50 [WB-15]. The greater length where there is no median provides allowance for a decrease in available turning paths.

Acceleration lanes for right-turning vehicles entering a traveled way may need to be considered when turning volumes exceed 100 vph. However, as discussed earlier in this chapter acceleration lanes are not always desirable where entering drivers can wait for an opportunity to merge without disrupting through traffic, such as at a signalized intersection. The use of acceleration lanes should generally be restricted to rural, free-flow, or

controlled access situations. The length of the acceleration lane is a function of the pooled highway speed and speed of the turning vehicle as shown in Figure 7-19. The Green Book, page ~~851~~, gives additional lengths for other selected entrance curve design speeds.

7.5.1 MINIMUM TURN LANE LENGTHS

A project's intent or funding may not allow for providing the desirable left-turn lengths as suggested in the Green Book, or be designed in accordance with the HCM or the method shown on Figure 7-17. In this situation and for uniform application, the suggested minimum lengths for left turn lanes are as described in this section. The parameters are:

- Suggested minimum lengths apply to divided roadways at unsignalized locations.
- No previously identified history of problems with accidents, operation or safety.
- No established warrants based on traffic volume.
- Locations with observed or anticipated high truck use need more storage length.

The lengths are determined by general class of roadway. It should be recognized that each location is unique and has to be analyzed based on its characteristics, including traffic control devices and the selected length may be different than those that follow.

Divided rural arterials and collectors with moderate to heavy through traffic with a posted speed of 50 mph [80 km/h] or greater use a taper length of 100 ft [30 m], deceleration length of 250 ft [75 m], and storage length 100 ft [30 m].

Divided rural arterials and collectors with light through traffic and a posted speed of 50 mph [80 km/h] or greater use a taper length of 100 ft [30 m], deceleration length of 150 ft [45 m], and storage length of 50 ft [15 m].

Divided urban arterials and collectors with heavy to moderate through traffic and a posted speed of 50 mph [80 km/h] or less use a taper length of 100 ft [30 m], deceleration length of 200 ft [60 m], and storage length of 100 ft [30 m].

Divided urban arterials and collectors with light through traffic and a posted speed of 50 mph [80 km/h] or less use a taper length of 100 ft [30 m], deceleration length of 200 ft [60 m], and storage length of 50 ft [15 m].

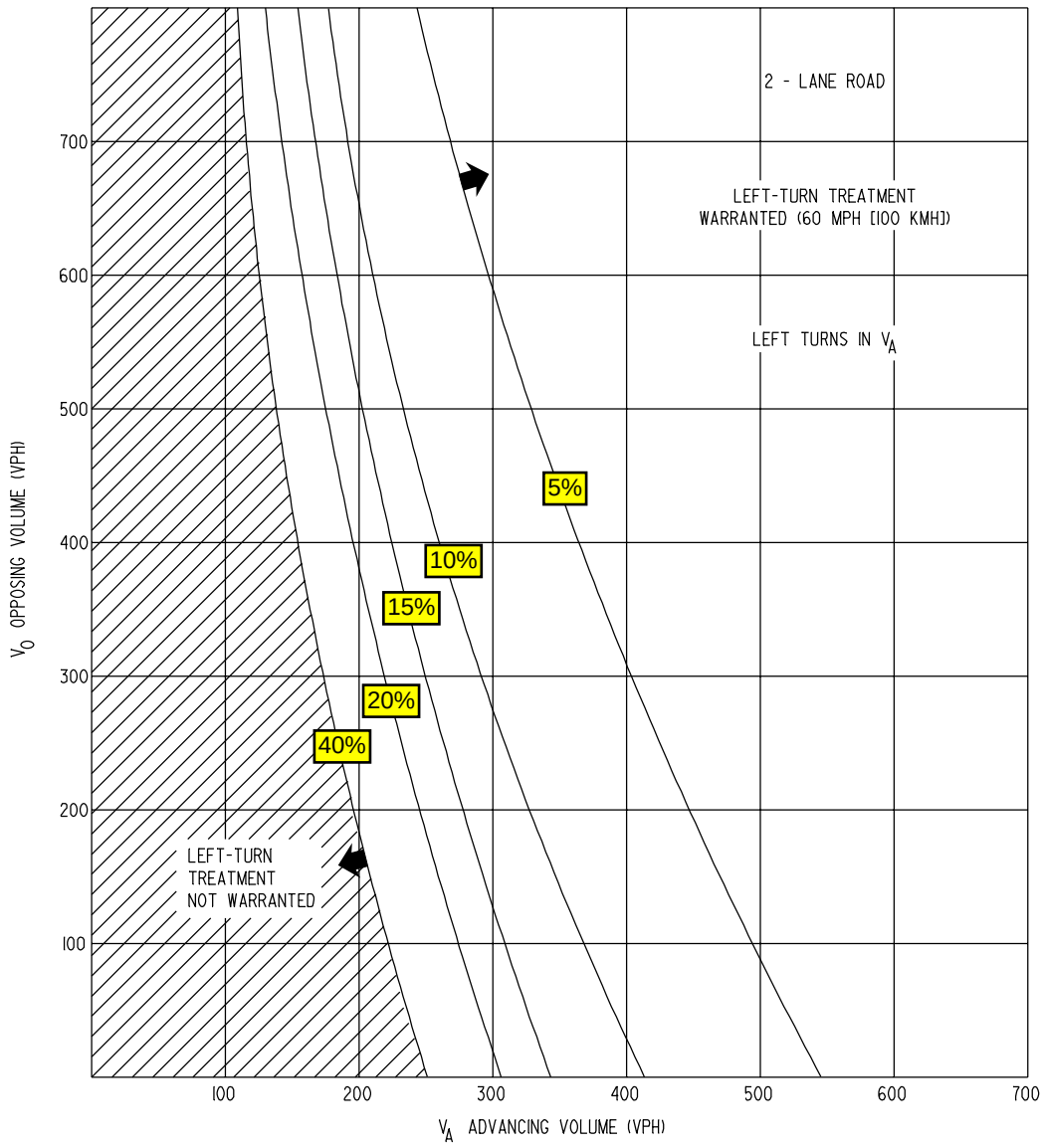
The designer should refer to the Green Book and the HCM for further discussion concerning these guidelines.

7.6 MEDIAN OPENINGS

The following is a general discussion of median opening design. The Green Book, pages ~~693 to 732~~, presents a comprehensive discussion on the concepts and design of median openings.

Median opening designs range from designing for simple U-turn movements to the more complex unsignalized and signalized rural and urban intersections that may include traffic from minor crossroads and streets or major roadways and commercial entrances. The design of median openings and median end treatments is based on traffic volumes, operating speeds, predominant types of turning vehicles and median width. Crossing and turning traffic must operate in conjunction with the through traffic on a divided highway. This requirement makes it necessary to know the volume and composition of all movements occurring simultaneously during the design hour. The discussion in this section is primarily directed to rural, unsignalized, divided roadways.

Figure 7-16
Graphical Guide for Left-Turn Lane for
60 mph [100 km/h] Operating Speed



the trailing edge for 20 ft [6.0 m].The bullet nose design can be designed to conform to the traffic movements permitted in the intersection. A wide median normally would have a portion of the nose flattened to be parallel to the median opening centerline and, depending upon the channelization design, a semicircular design may be more appropriate.

Figure 7-20
Preferred Median End Shapes
Based on Median Width

Controlling Median Width	Median End Shape
4 ft [1.2 m] or less	*Semicircular
4 to 66 ft [1.2 to 20 m]	Bullet Nose or Modified Bullet Nose
Over 66 ft [20 m]	Treated as separate intersection

*At locations with left turning cross road traffic, use a controlling radius of at least 40 ft [12 m].

7.6.3 LENGTHS OF MEDIAN OPENINGS

For any intersection on a divided highway, the length of the median opening should be as great as the width of the crossroad roadway pavement plus shoulders. The width and type of crossroad combined with the median width and selected control radius affect the median opening. The design should minimize any unsafe tracking encroachment into oncoming traffic from crossroads. AASHTO recommends that in no case should the opening be less than 40 ft [12 m] for a 90-degree intersection or less than the width of the crossroad pavement plus shoulders or plus 8 ft [2.4 m] for a crossroad without shoulders. Where the crossroad is a divided highway, the length of the opening should be at least equal to the width of the crossroad roadways, median and shoulders or 8 ft [2.4 m] if there are no shoulders.

Median openings are a function of median width and the selected control radius. Use of a 40 ft [12 m] minimum opening without regard to these two items should only be considered for minor, rural, unsignalized crossroads. Median openings of 50 to 64 ft [15 to 20 m] are more typical. The 40 ft [12 m] minimum length of opening does not apply to openings for U-turns where, depending upon the predominant vehicle, larger openings may be needed to ensure the vehicle can turn into the desired lane. As median widths become greater than 50 ft [15 m] the increased pavement area may create confusion as to proper vehicle paths and movements. These wider openings may need additional traffic control devices. ASSHTO recommends avoiding using median openings greater than 80 ft [25 m].

7.6.4 DESIRABLE MEDIAN OPENING DESIGNS FOR LEFT TURNS

Median openings that enable vehicles to turn on minimum paths, and at very low speeds, are adequate for intersections where traffic for the most part proceeds straight through the intersection. Where through-traffic volumes and speeds are high and left-turning movements are important, undue interference with through traffic should be avoided by providing median openings that permit turns without encroachment on adjacent lanes. This arrangement would enable turns to be made at speeds above those for the minimum vehicle paths and provide space for vehicle protection while turning or stopping.

For median openings having control radii greater than the minimum for the selected design vehicle, see the Green Book, pages ~~694 to 700~~ 690 to 696. The three radii R, R1 and R2 control bullet-nose end designs. Figure 7-21 shows the layout. Radius R is the control radius for the sharpest portion of the turn. R_1 defines the turnoff curve at the median edge. R_2 is the radius of the tip.

When a sufficiently large R_1 is used, an acceptable turning speed for vehicles leaving the major road is assured, and a sizable area inside the inner edge of the through-traffic lane between points 1 and 2 on Figure 7-21 may be

- Simple, clear signing.

10.11.5 BUSES

As previously discussed, where buses load and unload within the parking lot, traffic flow should be such that buses and automobiles do not conflict. Buses require adequate room for decelerating, proper turning radii, maneuvering into and out of an adequate loading area, and returning to the mainline traffic flow. Refer to Chapter 7 of this manual and the Green Book for design criteria such as required turning radii for bus operation.

10.11.6 KISS-AND-RIDE FACILITIES

A kiss-and-ride facility is located so that transit or commuter passengers can easily and safely access the terminal or loading zone with minimum conflicts with other vehicles; see Figure 10-19. To accomplish this, circulation in the kiss-and-ride facility should be one-way and flank the terminal or loading area. Parking should be at 45 degrees to allow for pull through and face the terminal or loading zone. To operate properly it is usually necessary to enforce kiss-and-ride restrictions.

10.11.7 PEDESTRIANS

Two pedestrian movements must be provided for park-and-ride lots that serve bus routes: a direct and safe approach from adjacent streets to the bus stop and pedestrian access from the parking area. Pedestrian circulation in parking lots is provided by aisles and crosswalks or, in larger lots, by walkways. The pedestrian path from any parking stall to the bus stop should be as direct as possible.

10.11.8 BICYCLES AND MOTORCYCLES

It is important to provide adequate bicycle storage racks at park-and-ride lots where large concentrations of bicycle traffic are expected. Similarly, a special parking area for motorcycles will improve utilization of space. Motorcycle storage should be on Portland cement concrete to prevent stands from sinking into hot asphalt pavement. Provisions for locking both bicycles and motorcycles to prevent theft are needed. This includes bicycle racks and lockers.

10.11.9 DISABLED

At lots for transfer to buses, the design should consider provisions for safe and convenient access for the elderly and disabled. Design requirements and provisions for disabled parking shall be in conformance with the *Americans with Disabilities Act of 1990* and the State of Delaware *Architectural Accessibility Standards.*

Facilities for the disabled should also be designed in accordance with the following:

- Disabled persons should reach the bus loading zone without crossing any access roads;
- Loading areas must meet ADA space requirements;
- Disabled persons must never be forced to travel behind parked cars; and
- Suitable ramps must be provided.

10.11.10 PARKING DIMENSIONS AND LOT LAYOUT

Parking areas and roadway layout for park-and-rides can be designed in much the same manner as other parking facilities. Facilities that interact with transit, where DTC does not specify a bus size, should use a 40 foot [12 m] transit vehicle as the design vehicle. Standard dimensions for car parking stalls are shown in Figure 10-20.

For design purposes only two size stalls should be considered—standard and intermediate. If compact car parking is to be provided, it should be in a prime location or the driver will select more convenient

paragraph format - justified

available stalls. Combining several different types of stalls also creates an undesirable and more complicated signing layout. If there is adequate room, limiting the design to accommodate the standard stall size, 9 by 18.5 ft [2.7 by 5.6 m] is preferred. The minimum bus-parking stall should be 13 by 32 ft [4 by 15.25 m].

Figure 10-18
Typical Car Parking Dimensions

Size	Stall Width ft [m]	Stall Length ft [m]	Aisle Width ft [m]
Standard	8.5-9.5 [2.6-2.9]	18-20 [5.5-6.0]	24-26 [7.5-8.0]
Inter-mediate	8.0-9.0 [2.4-2.7]	16-18 [4.9-5.5]	22-24 [6.7-7.3]
Compact	7.5-8.5 [2.3-2.6]	15-17 [4.6-5.2]	20-22 [6.0-6.7]

Substandard stall and aisle widths are false economy. Although they permit marking more stalls in a given area, vehicles tend to encroach upon adjacent stalls such that one or more spaces are unavailable for use. The end result is no gain in actual space usage.

Vehicles and other objects should be excluded from corners or parking spots where it is necessary to provide adequate intersection sight distances. Islands at the end of rows should be considered when laying out the lot. For pedestrian safety, the parking stalls and aisles should be parallel to the direction of the desired pedestrian flow. For efficient land area use, a row of parking on each side of the aisle is preferred.

Aisle width is a function of the parking angle and stall width. One-way aisles are generally used with angle parking and two-way circulation is generally used with 90-degree parking. Aisle lengths should not exceed 400 ft [120 m] to limit pedestrian walking distance. One-way aisles should favor counterclockwise circulation with head-in parking only. Due to lower vehicle undercarriage heights, a 6 in [150 mm] curb is recommended where head-in parking is being considered. Sidewalks should be a minimum of 5 ft [1.5 m] and loading areas should be 12 ft [3.6 m] wide. Pedestrian paths from parking spaces to loading areas should be as direct as possible. All sidewalks and curb areas are to be in conformance with ADA standards.

Figures 10-21 and 10-22 provide data for planning stall layouts for standard stall sizes of 9 by 18.5 ft [2.0 by 5.6 m]. Layouts for intermediate and compact stall sizes are available in the AASHTO ~~guide~~. Green Book

The parking area should be sloped to provide positive drainage. Ponding water in a lot is undesirable for both vehicle and pedestrian movement, particularly where freezing may create icy spots. The recommended minimum grade is 1%, the desirable is 2%, and the maximum is 5%. The designer should provide adequate access and areas for snow removal and/or storage. The pavement selection needs to recognize that a variety of traffic loads, particularly when transit is expected, may be applied to the lot and the pavement type and strength designed accordingly.

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
800 BAY ROAD
P.O. BOX 778
DOVER, DELAWARE 19903

CAROLANN WICKS, P.E.
SECRETARY

May 21, 2007

Dear Road Design Manual Holders:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual - April 2007 Revision

This is the formal issuance of the April 2007 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective June 1, 2007. You can download this revision from DelDOT's web site: http://deldot.gov/static/pubs forms/manuals/road design/index.shtml. The following changes have been made:

- Section 3.1.3 and its subsections were modified. The content of the text of the remainder of the chapter has not been changed.
- Figure 3-2 was added. The remaining figures of the chapter were rearranged, although their content did not change.

Please replace the pages in your manual with the appropriate sheets. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely yours,

Dennis M. O'Shea

Dennis M. O'Shea
Assistant Director - Design

DMO:los
cc: Robert Taylor, Chief Engineer
Michael Simmons, Assistant Director, Project Development, South Region
Drew Boyce, Assistant Director, Project Development, North Region
Kevin T. Canning, Quality Engineer
Linda M. Osiecki, Program Manager, Quality

DelDOT

tion agencies and continues to update the information to reflect new findings and the current state of knowledge.

This manual cannot attempt to cover the total scope of important published information related to highway design policies.

3.1.2 APPLICATION OF STANDARDS

Depending upon the design element being evaluated, AASHTO design criteria are expressed as design values, minimum values or as ranges of values for particular elements and conditions. Design values are empirically derived; any value lower may be unsafe and any value higher may be unnecessary and uneconomical. Minimum values should, depending upon the element being evaluated, not be lowered either because it will create an unsafe condition or, in some cases, will not physically work. Some design elements lend themselves to minimum and desirable values. Although a portion of a project may require the application of minimum values, other sections will allow the designer to use standards higher than the minimum. In evaluating a project for application of standards, user expectation is important. Consistency in application of standards is an important safety feature. Large variances in standards may create unacceptable driver behavior. AASHTO standards are developed to allow agencies to select those that best meet their needs and practices.

Design values presented in this manual are in metric and US Customary units and were developed independently within each system. The relationship between the metric and US Customary values is neither an exact (soft) conversion nor a completely rationalized (hard) conversion. The metric values are those that would have been used had the manual been presented exclusively in metric units; US Customary values are those that would had been used if this manual has been presented exclusively in US Customary units. Therefore, the user is advised to work completely in one system and not attempt to convert directly between the two. Figure 3-1 shows the equivalent US Customary and Metric units for the commonly used design speeds.

Figure 3-1
Corresponding Design Speeds in US Customary and Metric Units

US Customary	Metric
Design speed (mph)	**Corresponding Design speed [km/h]**
15	20
20	30
25	40
30	50
40	60
45	70
50	80
55	90
60	100
70	110

The standards adopted by the Delaware DOT, described herein, adhere to the basic framework of AASHTO design policies. But the specific standards contained herein reflect judgments by the Department as to their proven operational success in Delaware and application to the predominant geographical conditions.

Most standards are related to a facility's functional classification with the interstate system having the highest and local streets having the lowest. For instance, 10 ft [3 m] travel lanes may be acceptable on local streets but completely unacceptable on facilities with high volumes, higher operating speeds, a more diverse mix of vehicle types, and a goal of maintaining or improving system capacity.

3.1.3 DEPARTURE FROM STANDARDS

Most projects are expected to meet at least the minimum standard design values established for the project level. Occasionally there may be conditions that warrant consideration of a lower value. For projects on higher functional classification roadways, it is more critical to strive to meet or exceed all of the applicable standards, primarily because the motorist expects a higher standard and will drive the facility accordingly.

The need for exceptions to the standards must be identified early in the project development phase in order that approvals or denials will not delay completion of the design or require extensive redesign. However, the need to evaluate a lower design value may arise at any time during the design process and needs to be addressed expeditiously. Thorough documentation is essential. The need for exceptions should not be viewed as normal or routine. The forms in this chapter shall be used to document decisions on design criteria and as a basis for developing and documenting requests for exceptions. The primary focus of the request should be highway safety. The design and proposed mitigation should be the best practical alternative that considers whether or not other controlling design elements will be adversely affected.

The major controlling design elements that need to be evaluated for conformity to established standards include:

- Design speed;
- Through lane and auxiliary lane widths;
- Shoulder widths;
- Stopping sight distance on vertical and horizontal curves;
- Horizontal alignment (radius of curve);
- Vertical alignment;
- Minimum and maximum grades;
- Cross slopes;
- Superelevation rate;
- Horizontal clearance;
- Vertical clearance;
- Bridge width; and
- Structural capacity.

An exception for design speed should not be sought as this element establishes most if not all of the other parameters to be met. As discussed in this chapter and in several other sections in this manual, design speed is an achievable speed selected by the designer based on the various factors the designer must consider. Design elements that can not be met within that selected design speed should be supported by seeking a design exception in accordance with the following guidelines.

All variances from standards need to be documented in the project files, and, in many cases, require approval from the Chief Engineer and FHWA. The level of documentation depends on the project scope, functional classification and other factors. Depending upon the significance of the request, the support information may include some or all of the following:

- Existing roadway characteristics,
- Required and proposed design criteria;
- Cross section or geometric figures comparing the existing and proposed conditions;
- Supporting calculations and cost analysis;
- Analysis of accident records;
- A discussion on the compatibility with adjacent sections;
- Effect on right-of-way;
- Environmental constraints;
- Any proposed mitigation considered to help offset the variance from the design standard;
- Mitigation costs;
- Public support or opposition; and
- Other pertinent factors

3.1.3.1 NEW CONSTRUCTION AND RECONSTRUCTION PROJECTS

New construction and reconstruction projects on the interstate system and the NHS are expected to be in conformance with the appropriate standards and exceptions should be rare. All projects on the NHS (except preventive maintenance) shall conform to the standards in the current edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book). All projects on the interstate system (except preventive maintenance) shall also meet the design criteria contained in the current edition of *A Policy on Design Standards—Interstate System.* Projects having full federal oversight by FHWA are established in the current *Delaware*

Transportation/Federal Highway Administration Stewardship Agreement to Implement the Flexibility of Provisions of 23 United States Code Part 106. FHWA must approve all design exceptions for projects with full federal oversight.

New construction and reconstruction projects require a formal design exception as described in "Required Design Exception Documentation" (Figure 3-2) to justify the rationale for departure from the established design standards. The Chief Engineer must approve design exceptions for all new construction and reconstruction projects. "Design Exception Request" (Figure 3-3) is a guide format for developing a design exception request. It is important that the designer complete the "Design Control Checklist" (Figure 3-4) and the "Design Criteria Form" (Figure 3-5) for all new construction and reconstruction projects.

Figure 3-2
Required Design Exception Documentation

Project Type	Design Exception Request (Fig. 3-3)	Design Control Checklist (Fig. 3-4)	Design Criteria Form (Fig. 3-5)	Asst. Director, Design, and Chief Engineer Approval	FHWA Approval
New Construction or Reconstruction – Full Federal Oversight	X	X	X	X	X
New Construction or Reconstruction – State Administered	X	X	X	X	
Intermediate Level		X	X		
Preventive Maintenance		X			
Miscellaneous Improvement		X			

3.1.3.2 INTERMEDIATE PROJECTS

The scope of intermediate projects is improvements being retrofitted into the existing infrastructure. At times, it may be necessary to deviate from the design standards. It is important that the designer complete the "Design Control Checklist" (Figure 3-4) and the "Design Criteria Form" (Figure 3-5) for all intermediate projects. If design standards cannot be met due to site conditions, then it shall be noted on the Design Criteria Form. In addition, there must be a memo to the project file explaining the justification for the variances in the design standards signed by the Project Engineer, the Group Engineer and the Assistant Director. The level of documentation can vary depending on the project intent, environmental concerns and other issues. In most cases it is not necessary to provide the same level of documentation as provided in a formal design exception request because the project's scope has already been established. The following are examples of intermediate projects:

- Safety improvement projects
- Addition of channelizing islands with no reduction in existing lane or shoulder width
- Intersection improvements with no reduction in existing lane or shoulder width
- Minor lane or shoulder widening
- Traffic calming features, e.g. speed humps, chicanes, midblock median islands, chokers, or narrowed lanes. (Consult with FHWA for projects on NHS Routes.)
- In-kind superstructure replacement

3.1.3.3 PREVENTIVE MAINTENANCE PROJECTS

Preventive maintenance includes rehabilitation or restoration of specific elements of a highway facility when such activities are a cost- effective means of extending the pavement or bridge life and shall not degrade any existing or geometric aspects of the facility. The majority of the work for these projects is between existing curb lines or outer edges of existing shoulders. These types of projects are not required to provide any

documentation for design standards that cannot be met. A general list of preventative maintenance items is below:

- Pavement milling
- Pavement resurfacing with no reduction in lane widths
- Pavement repair (e.g. sawing, sealing, pothole patching)
- Rehabilitation of existing structures
  - Superstructure/substructure rehabilitation
  - Deck rehabilitation
  - Joint replacement or repair
  - Bearing replacement
  - Seismic retrofit
  - Structure painting
  - Scour countermeasures
  - Retaining structure rehabilitation
  - Culvert lining

3.1.3.4 MISCELLANEOUS IMPROVEMENT PROJECTS

These projects are typically beyond the existing edge of pavement and are intended to improve safety and aesthetics. They may also include improvements within the roadway intended to improve safety which do not degrade the existing highway geometrics. These projects are not required to provide any documentation for design standards that cannot be met. A general list of miscellaneous improvement project work items is below:

- New or replaced curb and/or sidewalk
- Modifying sidewalk to comply with ADA requirements
- Roadside safety enhancements
  - New guardrail installation
  - Guardrail repair/replacement
  - New impact attenuators
  - Impact attenuator repair or replacement
  - Upgrading bridge rails
  - Removal of obstructions
- Median barrier replacement
- Drainage improvements
- Signing
- Large ground-mounted signs
- New sign structures (sign bridge, cantilever and bridge-mounted, provided they meet clearance requirements)
- Existing sign structure rehabilitation
- Striping with no additional lanes nor reduction in existing lane width
- New or replaced raised pavement markers
- New or upgraded signals
- ITS (e.g. fiber optic cable, message signs, cameras, emergency call boxes)
- New or upgraded lighting systems
- Fencing, provided existing stopping sight distance is not degraded
- Glare screens, provided existing stopping sight distance is not degraded
- Repair of structural components resulting from traffic impact
- Advanced utility relocation projects
- Landscape improvements

Figure 3-3
Design Exception Request

State Project No._____Federal-Aid Project No._____

Date: _____**Oversight Project: Yes _____No____**

Design Exception Abstract: (Provide a short summary detailing the nature of the exception, reasons for the request, etc.)

Note:
For all NHS projects, the thirteen controlling criteria to be met are design speed; through lane and auxiliary lane width; shoulder width; bridge width, structural capacity, horizontal alignment; vertical alignment; grades; stopping sight distance, cross-slope; superelevation; horizontal clearance; and vertical clearance.

RECOMMENDATION:

The purpose of this project is to----------.

The most effective method of addressing this is----------.

Based upon the conditions presented, it is recommended that a design exception be approved for the controlling substandard design element as justified.

Recommended By: _____
Squad Manager

Recommended By: _____
Group Engineer

Recommended By: _____
Assistant Director-Transportation Solutions

Recommended By: _____
Assistant Director-Design

Approved By: _____ **Date**:_____
Chief Engineer

Approved By: _____ **Date**:_____
Federal Highway Administration (NHS oversight projects only)

Enclosures: (Include design criteria, figures, calculations, etc. to document request.)

Figure 3-4
Design Control Checklist

PROJECT DATA

Squad Leader/Project Manager: _____

Project Title: _____

Contract No.: _____

Federal Aid Project No: _____

Project Limits: _____

Type of Construction: _____

Project Scope and Initial Estimate: _____

_____

_____

_____

DESIGN DATA

Functional Classification:_____

Current AADT (Year)_____

Projected AADT (Year):_____

Projected DHV (Year):_____

% Trucks_____

Directional Distribution (%):_____

Design Speed _____

Design Vehicle_____

Design Level of Service _____

Clear Zone_____

Recommended By: _____

Squad Manager

Recommended By: _____

Group Engineer

Recommended By: _____

Assistant Director-Transportation Solutions

Approved By: _____

Director-Transportation Solutions

Figure 3-5
Design Criteria Form

Design Criteria		
Design Factor	**As per Road Design Manual**	**Provided**
Design Speed*		
Width of Through Lanes*		
Width of Auxiliary Lanes*		
Width of Outside Shoulder*		
Width of Inside Shoulder*		
Cross Slope*		
Width of Median		
Stopping Sight Distance*		
Minimum Horizontal Curve Radius*		
Minimum K (Crest)*		
Minimum K (Sag)*		
Maximum % Grade*		
Maximum front slope (Unprotected Section)		
Maximum back slope		
Barrier Offset		
Superelevation Rate (%)*		
Bridge Width*		
Vertical Clearance*		
Structural Capacity*		
Horizontal Clearance *		
Width of clear zone		

General Notes:
- Use this form primarily for new construction or reconstruction projects.

* The Chief Engineer must approve design criteria deviating from the requirements of the Road Design Manual through the use Figure 3-3 “Design Exception Request.”

Recommended By: _____
Project Manager

Recommended By: _____
Group Engineer

Approved By: _____
Assistant Director-Transportation Solutions

3.1.4 DETERMINATION OF STANDARDS

Figure 3-6 graphically defines the nomenclature used when describing the various elements that establish a roadway's cross section. The dimensions and geometrics needed to design this roadway section are known as a project's design standards. The basic information needed before beginning the process of establishing a project's design standards is:

- The ***functional classification*** of the road section to be improved. A roadway's classification is shown on DelDOT's current functional classification map but should be verified with the Division of Planning. A part of the initial field review and scoping meeting should be to verify that the area and roadway section being considered are truly representative of the designated classification.

- The ***scope of work*** proposed for the project under consideration is in the Project Initiation Form, project development documentation and other supporting data furnished to the designer.

- The ***traffic data*** for the road section is obtained from the Division of Planning and includes current traffic, projected traffic, percent trucks, accident history, etc.

With this information, the designer can proceed with the process of selecting the design speed.

3.2 STANDARDS BASED ON DESIGN SPEED

3.2.1 SELECTION OF DESIGN SPEED

The design speed establishes basic criteria for certain design elements. Two design standard considerations are related directly to the design speed:

- Curvature and superelevation, and
- Required sight distances.

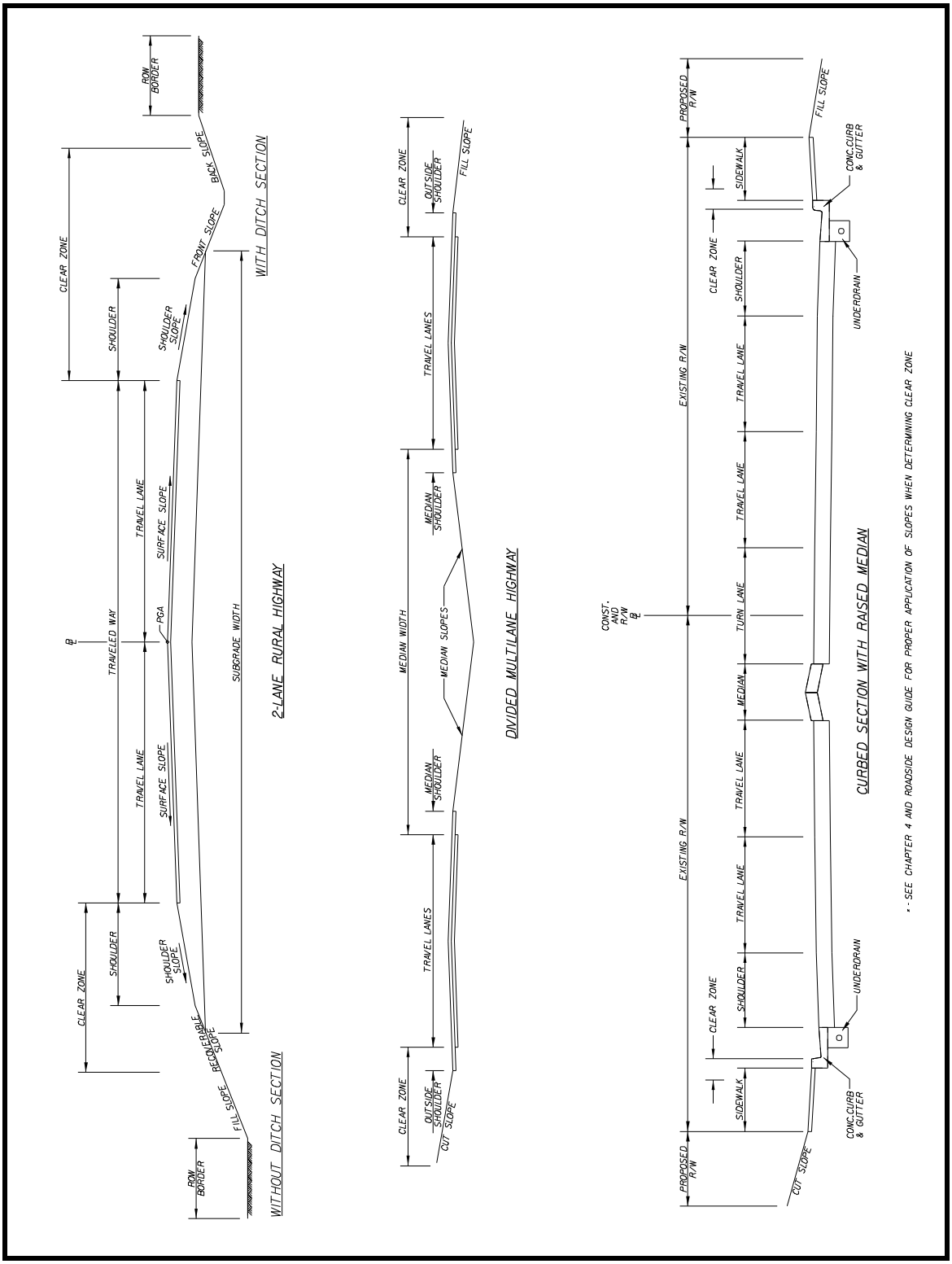
The designer's goal is to provide at least the minimum values, and preferably larger values, for these standards, regardless of traffic volumes, functional classification or any other consideration. These design elements are very closely related to traffic safety and should not be compromised.

A first step in determining the appropriate design standards is to establish a reasonable and realistic design speed. Since the majority of design controls are related to the design speed, this decision needs to be based on more factors than a roadway's functional classification and traffic volume.

The design speed selected should accommodate a high percentage of drivers, including the reasonable and prudent driver. Other considerations include topography, anticipated operating speeds, driver expectations, volume and mix of vehicles, the volume and type of non-vehicular traffic, driver familiarity, level of congestion reasonably acceptable to the motorists, and community values.

Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Figure 3-6
Typical Section Nomenclature



Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page 384, Exhibit 5-5;
- Collector Roads and Streets (Rural) — page 425, Exhibit 6-5;
- Collector Roads and Streets (Urban) — page 433;
- Arterials (Rural) — page 448, Exhibit 7-3;
- Divided Arterials (Rural) — page 455;
- Urban Arterials — page 472;
- Freeways — page 504.

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-5, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the

maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smoother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages 167 to 174, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

Vertical curvature, horizontal curvature, roadside obstructions, or any combination of these elements can restrict sight distance. Procedures for checking available sight distances are described in the Green Book, pages 127-131.

3.2.4 PASSING SIGHT DISTANCE

Consideration of passing sight distance is limited to two-lane, two-way highways on which vehicles frequently overtake slower-moving vehicles and the passing operation must be accomplished on a lane used by opposing traffic.

Passing sight distance for design is determined on the basis of the length needed to accomplish the passing maneuver. Derivation of the required distance is described in the Green Book, pages 118-126. AASHTO recommends

that, "In designing a highway these distances should be exceeded as much as practical ..."

These distances for design should not be confused with other distances used as warrants for placing no-passing pavement markings on completed highways. Values shown in the MUTCD are substantially less than the design distances and are derived from traffic operation control needs based on assumptions different from those for design.

Because of vertical and horizontal sight limitations, nearly all two-lane highways have some no-passing restrictions. In rolling terrain, the proportionate amount of no-passing sections usually becomes greater. Normally it is impracticable to attempt to provide passing sight distance throughout the entire length of a project. The principal design consideration is to try to provide adequate passing opportunities as frequently as possible.

There are no fixed values for the frequency of passing sections. Experience shows that highway capacity is measurably reduced when a significant percentage of a section of highway is restricted to sight distances less than 1500 ft [500 m]. Highways with high traffic volumes will require a higher proportion of passing opportunities than those with low traffic volumes. Where an analysis shows that a lack of passing sight distances has reduced capacity to near or below the expected traffic volumes, it is necessary to consider adjustments in the alignment and grade, or to provide additional lanes.

3.3 STANDARDS BASED ON TRAFFIC VOLUMES

Standards not directly related to design speed are influenced primarily by traffic volumes. Tables for these standards shown in the tables at the end of this chapter reflect variations by traffic volume ranges.

3.3.1 NUMBER OF LANES

The number of lanes required for any highway is directly related to the facility's traffic volume and desired level of service. But there are no simple, fixed criteria for these relationships. The recommended number of lanes is normally obtained through the project development process.

The *Highway Capacity Manual* gives two very general guidelines for determining the need for additional lanes. These numbers are based on long sections of roadway with uninterrupted traffic flow having the highest standards for design controls (horizontal and vertical geometrics and cross-sectional elements), ideal weather conditions, daylight, etc.

1. Under ideal conditions, a two-lane rural highway can accommodate about 900 passenger vehicles (two-way) per hour with a reasonably high level of service if there are adequate passing opportunities and no long, steep grades. Considerably more vehicles can be accommodated if motorists are willing to accept a lower level of service, a greater degree of congestion and lower operating speeds.
2. Under ideal conditions, a multi-lane highway can accommodate about 900 passenger vehicles per lane per hour. Again, considerably more vehicles can be accommodated, if lower levels of service can be tolerated.

Most roadways do not meet the ideal conditions. The HCM defines the ideal roadway as follows: (1) meets or exceeds design speed; (2) has 12 ft [3.6 m] travel lane widths; (3) has shoulder widths greater than 6 ft [1.8 m]; (4) has minimal no passing zones; (5) carries predominantly passenger cars; (6) has evenly distributed traffic flow; (7) has minimum crossing and entering traffic interference; and (8) has level terrain.

Although all these elements are rarely available within a project's limit, capacity is

usually not a problem on most of the rural roadways in Delaware. Exceptions are some of the principal arterial routes, particularly in the vicinity of urban areas.

Most proposed improvements will be in traffic volume ranges where the existing number of lanes will be adequate without the need for detailed study. However, capacity may be influenced where the traffic volume exceeds about 900 DHV or where there are unusual conditions of alignment, grade or signalization.

Designers working with Traffic and the Division of Planning should identify the need for additional through lanes or, if applicable, auxiliary climbing lanes. Such a change after project initiation is a major decision affecting all aspects of a project from cost to environmental and social impacts.

3.3.2 SURFACED LANE WIDTHS

The traveled way designated for vehicle operation (excluding shoulders) normally consists of two or more surfaced traffic lanes. The impact of providing adequate lane widths is wide ranging and includes maintaining and/or enhancing driver safety, driver comfort, the level of service, capacity, and the frequency and extent of shoulder and pavement surface maintenance.

For all new construction and reconstruction on arterial and collector roads, the desirable surfaced travel lane width is 12 ft [3.6 m]. If the scope of work is limited, speeds are low, truck volumes are light or there are no defined safety problems, surfaced lane widths of 11 ft [3.3 m] may be acceptable, particularly in urbanized areas with restricted right-of-way and increased pedestrian activity. However, for urban arterials with higher speeds, predominantly free-flowing conditions, and higher traffic volumes, surfaced lane widths of 12 ft [3.6 m] are desirable. For local roads and streets, surfaced traffic lanes normally should be 11 feet [3.3 m] wide but AASHTO allows lane widths of 9 [2.7 m] or 10 ft [3.0 m] where there is restricted or low truck use, low traffic volumes and low operating speeds. See Section 3.2.1 for information on selecting lane and shoulder widths.

For pavements on new construction or major reconstruction projects with existing or projected high concentrations of truck traffic, a wider pavement provides more edge strength and has been found to be structurally better for heavy loads. Consideration should be given to widening the pavement an additional 2 ft [0.6 m] under these circumstances. The lanes should be striped for 12 ft [3.6 m] lanes to keep trucks away from the edge of the pavement. The extra width can be considered part of the shoulder. If the mainline and shoulders are constructed of Portland cement concrete and the shoulders are structurally tied to the mainline, this additional width is not normally necessary. For divided highways, the widening should be adjacent to the outside shoulder; on two-lane roadways the widening should be equally divided on each side.

3.3.3 SHOULDER WIDTH

The total shoulder width is the distance from the edge of the traffic lane to the intersection of the shoulder slope with the front slope, or to the face of curb. In sections without curbs there are two terms used to describe the shoulder area. The “graded” width of shoulder is that measured from the edge of the traveled way to the intersection of the shoulder slope and the front slope. The “useable” width of shoulder is the actual width that can be used when a driver makes a stop.

Having a sharp break at the point of intersection of the edge of the graded shoulder and the front slope is not a good practice. Instead a rounding of 4 to 6 ft [1.2 to1.8 m] with a front slope 4:1 or flatter is the best practice. This rounding improves the general safety of the roadside by reducing the likelihood of encroachment, thus giving the errant driver more chance to regain control. Other considerations

are that rounding may reduce rollovers and the possibility that the vehicle may become airborne. A portion of the rounding (usually one half) can be considered part of the "useable" width. Where front slopes are steeper than 4:1, the rounding should occur outside of the useable shoulder width.

Well-designed and maintained shoulders are necessary on rural highways with any appreciable traffic volumes. Shoulders and their widths are an important consideration in establishing a project's design standards. The benefits of including a shoulder include:

- Providing a refuge when a driver makes an emergency or parking stop.
- Providing lateral recovery areas for vehicles that inadvertently leave the traffic lane.
- Providing improved sight distance in cut areas.
- Providing areas for maintenance operations, including snow removal and storage.
- Providing for alternative modes of travel by pedestrians, bicyclist, joggers, transit operations, etc.
- Structurally improving the service life of the pavement by increasing the stability of the roadway's base and surfacing materials at the edge of the through traffic lane.
- Providing the opportunity for improved subgrade drainage designs.

Section 3.2.1 discusses the selection of shoulder widths. Normally shoulder widths of 10 ft [3.0 m] are used on new construction projects for arterial highways with relatively high traffic volumes. Where truck traffic exceeds 250 DHV it is desirable to have a paved shoulder width of 12 ft [3.6 m]. AASHTO allows narrower shoulder widths on most roadways with lower traffic volumes. However, wider shoulders widths should be provided on these projects when practical. Where bicyclists and pedestrians are to be accommodated on the shoulders, a minimum useable shoulder width, clear of any rumble strips, of 4 ft [1.2 m] should be used. On highways with three or more lanes in each direction a median shoulder width of 10 ft [3.0 m] is desirable. This provides a refuge area for disabled vehicles without affecting roadway capacity and flow.

3.3.4 SURFACED SHOULDER WIDTH

The surfaced shoulder width is that part constructed to provide better all-weather load support than is afforded by natural soils or stabilized materials. The paved portion of the shoulder also protects the edge of the traffic lane pavement from deterioration and raveling. More discussion on shoulder surfacing is in Chapter Nine - Pavement Selection. Normally the shoulder's structural design, including surface material, is recommended by the Materials and Research Section.

3.3.5 SIDE SLOPES

Various cross section slopes are identified in Figure 3-6. Four of these slopes are described below.

- **Front Slope.** The slope extending outward and downward from the shoulder to the ditch line.
- **Back Slope**. The slope extending upward and outward from the ditch line to intersect the natural ground.
- **Fill Slope**. The slope extending outward and downward from the shoulder to intersect with the natural ground; it may include a ditch section.
- **Cut Slope**. The slope extending outward and upward from the shoulder, intersecting the ditch slope and then extending upward from the ditch back slope to natural ground.

It is generally desirable that these slopes be 6:1 or flatter. Often, from a practical standpoint, they must be steeper. There is a distinct relationship between the steepness of side

slopes, operating speeds and the desirable widths of clear zones. Chapter Four discusses the relationship of these slopes to establishing a roadway's clear zone. General criteria for side slopes are presented in Figure 4-4, in terms of both desirable slopes and maximum slopes. The desirable slopes should be provided wherever feasible.

3.3.6 HORIZONTAL CLEARANCE AND CLEAR ZONE

3.3.6.1 HORIZONTAL CLEARANCE

Horizontal clearance is the lateral distance from edge of traveled way to a roadside feature or object for a roadway with barrier curb. Roadways having curbed sections should be provided with a minimum horizontal clearance of 1.5 ft [0.5 m] beyond the face of curb, with wider offsets (if possible to the full clear zone width) provided where practical since most types of curbs provide little help in redirecting an errant vehicle. Please see the Green Book for more information regarding horizontal clearance and AASHTO's Roadside Design Guide for more information regarding the clear zone width. If the minimum horizontal clearance cannot be provided in curbed areas, then a design exception is required.

3.3.6.2 CLEAR ZONE

The clear zone is defined in AASHTO's Roadside Design Guide as "the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry." This border area includes any shoulders or auxiliary lanes. Adequate lateral clearance between the edges of traffic lanes and roadside obstructions has been shown to be a very important safety factor. Vehicles leaving the roadway should have a reasonable opportunity to recover control and return to the roadway without overturning or colliding with roadside obstacles such as trees, poles, headwalls or other large objects. The combination of a relatively flat slope and an obstacle-free roadside within the prescribed clear zone helps this situation.

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the safest available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-4), the Design Criteria Form (Figure 3-5) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there are steep grades accompanied by considerable no-passing distances.

More detailed guidelines and criteria for the design of grades, including critical lengths of grades and minimum and maximum grades are presented in Chapter Five - Alignment and Superelevation and the Green Book, pages 231-250. The maximum grades should be used infrequently, only as dictated by severe terrain conditions. When it is necessary to use maximum grades, the designer should check other design criteria and roadside features that may be improved to minimize the impact of using the higher design grade.

3.3.8 BRIDGES

The designer should coordinate with the Bridge Design Section when determining vertical clearances. A minimum vertical clearance for roads over interstate, U.S. and state routes is 16.5 ft [5 m]. Pedestrian bridges and overhead sign structures must have an extra 1 foot [0.3 m] of clearance, a total of 17.5 ft [5.3 m]. These clearances allow for a 4 in. [100 mm] future resurfacing.

3.3.9 MEDIANS

Geometric criteria for medians on multilane divided highways are discussed in Chapter Four.

tion agencies and continues to update the information to reflect new findings and the current state of knowledge.

This manual cannot attempt to cover the total scope of important published information related to highway design policies.

3.1.2 APPLICATION OF STANDARDS

Depending upon the design element being evaluated, AASHTO design criteria are expressed as design values, minimum values or as ranges of values for particular elements and conditions. Design values are empirically derived; any value lower may be unsafe and any value higher may be unnecessary and uneconomical. Minimum values should, depending upon the element being evaluated, not be lowered either because it will create an unsafe condition or, in some cases, will not physically work. Some design elements lend themselves to minimum and desirable values. Although a portion of a project may require the application of minimum values, other sections will allow the designer to use standards higher than the minimum. In evaluating a project for application of standards, user expectation is important. Consistency in application of standards is an important safety feature. Large variances in standards may create unacceptable driver behavior. AASHTO standards are developed to allow agencies to select those that best meet their needs and practices.

Design values presented in this manual are in metric and US Customary units and were developed independently within each system. The relationship between the metric and US Customary values is neither an exact (soft) conversion nor a completely rationalized (hard) conversion. The metric values are those that would have been used had the manual been presented exclusively in metric units; US Customary values are those that would had been used if this manual has been presented exclusively in US Customary units. Therefore, the user is advised to work completely in one system and not attempt to convert directly between the two. Figure 3-1 shows the equivalent US Customary and Metric units for the commonly used design speeds.

Figure 3-1
Corresponding Design Speeds in US Customary and Metric Units

US Customary	Metric
Design speed (mph)	**Corresponding Design speed [km/h]**
15	20
20	30
25	40
30	50
40	60
45	70
50	80
55	90
60	100
70	110

The standards adopted by the Delaware DOT, described herein, adhere to the basic framework of AASHTO design policies. But the specific standards contained herein reflect judgments by the Department as to their proven operational success in Delaware and application to the predominant geographical conditions.

Most standards are related to a facility's functional classification with the interstate system having the highest and local streets having the lowest. For instance, 10 ft [3 m] travel lanes may be acceptable on local streets but completely unacceptable on facilities with high volumes, higher operating speeds, a more diverse mix of vehicle types, and a goal of maintaining or improving system capacity.

3.1.3 DEPARTURE FROM STANDARDS

Most projects are expected to meet at least the minimum standard design values established for the project level. Occasionally there may be conditions that warrant consideration of a lower value. For projects on higher functional classification roadways, it is more critical to strive to meet or exceed all of the applicable standards, primarily because the motorist expects a higher standard and will drive the facility accordingly.

The need for exceptions to the standards must be identified early in the project development phase in order that approvals or denials will not delay completion of the design or require extensive redesign. However, the need to evaluate a lower design value may arise at any time during the design process and needs to be addressed expeditiously. Thorough documentation is essential. The need for exceptions should not be viewed as normal or routine. The forms in this chapter shall be used to document decisions on design criteria and as a basis for developing and documenting requests for exceptions. The primary focus of the request should be highway safety. The design and proposed mitigation should be the best practical alternative that considers whether or not other controlling design elements will be adversely affected.

The major controlling design elements that need to be evaluated for conformity to established standards include:

- Design speed;
- Through lane and auxiliary lane widths;
- Shoulder widths;
- Stopping sight distance on vertical and horizontal curves;
- Horizontal alignment (radius of curve);
- Vertical alignment;
- Minimum and maximum grades;
- Cross slopes;
- Superelevation rate;
- Horizontal clearance;
- Vertical clearance;
- Bridge width; and
- Structural capacity.

An exception for design speed should not be sought as this element establishes most if not all of the other parameters to be met. As discussed in this chapter and in several other sections in this manual, design speed is an achievable speed selected by the designer based on the various factors the designer must consider. Design elements that can not be met within that selected design speed should be supported by seeking a design exception in accordance with the following guidelines.

All variances from standards need to be documented in the project files, and, in many cases, require approval from the Chief Engineer and FHWA. The level of documentation depends on the project scope, functional classification and other factors. Depending upon the significance of the request, the support information may include some or all of the following:

- Existing roadway characteristics,
- Required and proposed design criteria;
- Cross section or geometric figures comparing the existing and proposed conditions;
- Supporting calculations and cost analysis;
- Analysis of accident records;
- A discussion on the compatibility with adjacent sections;
- Effect on right-of-way;
- Environmental constraints;
- Any proposed mitigation considered to help offset the variance from the design standard;
- Mitigation costs;
- Public support or opposition; and
- Other pertinent factors

3.1.3.1 NEW CONSTRUCTION AND RECONSTRUCTION PROJECTS

New construction and reconstruction projects on the interstate system and the NHS are expected to be in conformance with the appropriate standards and exceptions should be rare. All projects on the NHS (except preventive maintenance) shall conform to the standards in the current edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book). All projects on the interstate system (except preventive maintenance) shall also meet the design criteria contained in the current edition of *A Policy on Design Standards—Interstate System.* Projects having full federal oversight by FHWA are established in the current *Delaware*

Transportation/Federal Highway Administration Stewardship Agreement to Implement the Flexibility of Provisions of 23 United States Code Part 106. FHWA must approve all design exceptions for projects with full federal oversight.

New construction and reconstruction projects require a formal design exception as described in "Required Design Exception Documentation" (Figure 3-2) to justify the rationale for departure from the established design standards. The Chief Engineer must approve design exceptions for all new construction and reconstruction projects. "Design Exception Request" (Figure 3-3) is a guide format for developing a design exception request. It is important that the designer complete the "Design Control Checklist" (Figure 3-4) and the "Design Criteria Form" (Figure 3-5) for all new construction and reconstruction projects.

Figure 3-2
Required Design Exception Documentation

Project Type	Design Exception Request (Fig. 3-3)	Design Control Checklist (Fig. 3-4)	Design Criteria Form (Fig. 3-5)	Asst. Director, Design, and Chief Engineer Approval	FHWA Approval
New Construction or Reconstruction – Full Federal Oversight	X	X	X	X	X
New Construction or Reconstruction – State Administered	X	X	X	X	
Intermediate Level		X	X		
Preventive Maintenance		X			
Miscellaneous Improvement		X			

3.1.3.2 INTERMEDIATE PROJECTS

The scope of intermediate projects is improvements being retrofitted into the existing infrastructure. At times, it may be necessary to deviate from the design standards. It is important that the designer complete the "Design Control Checklist" (Figure 3-4) and the "Design Criteria Form" (Figure 3-5) for all intermediate projects. If design standards cannot be met due to site conditions, then it shall be noted on the Design Criteria Form. In addition, there must be a memo to the project file explaining the justification for the variances in the design standards signed by the Project Engineer, the Group Engineer and the Assistant Director. The level of documentation can vary depending on the project intent, environmental concerns and other issues. In most cases it is not necessary to provide the same level of documentation as provided in a formal design exception request because the project's scope has already been established. The following are examples of intermediate projects:

- Safety improvement projects
- Addition of channelizing islands with no reduction in existing lane or shoulder width
- Intersection improvements with no reduction in existing lane or shoulder width
- Minor lane or shoulder widening
- Traffic calming features, e.g. speed humps, chicanes, midblock median islands, chokers, or narrowed lanes. (Consult with FHWA for projects on NHS Routes.)
- In-kind superstructure replacement

3.1.3.3 PREVENTIVE MAINTENANCE PROJECTS

Preventive maintenance includes rehabilitation or restoration of specific elements of a highway facility when such activities are a cost- effective means of extending the pavement or bridge life and shall not degrade any existing or geometric aspects of the facility. The majority of the work for these projects is between existing curb lines or outer edges of existing shoulders. These types of projects are not required to provide any

documentation for design standards that cannot be met. A general list of preventative maintenance items is below:

- Pavement milling
- Pavement resurfacing with no reduction in lane widths
- Pavement repair (e.g. sawing, sealing, pothole patching)
- Rehabilitation of existing structures
  - Superstructure/substructure rehabilitation
  - Deck rehabilitation
  - Joint replacement or repair
  - Bearing replacement
  - Seismic retrofit
  - Structure painting
  - Scour countermeasures
  - Retaining structure rehabilitation
  - Culvert lining

3.1.3.4 MISCELLANEOUS IMPROVEMENT PROJECTS

These projects are typically beyond the existing edge of pavement and are intended to improve safety and aesthetics. They may also include improvements within the roadway intended to improve safety which do not degrade the existing highway geometrics. These projects are not required to provide any documentation for design standards that cannot be met. A general list of miscellaneous improvement project work items is below:

- New or replaced curb and/or sidewalk
- Modifying sidewalk to comply with ADA requirements
- Roadside safety enhancements
  - New guardrail installation
  - Guardrail repair/replacement
  - New impact attenuators
  - Impact attenuator repair or replacement
  - Upgrading bridge rails
  - Removal of obstructions
- Median barrier replacement
- Drainage improvements
- Signing
- Large ground-mounted signs
- New sign structures (sign bridge, cantilever and bridge-mounted, provided they meet clearance requirements)
- Existing sign structure rehabilitation
- Striping with no additional lanes nor reduction in existing lane width
- New or replaced raised pavement markers
- New or upgraded signals
- ITS (e.g. fiber optic cable, message signs, cameras, emergency call boxes)
- New or upgraded lighting systems
- Fencing, provided existing stopping sight distance is not degraded
- Glare screens, provided existing stopping sight distance is not degraded
- Repair of structural components resulting from traffic impact
- Advanced utility relocation projects
- Landscape improvements

Figure 3-3
Design Exception Request

State Project No._____Federal-Aid Project No._____

Date: _____**Oversight Project: Yes _____No____**

Design Exception Abstract: (Provide a short summary detailing the nature of the exception, reasons for the request, etc.)

Note:
For all NHS projects, the thirteen controlling criteria to be met are design speed; through lane and auxiliary lane width; shoulder width; bridge width, structural capacity, horizontal alignment; vertical alignment; grades; stopping sight distance, cross-slope; superelevation; horizontal clearance; and vertical clearance.

RECOMMENDATION:

The purpose of this project is to----------.

The most effective method of addressing this is----------.

Based upon the conditions presented, it is recommended that a design exception be approved for the controlling substandard design element as justified.

Recommended By: _____
Squad Manager

Recommended By: _____
Group Engineer

Recommended By: _____
Assistant Director-Transportation Solutions

Recommended By: _____
Assistant Director-Design

Approved By: _____ **Date**:_____
Chief Engineer

Approved By: _____ **Date**:_____
Federal Highway Administration (NHS oversight projects only)

Enclosures: (Include design criteria, figures, calculations, etc. to document request.)

Figure 3-4
Design Control Checklist

PROJECT DATA

Squad Leader/Project Manager: _____

Project Title: _____

Contract No.: _____

Federal Aid Project No: _____

Project Limits: _____

Type of Construction: _____

Project Scope and Initial Estimate: _____

_____

_____

_____

DESIGN DATA

Functional Classification:_____

Current AADT (Year)_____

Projected AADT (Year):_____

Projected DHV (Year):_____

% Trucks_____

Directional Distribution (%):_____

Design Speed _____

Design Vehicle_____

Design Level of Service _____

Clear Zone_____

Recommended By: _____

Squad Manager

Recommended By: _____

Group Engineer

Recommended By: _____

Assistant Director-Transportation Solutions

Approved By: _____

Director-Transportation Solutions

Figure 3-5
Design Criteria Form

Design Criteria		
Design Factor	**As per Road Design Manual**	**Provided**
Design Speed*		
Width of Through Lanes*		
Width of Auxiliary Lanes*		
Width of Outside Shoulder*		
Width of Inside Shoulder*		
Cross Slope*		
Width of Median		
Stopping Sight Distance*		
Minimum Horizontal Curve Radius*		
Minimum K (Crest)*		
Minimum K (Sag)*		
Maximum % Grade*		
Maximum front slope (Unprotected Section)		
Maximum back slope		
Barrier Offset		
Superelevation Rate (%)*		
Bridge Width*		
Vertical Clearance*		
Structural Capacity*		
Horizontal Clearance *		
Width of clear zone		

General Notes:
- Use this form primarily for new construction or reconstruction projects.

* The Chief Engineer must approve design criteria deviating from the requirements of the Road Design Manual through the use Figure 3-3 "Design Exception Request."

Recommended By: _____
Project Manager

Recommended By: _____
Group Engineer

Approved By: _____
Assistant Director-Transportation Solutions

3.1.4 DETERMINATION OF STANDARDS

Figure 3-6 graphically defines the nomenclature used when describing the various elements that establish a roadway's cross section. The dimensions and geometrics needed to design this roadway section are known as a project's design standards. The basic information needed before beginning the process of establishing a project's design standards is:

- The ***functional classification*** of the road section to be improved. A roadway's classification is shown on DelDOT's current functional classification map but should be verified with the Division of Planning. A part of the initial field review and scoping meeting should be to verify that the area and roadway section being considered are truly representative of the designated classification.

- The ***scope of work*** proposed for the project under consideration is in the Project Initiation Form, project development documentation and other supporting data furnished to the designer.

- The ***traffic data*** for the road section is obtained from the Division of Planning and includes current traffic, projected traffic, percent trucks, accident history, etc.

With this information, the designer can proceed with the process of selecting the design speed.

3.2 STANDARDS BASED ON DESIGN SPEED

3.2.1 SELECTION OF DESIGN SPEED

The design speed establishes basic criteria for certain design elements. Two design standard considerations are related directly to the design speed:

- Curvature and superelevation, and
- Required sight distances.

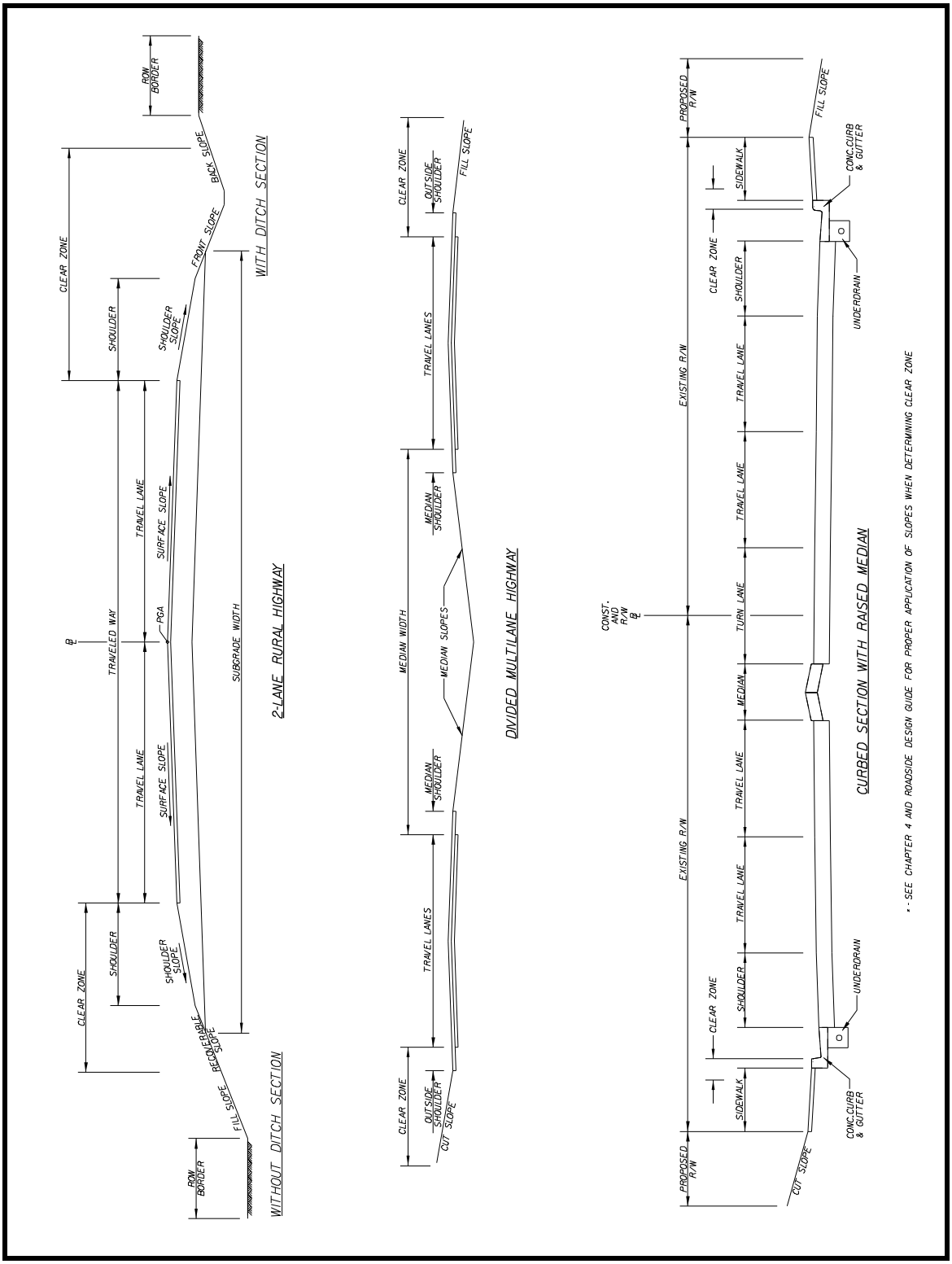
The designer's goal is to provide at least the minimum values, and preferably larger values, for these standards, regardless of traffic volumes, functional classification or any other consideration. These design elements are very closely related to traffic safety and should not be compromised.

A first step in determining the appropriate design standards is to establish a reasonable and realistic design speed. Since the majority of design controls are related to the design speed, this decision needs to be based on more factors than a roadway's functional classification and traffic volume.

The design speed selected should accommodate a high percentage of drivers, including the reasonable and prudent driver. Other considerations include topography, anticipated operating speeds, driver expectations, volume and mix of vehicles, the volume and type of non-vehicular traffic, driver familiarity, level of congestion reasonably acceptable to the motorists, and community values.

Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Figure 3-6
Typical Section Nomenclature



Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page 384, Exhibit 5-5;
- Collector Roads and Streets (Rural) — page 425, Exhibit 6-5;
- Collector Roads and Streets (Urban) — page 433;
- Arterials (Rural) — page 448, Exhibit 7-3;
- Divided Arterials (Rural) — page 455;
- Urban Arterials — page 472;
- Freeways — page 504.

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-5, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the

maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smoother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages 167 to 174, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

Vertical curvature, horizontal curvature, roadside obstructions, or any combination of these elements can restrict sight distance. Procedures for checking available sight distances are described in the Green Book, pages 127-131.

3.2.4 PASSING SIGHT DISTANCE

Consideration of passing sight distance is limited to two-lane, two-way highways on which vehicles frequently overtake slower-moving vehicles and the passing operation must be accomplished on a lane used by opposing traffic.

Passing sight distance for design is determined on the basis of the length needed to accomplish the passing maneuver. Derivation of the required distance is described in the Green Book, pages 118-126. AASHTO recommends

that, "In designing a highway these distances should be exceeded as much as practical ..."

These distances for design should not be confused with other distances used as warrants for placing no-passing pavement markings on completed highways. Values shown in the MUTCD are substantially less than the design distances and are derived from traffic operation control needs based on assumptions different from those for design.

Because of vertical and horizontal sight limitations, nearly all two-lane highways have some no-passing restrictions. In rolling terrain, the proportionate amount of no-passing sections usually becomes greater. Normally it is impracticable to attempt to provide passing sight distance throughout the entire length of a project. The principal design consideration is to try to provide adequate passing opportunities as frequently as possible.

There are no fixed values for the frequency of passing sections. Experience shows that highway capacity is measurably reduced when a significant percentage of a section of highway is restricted to sight distances less than 1500 ft [500 m]. Highways with high traffic volumes will require a higher proportion of passing opportunities than those with low traffic volumes. Where an analysis shows that a lack of passing sight distances has reduced capacity to near or below the expected traffic volumes, it is necessary to consider adjustments in the alignment and grade, or to provide additional lanes.

3.3 STANDARDS BASED ON TRAFFIC VOLUMES

Standards not directly related to design speed are influenced primarily by traffic volumes. Tables for these standards shown in the tables at the end of this chapter reflect variations by traffic volume ranges.

3.3.1 NUMBER OF LANES

The number of lanes required for any highway is directly related to the facility's traffic volume and desired level of service. But there are no simple, fixed criteria for these relationships. The recommended number of lanes is normally obtained through the project development process.

The *Highway Capacity Manual* gives two very general guidelines for determining the need for additional lanes. These numbers are based on long sections of roadway with uninterrupted traffic flow having the highest standards for design controls (horizontal and vertical geometrics and cross-sectional elements), ideal weather conditions, daylight, etc.

1. Under ideal conditions, a two-lane rural highway can accommodate about 900 passenger vehicles (two-way) per hour with a reasonably high level of service if there are adequate passing opportunities and no long, steep grades. Considerably more vehicles can be accommodated if motorists are willing to accept a lower level of service, a greater degree of congestion and lower operating speeds.
2. Under ideal conditions, a multi-lane highway can accommodate about 900 passenger vehicles per lane per hour. Again, considerably more vehicles can be accommodated, if lower levels of service can be tolerated.

Most roadways do not meet the ideal conditions. The HCM defines the ideal roadway as follows: (1) meets or exceeds design speed; (2) has 12 ft [3.6 m] travel lane widths; (3) has shoulder widths greater than 6 ft [1.8 m]; (4) has minimal no passing zones; (5) carries predominantly passenger cars; (6) has evenly distributed traffic flow; (7) has minimum crossing and entering traffic interference; and (8) has level terrain.

Although all these elements are rarely available within a project's limit, capacity is

usually not a problem on most of the rural roadways in Delaware. Exceptions are some of the principal arterial routes, particularly in the vicinity of urban areas.

Most proposed improvements will be in traffic volume ranges where the existing number of lanes will be adequate without the need for detailed study. However, capacity may be influenced where the traffic volume exceeds about 900 DHV or where there are unusual conditions of alignment, grade or signalization.

Designers working with Traffic and the Division of Planning should identify the need for additional through lanes or, if applicable, auxiliary climbing lanes. Such a change after project initiation is a major decision affecting all aspects of a project from cost to environmental and social impacts.

3.3.2 SURFACED LANE WIDTHS

The traveled way designated for vehicle operation (excluding shoulders) normally consists of two or more surfaced traffic lanes. The impact of providing adequate lane widths is wide ranging and includes maintaining and/or enhancing driver safety, driver comfort, the level of service, capacity, and the frequency and extent of shoulder and pavement surface maintenance.

For all new construction and reconstruction on arterial and collector roads, the desirable surfaced travel lane width is 12 ft [3.6 m]. If the scope of work is limited, speeds are low, truck volumes are light or there are no defined safety problems, surfaced lane widths of 11 ft [3.3 m] may be acceptable, particularly in urbanized areas with restricted right-of-way and increased pedestrian activity. However, for urban arterials with higher speeds, predominantly free-flowing conditions, and higher traffic volumes, surfaced lane widths of 12 ft [3.6 m] are desirable. For local roads and streets, surfaced traffic lanes normally should be 11 feet [3.3 m] wide but AASHTO allows lane widths of 9 [2.7 m] or 10 ft [3.0 m] where there is restricted or low truck use, low traffic volumes and low operating speeds. See Section 3.2.1 for information on selecting lane and shoulder widths.

For pavements on new construction or major reconstruction projects with existing or projected high concentrations of truck traffic, a wider pavement provides more edge strength and has been found to be structurally better for heavy loads. Consideration should be given to widening the pavement an additional 2 ft [0.6 m] under these circumstances. The lanes should be striped for 12 ft [3.6 m] lanes to keep trucks away from the edge of the pavement. The extra width can be considered part of the shoulder. If the mainline and shoulders are constructed of Portland cement concrete and the shoulders are structurally tied to the mainline, this additional width is not normally necessary. For divided highways, the widening should be adjacent to the outside shoulder; on two-lane roadways the widening should be equally divided on each side.

3.3.3 SHOULDER WIDTH

The total shoulder width is the distance from the edge of the traffic lane to the intersection of the shoulder slope with the front slope, or to the face of curb. In sections without curbs there are two terms used to describe the shoulder area. The “graded” width of shoulder is that measured from the edge of the traveled way to the intersection of the shoulder slope and the front slope. The “useable” width of shoulder is the actual width that can be used when a driver makes a stop.

Having a sharp break at the point of intersection of the edge of the graded shoulder and the front slope is not a good practice. Instead a rounding of 4 to 6 ft [1.2 to1.8 m] with a front slope 4:1 or flatter is the best practice. This rounding improves the general safety of the roadside by reducing the likelihood of encroachment, thus giving the errant driver more chance to regain control. Other considerations

are that rounding may reduce rollovers and the possibility that the vehicle may become airborne. A portion of the rounding (usually one half) can be considered part of the "useable" width. Where front slopes are steeper than 4:1, the rounding should occur outside of the useable shoulder width.

Well-designed and maintained shoulders are necessary on rural highways with any appreciable traffic volumes. Shoulders and their widths are an important consideration in establishing a project's design standards. The benefits of including a shoulder include:

- Providing a refuge when a driver makes an emergency or parking stop.
- Providing lateral recovery areas for vehicles that inadvertently leave the traffic lane.
- Providing improved sight distance in cut areas.
- Providing areas for maintenance operations, including snow removal and storage.
- Providing for alternative modes of travel by pedestrians, bicyclist, joggers, transit operations, etc.
- Structurally improving the service life of the pavement by increasing the stability of the roadway's base and surfacing materials at the edge of the through traffic lane.
- Providing the opportunity for improved subgrade drainage designs.

Section 3.2.1 discusses the selection of shoulder widths. Normally shoulder widths of 10 ft [3.0 m] are used on new construction projects for arterial highways with relatively high traffic volumes. Where truck traffic exceeds 250 DHV it is desirable to have a paved shoulder width of 12 ft [3.6 m]. AASHTO allows narrower shoulder widths on most roadways with lower traffic volumes. However, wider shoulders widths should be provided on these projects when practical. Where bicyclists and pedestrians are to be accommodated on the shoulders, a minimum useable shoulder width, clear of any rumble strips, of 4 ft [1.2 m] should be used. On highways with three or more lanes in each direction a median shoulder width of 10 ft [3.0 m] is desirable. This provides a refuge area for disabled vehicles without affecting roadway capacity and flow.

3.3.4 SURFACED SHOULDER WIDTH

The surfaced shoulder width is that part constructed to provide better all-weather load support than is afforded by natural soils or stabilized materials. The paved portion of the shoulder also protects the edge of the traffic lane pavement from deterioration and raveling. More discussion on shoulder surfacing is in Chapter Nine - Pavement Selection. Normally the shoulder's structural design, including surface material, is recommended by the Materials and Research Section.

3.3.5 SIDE SLOPES

Various cross section slopes are identified in Figure 3-6. Four of these slopes are described below.

- **Front Slope.** The slope extending outward and downward from the shoulder to the ditch line.
- **Back Slope**. The slope extending upward and outward from the ditch line to intersect the natural ground.
- **Fill Slope**. The slope extending outward and downward from the shoulder to intersect with the natural ground; it may include a ditch section.
- **Cut Slope**. The slope extending outward and upward from the shoulder, intersecting the ditch slope and then extending upward from the ditch back slope to natural ground.

It is generally desirable that these slopes be 6:1 or flatter. Often, from a practical standpoint, they must be steeper. There is a distinct relationship between the steepness of side

slopes, operating speeds and the desirable widths of clear zones. Chapter Four discusses the relationship of these slopes to establishing a roadway's clear zone. General criteria for side slopes are presented in Figure 4-4, in terms of both desirable slopes and maximum slopes. The desirable slopes should be provided wherever feasible.

3.3.6 HORIZONTAL CLEARANCE AND CLEAR ZONE

3.3.6.1 HORIZONTAL CLEARANCE

Horizontal clearance is the lateral distance from edge of traveled way to a roadside feature or object for a roadway with barrier curb. Roadways having curbed sections should be provided with a minimum horizontal clearance of 1.5 ft [0.5 m] beyond the face of curb, with wider offsets (if possible to the full clear zone width) provided where practical since most types of curbs provide little help in redirecting an errant vehicle. Please see the Green Book for more information regarding horizontal clearance and AASHTO's Roadside Design Guide for more information regarding the clear zone width. If the minimum horizontal clearance cannot be provided in curbed areas, then a design exception is required.

3.3.6.2 CLEAR ZONE

The clear zone is defined in AASHTO's Roadside Design Guide as "the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry." This border area includes any shoulders or auxiliary lanes. Adequate lateral clearance between the edges of traffic lanes and roadside obstructions has been shown to be a very important safety factor. Vehicles leaving the roadway should have a reasonable opportunity to recover control and return to the roadway without overturning or colliding with roadside obstacles such as trees, poles, headwalls or other large objects. The combination of a relatively flat slope and an obstacle-free roadside within the prescribed clear zone helps this situation.

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the safest available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-4), the Design Criteria Form (Figure 3-5) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there are steep grades accompanied by considerable no-passing distances.

More detailed guidelines and criteria for the design of grades, including critical lengths of grades and minimum and maximum grades are presented in Chapter Five - Alignment and Superelevation and the Green Book, pages 231-250. The maximum grades should be used infrequently, only as dictated by severe terrain conditions. When it is necessary to use maximum grades, the designer should check other design criteria and roadside features that may be improved to minimize the impact of using the higher design grade.

3.3.8 BRIDGES

The designer should coordinate with the Bridge Design Section when determining vertical clearances. A minimum vertical clearance for roads over interstate, U.S. and state routes is 16.5 ft [5 m]. Pedestrian bridges and overhead sign structures must have an extra 1 foot [0.3 m] of clearance, a total of 17.5 ft [5.3 m]. These clearances allow for a 4 in. [100 mm] future resurfacing.

3.3.9 MEDIANS

Geometric criteria for medians on multilane divided highways are discussed in Chapter Four.

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DEPARTMENT OF TRANSPORTATION
800 BAY ROAD
P.O. BOX 778
DOVER, DELAWARE 19903

CAROLANN WICKS, P.E.
SECRETARY

October 9, 2007

Dear Road Design Manual Holder:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual - August 2007 Revision

This is the formal issuance of the August 2007 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective October 15, 2007. You can download this revision from DelDOT's web site, http://www.deldot.gov/information/pubs_forms/manuals/road_design/index.shtml. Changes have been made to a portion of Section 9.3.4 regarding the use of underdrains and roadbed drainage.

Please replace the pages in your manual with the appropriate sheets. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely yours,

Dennis M. O'Shea

Dennis M. O'Shea
Assistant Director - Design

DMO:los
cc: Robert Taylor, Chief Engineer
Michael Simmons, Assistant Director, Project Development, South Region
Drew Boyce, Assistant Director, Project Development, North Region
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Linda M. Osiecki, Program Manager, Quality

DelDOT

directly for designing flexible pavements but must be converted to a modulus of subgrade reaction (k-value) for the design of rigid pavements or composite pavements. The resilient modulus is also a soil property used in analyzing multilayered material systems for predicting roughness, cracking, faulting, rutting, and other potential distresses.

The value of a roadbed's resilient modulus is dependent on how well the roadbed soil is placed in conformance with the specified compaction parameters. For most projects, the material is to be placed in accordance with the *Standard Specifications* with no special treatment. However, the Soil and Pavement Design Report may indicate that there is anticipated difficulty with the existing roadbed soil meeting the design M_R value. For soils that are excessively expansive the report may recommend these soils be covered by select material sufficiently deep enough to reduce or eliminate the expansive affect of the natural material. Other solutions may include the adding of an admixture to reduce the water content or the use of a geotextile.

One of the more difficult soils encountered on projects are those having a large organic content. These materials are extremely compressible, unstable and frequently non-uniform in properties and depth. These soils are the most complicated and expensive to deal with in order to provide an adequate roadbed. Small, shallow or localized deposits are most often excavated and replaced with suitable material. Deeper and more expansive areas involve more detailed geotechnical design, more complicated construction techniques and costs. Treatments other than complete removal are more time dependent allowing for the slow consolidation and removal of excess moisture. Methods available include surcharge embankments for preconsolidation of the underlying material usually involving sand drains which allow the water to rise to the surface and be removed. The Materials and Research Section is responsible for identifying and designing the most economical method treating this type of problem area.

Underdrains (a system of perforated pipes to collect and transmit the water to an outfall site) are recommended for use on all roadway projects to adequately address drainage and removing water from the roadbed. If site conditions indicate that underdrains may not be required, contact the Materials and Research Section to initiate further investigation.

The soil and pavement condition survey will normally identify roadbed drainage problem areas or soils highly susceptible to expansion or loss of strength with increase in water content. When either of these conditions exist, the Materials and Research Section may recommend additional work and/or materials to address the existing conditions.

Another type of material encountered in constructing roadbeds is classified as cohesionless (sandy) soil and is much more difficult for the contractor to place and compact; it is readily displaced under the load of the equipment. To stabilize this type of soil it may be necessary to blend granular material or add a suitable admixture. Wet clay soils may also be encountered. Because of high moisture content this type of soil is unstable and cannot be compacted. Long periods of dry weather and exposure to the air are required to reduce the water content. To reduce the time necessary to reuse these materials, the recommendation may be to add a suitable admixture that hastens drying or cover the area with a more suitable select material. Removing the material and replacing it with suitable material allowing construction to continue is an option. The material may be used in areas that don't require compaction or moved to an available site for air-drying and reuse at a later time.

9.3.5 PAVING MATERIALS

Depending upon materials that comprise a pavement, the pavement structure is identified as either a flexible or rigid pavement. Combining these two types of paving materials in a pavement structure as a subbase or surface course results in a composite pavement.

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DOVER, DELAWARE 19903

CAROLANN WICKS, P.E.
SECRETARY

August 1, 2008

Dear Road Design Manual Holder:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual - July 2008 Revision

This is the formal issuance of the July 2008 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision concerns Chapter 6, entitled "Drainage and Stormwater Management", replaces the 1985 Chapter 6. This revision will be effective August 15, 2008. You can download this revision from DelDOT's web site, http://www.deldot.gov/information/pubs_forms/manuals/road_design/index.shtml.

Please replace Chapter 6 in your manual with this revision. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely yours,

Dennis M. O'Shea
Assistant Director - Design

DMO:los
cc: Robert Taylor, Chief Engineer
Michael Simmons, Assistant Director, Project Development, South Region
Drew Boyce, Assistant Director, Project Development, North Region
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CAROLANN WICKS, P.E.
SECRETARY

April 20, 2009

Dear Road Design Manual Holder:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual - April 2009 Revision

This is the formal issuance of the April 2009 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective April 30, 2009. You can download this revision from DelDOT's web site, http://www.deldot.gov/information/pubs_forms/manuals/road_design/index.shtml. The web site also includes the revision with changes highlighted for your reference. Changes have been made to a portion of Chapter 3 regarding the types of construction for projects and the reference to DelDOT's Stewardship Agreement with FHWA.

Please replace the pages in your manual with the appropriate sheets. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely yours,

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cc: Natalie Barnhart, Chief Engineer
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Kevin T. Canning, Quality Engineer
Linda M. Osiecki, Program Manager, Quality

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3.1.3 DEPARTURE FROM STANDARDS

Most projects are expected to meet at least the minimum standard design values established for the project level. Occasionally there may be conditions that warrant consideration of a lower value. For projects on higher functional classification roadways, it is more critical to strive to meet or exceed all of the applicable standards, primarily because the motorist expects a higher standard and will drive the facility accordingly.

The need for exceptions to the standards must be identified early in the project development phase in order that approvals or denials will not delay completion of the design or require extensive redesign. However, the need to evaluate a lower design value may arise at any time during the design process and needs to be addressed expeditiously. Thorough documentation is essential. The need for exceptions should not be viewed as normal or routine. The forms in this chapter shall be used to document decisions on design criteria and as a basis for developing and documenting requests for exceptions. The primary focus of the request should be highway safety. The design and proposed mitigation should be the best practical alternative that considers whether or not other controlling design elements will be adversely affected.

The major controlling design elements that need to be evaluated for conformity to established standards include:

- Design speed;
- Through lane and auxiliary lane widths;
- Shoulder widths;
- Stopping sight distance on vertical and horizontal curves;
- Horizontal alignment (radius of curve);
- Vertical alignment;
- Minimum and maximum grades;
- Cross slopes;
- Superelevation rate;
- Horizontal clearance;
- Vertical clearance;
- Bridge width; and
- Structural capacity.

An exception for design speed should not be sought as this element establishes most if not all of the other parameters to be met. As discussed in this chapter and in several other sections in this manual, design speed is an achievable speed selected by the designer based on the various factors the designer must consider. Design elements that cannot be met within that selected design speed should be supported by seeking a design exception in accordance with the following guidelines.

All variances from standards need to be documented in the project files, and, in many cases, require approval from the Chief Engineer and FHWA. The level of documentation depends on the project scope, functional classification and other factors. Depending upon the significance of the request, the support information may include some or all of the following:

- Existing roadway characteristics,
- Required and proposed design criteria;
- Cross section or geometric figures comparing the existing and proposed conditions;
- Supporting calculations and cost analysis;
- Analysis of accident records;
- A discussion on the compatibility with adjacent sections;
- Effect on right-of-way;
- Environmental constraints;
- Any proposed mitigation considered to help offset the variance from the design standard;
- Mitigation costs;
- Public support or opposition; and
- Other pertinent factors

There are several project types described in the following sections. Figure 3-2 shows the types of construction that fall under each project type. The type of construction shall be shown on the title sheet of the construction plans.

Figure 3-2
Types of Construction

Project Type	Type of Construction
New Construction and Reconstruction	Bridge Replacement
	New Construction (Road/Bridge)
	4-R
Intermediate	Bridge Rehabilitation
	Bridge Superstructure Replacement
	Intersection Improvements
	Safety Improvements
	Traffic Calming
Preventive Maintenance	Bridge Painting
	Bridge Preservation
	Bridge Preventive Maintenance
	Community Transportation Fund
	Pavement Preservation
	Scour Countermeasures
Miscellaneous Improvement	Advanced Utility
	Bike and Pedestrian Improvements
	Drainage Improvements
	ITS
	Landscaping
	Lighting
	Railroad Crossing
	Signal
	Signing and Striping
	Small Structure Repair
	Transportation Enhancement
	Miscellaneous Improvements (e.g., Wetland Mitigation, DTC Rail, Facilities)

3.1.3.1 NEW CONSTRUCTION AND RECONSTRUCTION PROJECTS

New construction and reconstruction projects on the interstate system and the NHS are expected to be in conformance with the appropriate standards and exceptions should be rare. All projects on the NHS (except preventive maintenance) shall conform to the standards in the current edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book). All projects on the interstate system (except preventive maintenance) shall also meet the design criteria contained in the current edition of *A Policy on Design Standards—Interstate System.* Projects having full federal oversight by FHWA are established in the current *Stewardship and Oversight Agreement* between FHWA and DelDOT. FHWA must approve all design exceptions for projects with full federal oversight.

New construction and reconstruction projects require a formal design exception as described in "Required Design Exception Documentation" (Figure 3-3) to justify the rationale for departure from the established design standards. The Chief Engineer must approve design exceptions for all new construction and reconstruction projects. "Design Exception Request" (Figure 3-4) is a guide format for developing a design exception request. It is important that the designer complete the "Design Control Checklist" (Figure 3-5) and the "Design Criteria Form" (Figure 3-6) for all new construction and reconstruction projects.

Figure 3-3
Required Design Exception Documentation

Project Type	Design Exception Request Fig. 3-4)	Design Control Checklist Fig. 3-5)	Design Criteria Form Fig. 3-6)	Asst. Director, Design, and Chief Engineer Approval	FHWA Approval
New Construction or Reconstruction – Full Federal Oversight	X	X	X	X	X
New Construction or Reconstruction – State Administered	X	X	X	X	
Intermediate Level		X	X		
Preventive Maintenance		X			
Miscellaneous Improvement		X			

3.1.3.2 INTERMEDIATE PROJECTS

The scope of intermediate projects is improvements being retrofitted into the existing infrastructure. At times, it may be necessary to deviate from the design standards. It is important that the designer complete the "Design Control Checklist" (Figure 3-5) and the "Design Criteria Form" (Figure 3-6) for all intermediate projects. If design standards cannot be met due to site conditions, then it shall be noted on the Design Criteria Form. In addition, there must be a memo to the project file explaining the justification for the variances in the design standards signed by the Project Engineer, the Group Engineer and the Assistant Director. The level of documentation can vary depending on the project intent, environmental concerns and other issues. In most cases it is not necessary to provide the same level of documentation as provided in a formal design exception request because the project's scope has already been established. The following are examples of intermediate projects:

- Safety improvement projects
- Addition of channelizing islands with no reduction in existing lane or shoulder width
- Intersection improvements with no reduction in existing lane or shoulder width
- Minor lane or shoulder widening
- Traffic calming features, e.g. speed humps, chicanes, midblock median islands, chokers, or narrowed lanes. (Consult with FHWA for projects on NHS Routes.)
- Superstructure replacement
- Bridge rehabilitation

3.1.3.3 PREVENTIVE MAINTENANCE PROJECTS

Preventive maintenance includes rehabilitation or restoration of specific elements of a highway facility when such activities are a cost- effective means of extending the pavement or bridge life and shall not degrade any existing or geometric aspects of the facility. The majority of the work for these projects is between existing curb lines or outer edges of existing shoulders. These types of projects are not required to provide any documentation for design standards that cannot be met. A general list of preventative maintenance items is below:

- Pavement Preservation - Pavement milling and resurfacing of the same thickness with no reduction in lane widths, or pavement repair (e.g. sawing, sealing, pothole patching)
- Bridge Preservation
    - Bridge painting
    - Bridge Preventive Maintenance
        - Deck rehabilitation
        - Joint replacement or repair
        - Bearing replacement
        - Pile Jackets
    - Scour countermeasures

- Seismic retrofit

3.1.3.4 MISCELLANEOUS IMPROVEMENT PROJECTS

These projects are typically beyond the existing edge of pavement and are intended to improve safety and aesthetics. They may also include improvements within the roadway intended to improve safety which do not degrade the existing highway geometrics. These projects are not required to provide any documentation for design standards that cannot be met. A general list of miscellaneous improvement project work items is below:

- New or replaced curb and/or sidewalk
- Modifying sidewalk to comply with ADA requirements
- Roadside safety enhancements
  - New guardrail installation
  - Guardrail repair/replacement
  - New impact attenuators
  - Impact attenuator repair or replacement
  - Upgrading bridge rails
  - Removal of obstructions
- Median barrier replacement
- Drainage improvements
- Signing
- Small Structure Repair
  - Culvert lining
  - Retaining structure repair
- Large ground-mounted signs
- New sign structures (sign bridge, cantilever and bridge-mounted, provided they meet clearance requirements)
- Existing sign structure rehabilitation
- Striping with no additional lanes nor reduction in existing lane width
- New or replaced raised pavement markers
- New or upgraded signals
- ITS (e.g. fiber optic cable, message signs, cameras, emergency call boxes)
- New or upgraded lighting systems
- Fencing, provided existing stopping sight distance is not degraded
- Glare screens, provided existing stopping sight distance is not degraded
- Repair of structural components resulting from traffic impact
- Advanced utility relocation projects
- Landscape improvements

Figure 3-4
Design Exception Request

State Project No._____Federal-Aid Project No._____

Date: _____**Oversight Project: Yes _____No____**

Design Exception Abstract: (Provide a short summary detailing the nature of the exception, reasons for the request, etc.)

Note:
For all NHS projects, the thirteen controlling criteria to be met are design speed; through lane and auxiliary lane width; shoulder width; bridge width, structural capacity, horizontal alignment; vertical alignment; grades; stopping sight distance, cross-slope; superelevation; horizontal clearance; and vertical clearance.

RECOMMENDATION:

The purpose of this project is to----------.

The most effective method of addressing this is----------.

Based upon the conditions presented, it is recommended that a design exception be approved for the controlling substandard design element as justified.

Recommended By: _____
Squad Manager

Recommended By: _____
Group Engineer

Recommended By: _____
Assistant Director-Transportation Solutions

Recommended By: _____
Assistant Director-Design

Approved By: _____ **Date**: _____
Chief Engineer

Approved By: _____ **Date**: _____
Federal Highway Administration (NHS oversight projects only)

Enclosures: (Include design criteria, figures, calculations, etc. to document request.)

Figure 3-5
Design Control Checklist

PROJECT DATA

Squad Leader/Project Manager: _____

Project Title: _____

Contract No.: _____

Federal Aid Project No: _____

Project Limits: _____

Type of Construction:_____

Project Scope and Initial Estimate: _____

_____

_____

_____

DESIGN DATA

Functional Classification:_____

Current AADT (Year)_____

Projected AADT (Year):_____

Projected DHV (Year):_____

% Trucks_____

Directional Distribution (%):_____

Design Speed _____

Design Vehicle_____

Design Level of Service _____

Clear Zone_____

Recommended By: _____

Squad Manager

Recommended By: _____

Group Engineer

Recommended By: _____

Assistant Director-Transportation Solutions

Approved By: _____

Director-Transportation Solutions

Figure 3-6
Design Criteria Form

Design Criteria		
Design Factor	**As per Road Design Manual**	**Provided**
Design Speed*		
Width of Through Lanes*		
Width of Auxiliary Lanes*		
Width of Outside Shoulder*		
Width of Inside Shoulder*		
Cross Slope*		
Width of Median		
Stopping Sight Distance*		
Minimum Horizontal Curve Radius*		
Minimum K (Crest)*		
Minimum K (Sag)*		
Maximum % Grade*		
Maximum front slope (Unprotected Section)		
Maximum back slope		
Barrier Offset		
Superelevation Rate (%)*		
Bridge Width*		
Vertical Clearance*		
Structural Capacity*		
Horizontal Clearance *		
Width of clear zone		

General Notes:

- Use this form primarily for new construction or reconstruction projects.

* The Chief Engineer must approve design criteria deviating from the requirements of the Road Design Manual through the use Figure 3-4 "Design Exception Request."

Recommended By: _____
Project Manager

Recommended By: _____
Group Engineer

Approved By: _____
Assistant Director-Transportation Solutions

3.1.4 DETERMINATION OF STANDARDS

Figure 3-7 graphically defines the nomenclature used when describing the various elements that establish a roadway's cross section. The dimensions and geometrics needed to design this roadway section are known as a project's design standards. The basic information needed before beginning the process of establishing a project's design standards is:

- The ***functional classification*** of the road section to be improved. A roadway's classification is shown on DelDOT's current functional classification map but should be verified with the Division of Planning. A part of the initial field review and scoping meeting should be to verify that the area and roadway section being considered are truly representative of the designated classification.

- The ***scope of work*** proposed for the project under consideration is in the Project Initiation Form, project development documentation and other supporting data furnished to the designer.

- The ***traffic data*** for the road section is obtained from the Division of Planning and includes current traffic, projected traffic, percent trucks, accident history, etc.

With this information, the designer can proceed with the process of selecting the design speed.

3.2 STANDARDS BASED ON DESIGN SPEED

3.2.1 SELECTION OF DESIGN SPEED

The design speed establishes basic criteria for certain design elements. Two design standard considerations are related directly to the design speed:

- Curvature and superelevation, and
- Required sight distances.

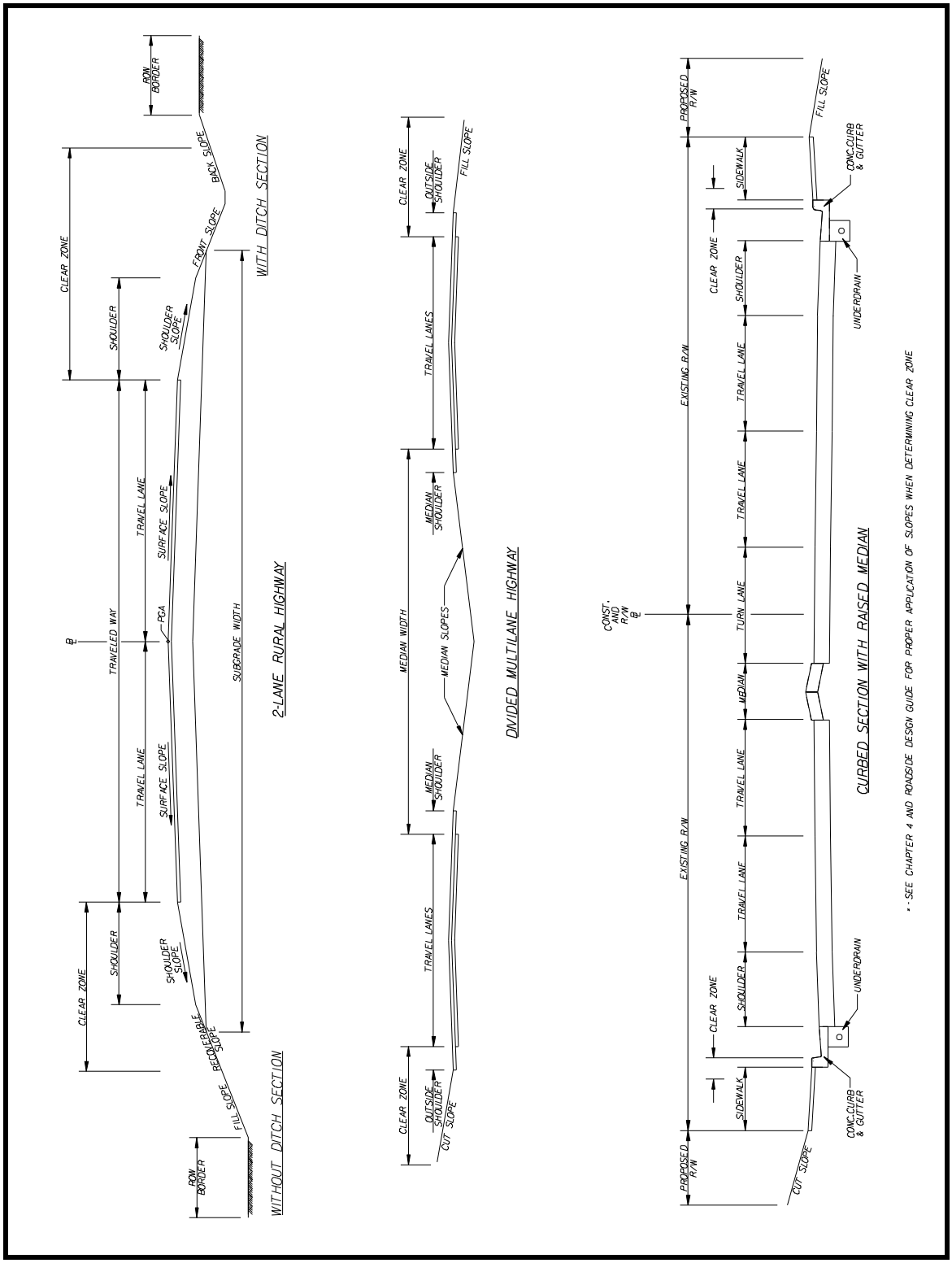
The designer's goal is to provide at least the minimum values, and preferably larger values, for these standards, regardless of traffic volumes, functional classification or any other consideration. These design elements are very closely related to traffic safety and should not be compromised.

A first step in determining the appropriate design standards is to establish a reasonable and realistic design speed. Since the majority of design controls are related to the design speed, this decision needs to be based on more factors than a roadway's functional classification and traffic volume.

The design speed selected should accommodate a high percentage of drivers, including the reasonable and prudent driver. Other considerations include topography, anticipated operating speeds, driver expectations, volume and mix of vehicles, the volume and type of non-vehicular traffic, driver familiarity, level of congestion reasonably acceptable to the motorists, and community values.

Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Figure 3-7
Typical Section Nomenclature



Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page 384, Exhibit 5-5;
- Collector Roads and Streets (Rural) — page 425, Exhibit 6-5;
- Collector Roads and Streets (Urban) — page 433;
- Arterials (Rural) — page 448, Exhibit 7-3;
- Divided Arterials (Rural) — page 455;
- Urban Arterials — page 472;
- Freeways — page 504.

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-6, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the

maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smoother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages 167 to 174, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

Vertical curvature, horizontal curvature, roadside obstructions, or any combination of these elements can restrict sight distance. Procedures for checking available sight distances are described in the Green Book, pages 127-131.

3.2.4 PASSING SIGHT DISTANCE

Consideration of passing sight distance is limited to two-lane, two-way highways on which vehicles frequently overtake slower-moving vehicles and the passing operation must be accomplished on a lane used by opposing traffic.

Passing sight distance for design is determined on the basis of the length needed to accomplish the passing maneuver. Derivation of the required distance is described in the Green Book, pages 118-126. AASHTO recommends

that, "In designing a highway these distances should be exceeded as much as practical ..."

These distances for design should not be confused with other distances used as warrants for placing no-passing pavement markings on completed highways. Values shown in the MUTCD are substantially less than the design distances and are derived from traffic operation control needs based on assumptions different from those for design.

Because of vertical and horizontal sight limitations, nearly all two-lane highways have some no-passing restrictions. In rolling terrain, the proportionate amount of no-passing sections usually becomes greater. Normally it is impracticable to attempt to provide passing sight distance throughout the entire length of a project. The principal design consideration is to try to provide adequate passing opportunities as frequently as possible.

There are no fixed values for the frequency of passing sections. Experience shows that highway capacity is measurably reduced when a significant percentage of a section of highway is restricted to sight distances less than 1500 ft [500 m]. Highways with high traffic volumes will require a higher proportion of passing opportunities than those with low traffic volumes. Where an analysis shows that a lack of passing sight distances has reduced capacity to near or below the expected traffic volumes, it is necessary to consider adjustments in the alignment and grade, or to provide additional lanes.

3.3 STANDARDS BASED ON TRAFFIC VOLUMES

Standards not directly related to design speed are influenced primarily by traffic volumes. Tables for these standards shown in the tables at the end of this chapter reflect variations by traffic volume ranges.

3.3.1 NUMBER OF LANES

The number of lanes required for any highway is directly related to the facility's traffic volume and desired level of service. But there are no simple, fixed criteria for these relationships. The recommended number of lanes is normally obtained through the project development process.

The *Highway Capacity Manual* gives two very general guidelines for determining the need for additional lanes. These numbers are based on long sections of roadway with uninterrupted traffic flow having the highest standards for design controls (horizontal and vertical geometrics and cross-sectional elements), ideal weather conditions, daylight, etc.

1. Under ideal conditions, a two-lane rural highway can accommodate about 900 passenger vehicles (two-way) per hour with a reasonably high level of service if there are adequate passing opportunities and no long, steep grades. Considerably more vehicles can be accommodated if motorists are willing to accept a lower level of service, a greater degree of congestion and lower operating speeds.
2. Under ideal conditions, a multi-lane highway can accommodate about 900 passenger vehicles per lane per hour. Again, considerably more vehicles can be accommodated, if lower levels of service can be tolerated.

Most roadways do not meet the ideal conditions. The HCM defines the ideal roadway as follows: (1) meets or exceeds design speed; (2) has 12 ft [3.6 m] travel lane widths; (3) has shoulder widths greater than 6 ft [1.8 m]; (4) has minimal no passing zones; (5) carries predominantly passenger cars; (6) has evenly distributed traffic flow; (7) has minimum crossing and entering traffic interference; and (8) has level terrain.

Although all these elements are rarely available within a project's limit, capacity is

usually not a problem on most of the rural roadways in Delaware. Exceptions are some of the principal arterial routes, particularly in the vicinity of urban areas.

Most proposed improvements will be in traffic volume ranges where the existing number of lanes will be adequate without the need for detailed study. However, capacity may be influenced where the traffic volume exceeds about 900 DHV or where there are unusual conditions of alignment, grade or signalization.

Designers working with Traffic and the Division of Planning should identify the need for additional through lanes or, if applicable, auxiliary climbing lanes. Such a change after project initiation is a major decision affecting all aspects of a project from cost to environmental and social impacts.

3.3.2 SURFACED LANE WIDTHS

The traveled way designated for vehicle operation (excluding shoulders) normally consists of two or more surfaced traffic lanes. The impact of providing adequate lane widths is wide ranging and includes maintaining and/or enhancing driver safety, driver comfort, the level of service, capacity, and the frequency and extent of shoulder and pavement surface maintenance.

For all new construction and reconstruction on arterial and collector roads, the desirable surfaced travel lane width is 12 ft [3.6 m]. If the scope of work is limited, speeds are low, truck volumes are light or there are no defined safety problems, surfaced lane widths of 11 ft [3.3 m] may be acceptable, particularly in urbanized areas with restricted right-of-way and increased pedestrian activity. However, for urban arterials with higher speeds, predominantly free-flowing conditions, and higher traffic volumes, surfaced lane widths of 12 ft [3.6 m] are desirable. For local roads and streets, surfaced traffic lanes normally should be 11 feet [3.3 m] wide but AASHTO allows lane widths of 9 [2.7 m] or 10 ft [3.0 m] where there is restricted or low truck use, low traffic volumes and low operating speeds. See Section 3.2.1 for information on selecting lane and shoulder widths.

For pavements on new construction or major reconstruction projects with existing or projected high concentrations of truck traffic, a wider pavement provides more edge strength and has been found to be structurally better for heavy loads. Consideration should be given to widening the pavement an additional 2 ft [0.6 m] under these circumstances. The lanes should be striped for 12 ft [3.6 m] lanes to keep trucks away from the edge of the pavement. The extra width can be considered part of the shoulder. If the mainline and shoulders are constructed of Portland cement concrete and the shoulders are structurally tied to the mainline, this additional width is not normally necessary. For divided highways, the widening should be adjacent to the outside shoulder; on two-lane roadways the widening should be equally divided on each side.

3.3.3 SHOULDER WIDTH

The total shoulder width is the distance from the edge of the traffic lane to the intersection of the shoulder slope with the front slope, or to the face of curb. In sections without curbs there are two terms used to describe the shoulder area. The “graded” width of shoulder is that measured from the edge of the traveled way to the intersection of the shoulder slope and the front slope. The “useable” width of shoulder is the actual width that can be used when a driver makes a stop.

Having a sharp break at the point of intersection of the edge of the graded shoulder and the front slope is not a good practice. Instead a rounding of 4 to 6 ft [1.2 to1.8 m] with a front slope 4:1 or flatter is the best practice. This rounding improves the general safety of the roadside by reducing the likelihood of encroachment, thus giving the errant driver more chance to regain control. Other considerations are that rounding may reduce rollovers and the possibility that the vehicle may become air-

borne. A portion of the rounding (usually one half) can be considered part of the "useable" width. Where front slopes are steeper than 4:1, the rounding should occur outside of the useable shoulder width.

Well-designed and maintained shoulders are necessary on rural highways with any appreciable traffic volumes. Shoulders and their widths are an important consideration in establishing a project's design standards. The benefits of including a shoulder include:

- Providing a refuge when a driver makes an emergency or parking stop.
- Providing lateral recovery areas for vehicles that inadvertently leave the traffic lane.
- Providing improved sight distance in cut areas.
- Providing areas for maintenance operations, including snow removal and storage.
- Providing for alternative modes of travel by pedestrians, bicyclist, joggers, transit operations, etc.
- Structurally improving the service life of the pavement by increasing the stability of the roadway's base and surfacing materials at the edge of the through traffic lane.
- Providing the opportunity for improved subgrade drainage designs.

Section 3.2.1 discusses the selection of shoulder widths. Normally shoulder widths of 10 ft [3.0 m] are used on new construction projects for arterial highways with relatively high traffic volumes. Where truck traffic exceeds 250 DHV it is desirable to have a paved shoulder width of 12 ft [3.6 m]. AASHTO allows narrower shoulder widths on most roadways with lower traffic volumes. However, wider shoulders widths should be provided on these projects when practical. Where bicyclists and pedestrians are to be accommodated on the shoulders, a minimum useable shoulder width, clear of any rumble strips, of 4 ft [1.2 m] should be used. On highways with three or more lanes in each direction a median shoulder width of 10 ft [3.0 m] is desirable. This provides a refuge area for disabled vehicles without affecting roadway capacity and flow.

3.3.4 SURFACED SHOULDER WIDTH

The surfaced shoulder width is that part constructed to provide better all-weather load support than is afforded by natural soils or stabilized materials. The paved portion of the shoulder also protects the edge of the traffic lane pavement from deterioration and raveling. More discussion on shoulder surfacing is in Chapter Nine - Pavement Selection. Normally the shoulder's structural design, including surface material, is recommended by the Materials and Research Section.

3.3.5 SIDE SLOPES

Various cross section slopes are identified in Figure 3-7. Four of these slopes are described below.

- **Front Slope.** The slope extending outward and downward from the shoulder to the ditch line.
- **Back Slope**. The slope extending upward and outward from the ditch line to intersect the natural ground.
- **Fill Slope**. The slope extending outward and downward from the shoulder to intersect with the natural ground; it may include a ditch section.
- **Cut Slope**. The slope extending outward and upward from the shoulder, intersecting the ditch slope and then extending upward from the ditch back slope to natural ground.

It is generally desirable that these slopes be 6:1 or flatter. Often, from a practical standpoint, they must be steeper. There is a distinct relationship between the steepness of side slopes, operating speeds and the desirable widths of clear zones. Chapter Four discusses

the relationship of these slopes to establishing a roadway's clear zone. General criteria for side slopes are presented in Figure 4-4, in terms of both desirable slopes and maximum slopes. The desirable slopes should be provided wherever feasible.

3.3.6 HORIZONTAL CLEARANCE AND CLEAR ZONE

3.3.6.1 HORIZONTAL CLEARANCE

Horizontal clearance is the lateral distance from edge of traveled way to a roadside feature or object for a roadway with barrier curb. Roadways having curbed sections should be provided with a minimum horizontal clearance of 1.5 ft [0.5 m] beyond the face of curb, with wider offsets (if possible to the full clear zone width) provided where practical since most types of curbs provide little help in redirecting an errant vehicle. Please see the Green Book for more information regarding horizontal clearance and AASHTO's Roadside Design Guide for more information regarding the clear zone width. If the minimum horizontal clearance cannot be provided in curbed areas, then a design exception is required.

3.3.6.2 CLEAR ZONE

The clear zone is defined in AASHTO's Roadside Design Guide as "the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry." This border area includes any shoulders or auxiliary lanes. Adequate lateral clearance between the edges of traffic lanes and roadside obstructions has been shown to be a very important safety factor. Vehicles leaving the roadway should have a reasonable opportunity to recover control and return to the roadway without overturning or colliding with roadside obstacles such as trees, poles, headwalls or other large objects. The combination of a relatively flat slope and an obstacle-free roadside within the prescribed clear zone helps this situation.

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the safest available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-5), the Design Criteria Form (Figure 3-6) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there

are steep grades accompanied by considerable no-passing distances.

More detailed guidelines and criteria for the design of grades, including critical lengths of grades and minimum and maximum grades are presented in Chapter Five - Alignment and Superelevation and the Green Book, pages 231-250. The maximum grades should be used infrequently, only as dictated by severe terrain conditions. When it is necessary to use maximum grades, the designer should check other design criteria and roadside features that may be improved to minimize the impact of using the higher design grade.

3.3.8 BRIDGES

The designer should coordinate with the Bridge Design Section when determining vertical clearances. A minimum vertical clearance for roads over interstate, U.S. and state routes is 16.5 ft [5 m]. Pedestrian bridges and overhead sign structures must have an extra 1 foot [0.3 m] of clearance, a total of 17.5 ft [5.3 m]. These clearances allow for a 4 in. [100 mm] future resurfacing.

3.3.9 MEDIANS

Geometric criteria for medians on multilane divided highways are discussed in Chapter Four.

There are several project types described in the following sections. Figure 3-2 shows the types of construction that fall under each project type. The type of construction shall be shown on the title sheet of the construction plans.

Figure 3-2
Types of Construction

Project Type	Type of Construction
New Construction and Reconstruction	Bridge Replacement
	New Construction (Road/Bridge)
	4-R
Intermediate	Bridge Rehabilitation
	Bridge Superstructure Replacement
	Intersection Improvements
	Safety Improvements
	Traffic Calming
Preventive Maintenance	Bridge Painting
	Bridge Preservation
	Bridge Preventive Maintenance
	Community Transportation Fund
	Pavement Preservation
	Scour Countermeasures
Miscellaneous Improvement	Advanced Utility
	Bike and Pedestrian Improvements
	Drainage Improvements
	ITS
	Landscaping
	Lighting
	Railroad Crossing
	Signal
	Signing and Striping
	Small Structure Repair
	Transportation Enhancement
	Miscellaneous Improvements (e.g., Wetland Mitigation, DTC Rail, Facilities)

3.1.3.1 NEW CONSTRUCTION AND RECONSTRUCTION PROJECTS

New construction and reconstruction projects on the interstate system and the NHS are expected to be in conformance with the appropriate standards and exceptions should be rare. All projects on the NHS (except preventive maintenance) shall conform to the standards in the current edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book). All projects on the interstate system (except preventive maintenance) shall also meet the design criteria contained in the current edition of *A Policy on Design Standards—Interstate System.* Projects having full federal oversight by FHWA are established in the current *Stewardship and Oversight Agreement* between FHWA and DelDOT. FHWA must approve all design exceptions for projects with full federal oversight.

New construction and reconstruction projects require a formal design exception as described in "Required Design Exception Documentation" (Figure 3-3) to justify the rationale for departure from the established design standards. The Chief Engineer must approve design exceptions for all new construction and reconstruction projects. "Design Exception Request" (Figure 3-4) is a guide format for developing a design exception request. It is important that the designer complete the "Design Control Checklist" (Figure 3-5) and the "Design Criteria Form" (Figure 3-6) for all new construction and reconstruction projects.

Figure 3-3
Required Design Exception Documentation

Project Type	Design Exception Request Fig. 3-4)	Design Control Checklist Fig. 3-5)	Design Criteria Form Fig. 3-6)	Asst. Director, Design, and Chief Engineer Approval	FHWA Approval
New Construction or Reconstruction – Full Federal Oversight	X	X	X	X	X
New Construction or Reconstruction – State Administered	X	X	X	X	
Intermediate Level		X	X		
Preventive Maintenance		X			
Miscellaneous Improvement		X			

3.1.3.2 INTERMEDIATE PROJECTS

The scope of intermediate projects is improvements being retrofitted into the existing infrastructure. At times, it may be necessary to deviate from the design standards. It is important that the designer complete the "Design Control Checklist" (Figure 3-5) and the "Design Criteria Form" (Figure 3-6) for all intermediate projects. If design standards cannot be met due to site conditions, then it shall be noted on the Design Criteria Form. In addition, there must be a memo to the project file explaining the justification for the variances in the design standards signed by the Project Engineer, the Group Engineer and the Assistant Director. The level of documentation can vary depending on the project intent, environmental concerns and other issues. In most cases it is not necessary to provide the same level of documentation as provided in a formal design exception request because the project's scope has already been established. The following are examples of intermediate projects:

- Safety improvement projects
- Addition of channelizing islands with no reduction in existing lane or shoulder width
- Intersection improvements with no reduction in existing lane or shoulder width
- Minor lane or shoulder widening
- Traffic calming features, e.g. speed humps, chicanes, midblock median islands, chokers, or narrowed lanes. (Consult with FHWA for projects on NHS Routes.)
- Superstructure replacement
- Bridge rehabilitation

3.1.3.3 PREVENTIVE MAINTENANCE PROJECTS

Preventive maintenance includes rehabilitation or restoration of specific elements of a highway facility when such activities are a cost- effective means of extending the pavement or bridge life and shall not degrade any existing or geometric aspects of the facility. The majority of the work for these projects is between existing curb lines or outer edges of existing shoulders. These types of projects are not required to provide any documentation for design standards that cannot be met. A general list of preventative maintenance items is below:

- Pavement Preservation - Pavement milling and resurfacing of the same thickness with no reduction in lane widths, or pavement repair (e.g. sawing, sealing, pothole patching)
- Bridge Preservation
  - Bridge painting
  - Bridge Preventive Maintenance
    - Deck rehabilitation
    - Joint replacement or repair
    - Bearing replacement
    - Pile Jackets
  - Scour countermeasures

- Seismic retrofit

3.1.3.4 MISCELLANEOUS IMPROVEMENT PROJECTS

These projects are typically beyond the existing edge of pavement and are intended to improve safety and aesthetics. They may also include improvements within the roadway intended to improve safety which do not degrade the existing highway geometrics. These projects are not required to provide any documentation for design standards that cannot be met. A general list of miscellaneous improvement project work items is below:

- New or replaced curb and/or sidewalk
- Modifying sidewalk to comply with ADA requirements
- Roadside safety enhancements
  - New guardrail installation
  - Guardrail repair/replacement
  - New impact attenuators
  - Impact attenuator repair or replacement
  - Upgrading bridge rails
  - Removal of obstructions
- Median barrier replacement
- Drainage improvements
- Signing
- Small Structure Repair
  - Culvert lining
  - Retaining structure repair
- Large ground-mounted signs
- New sign structures (sign bridge, cantilever and bridge-mounted, provided they meet clearance requirements)
- Existing sign structure rehabilitation
- Striping with no additional lanes nor reduction in existing lane width
- New or replaced raised pavement markers
- New or upgraded signals
- ITS (e.g. fiber optic cable, message signs, cameras, emergency call boxes)
- New or upgraded lighting systems
- Fencing, provided existing stopping sight distance is not degraded
- Glare screens, provided existing stopping sight distance is not degraded
- Repair of structural components resulting from traffic impact
- Advanced utility relocation projects
- Landscape improvements

Figure 3-4
Design Exception Request

State Project No._____Federal-Aid Project No._____

Date: _____**Oversight Project: Yes _____No____**

Design Exception Abstract: (Provide a short summary detailing the nature of the exception, reasons for the request, etc.)

Note:
For all NHS projects, the thirteen controlling criteria to be met are design speed; through lane and auxiliary lane width; shoulder width; bridge width, structural capacity, horizontal alignment; vertical alignment; grades; stopping sight distance, cross-slope; superelevation; horizontal clearance; and vertical clearance.

RECOMMENDATION:

The purpose of this project is to----------.

The most effective method of addressing this is----------.

Based upon the conditions presented, it is recommended that a design exception be approved for the controlling substandard design element as justified.

Recommended By: _____
Squad Manager

Recommended By: _____
Group Engineer

Recommended By: _____
Assistant Director-Transportation Solutions

Recommended By: _____
Assistant Director-Design

Approved By: _____ **Date**: _____
Chief Engineer

Approved By: _____ **Date**: _____
Federal Highway Administration (NHS oversight projects only)

Enclosures: (Include design criteria, figures, calculations, etc. to document request.)

Figure 3-5
Design Control Checklist

PROJECT DATA

Squad Leader/Project Manager: _____

Project Title: _____

Contract No.: _____

Federal Aid Project No: _____

Project Limits: _____

Type of Construction:_____

Project Scope and Initial Estimate: _____

_____

_____

_____

DESIGN DATA

Functional Classification:_____

Current AADT (Year)_____

Projected AADT (Year):_____

Projected DHV (Year):_____

% Trucks_____

Directional Distribution (%):_____

Design Speed _____

Design Vehicle_____

Design Level of Service _____

Clear Zone_____

Recommended By: _____

Squad Manager

Recommended By: _____

Group Engineer

Recommended By: _____

Assistant Director-Transportation Solutions

Approved By: _____

Director-Transportation Solutions

Figure 3-6
Design Criteria Form

Design Criteria		
Design Factor	**As per Road Design Manual**	**Provided**
Design Speed*		
Width of Through Lanes*		
Width of Auxiliary Lanes*		
Width of Outside Shoulder*		
Width of Inside Shoulder*		
Cross Slope*		
Width of Median		
Stopping Sight Distance*		
Minimum Horizontal Curve Radius*		
Minimum K (Crest)*		
Minimum K (Sag)*		
Maximum % Grade*		
Maximum front slope (Unprotected Section)		
Maximum back slope		
Barrier Offset		
Superelevation Rate (%)*		
Bridge Width*		
Vertical Clearance*		
Structural Capacity*		
Horizontal Clearance *		
Width of clear zone		

General Notes:

- Use this form primarily for new construction or reconstruction projects.

* The Chief Engineer must approve design criteria deviating from the requirements of the Road Design Manual through the use Figure 3-4 "Design Exception Request."

Recommended By: _____
Project Manager

Recommended By: _____
Group Engineer

Approved By: _____
Assistant Director-Transportation Solutions

3.1.4 DETERMINATION OF STANDARDS

Figure 3-7 graphically defines the nomenclature used when describing the various elements that establish a roadway's cross section. The dimensions and geometrics needed to design this roadway section are known as a project's design standards. The basic information needed before beginning the process of establishing a project's design standards is:

- The ***functional classification*** of the road section to be improved. A roadway's classification is shown on DelDOT's current functional classification map but should be verified with the Division of Planning. A part of the initial field review and scoping meeting should be to verify that the area and roadway section being considered are truly representative of the designated classification.

- The ***scope of work*** proposed for the project under consideration is in the Project Initiation Form, project development documentation and other supporting data furnished to the designer.

- The ***traffic data*** for the road section is obtained from the Division of Planning and includes current traffic, projected traffic, percent trucks, accident history, etc.

With this information, the designer can proceed with the process of selecting the design speed.

3.2 STANDARDS BASED ON DESIGN SPEED

3.2.1 SELECTION OF DESIGN SPEED

The design speed establishes basic criteria for certain design elements. Two design standard considerations are related directly to the design speed:

- Curvature and superelevation, and
- Required sight distances.

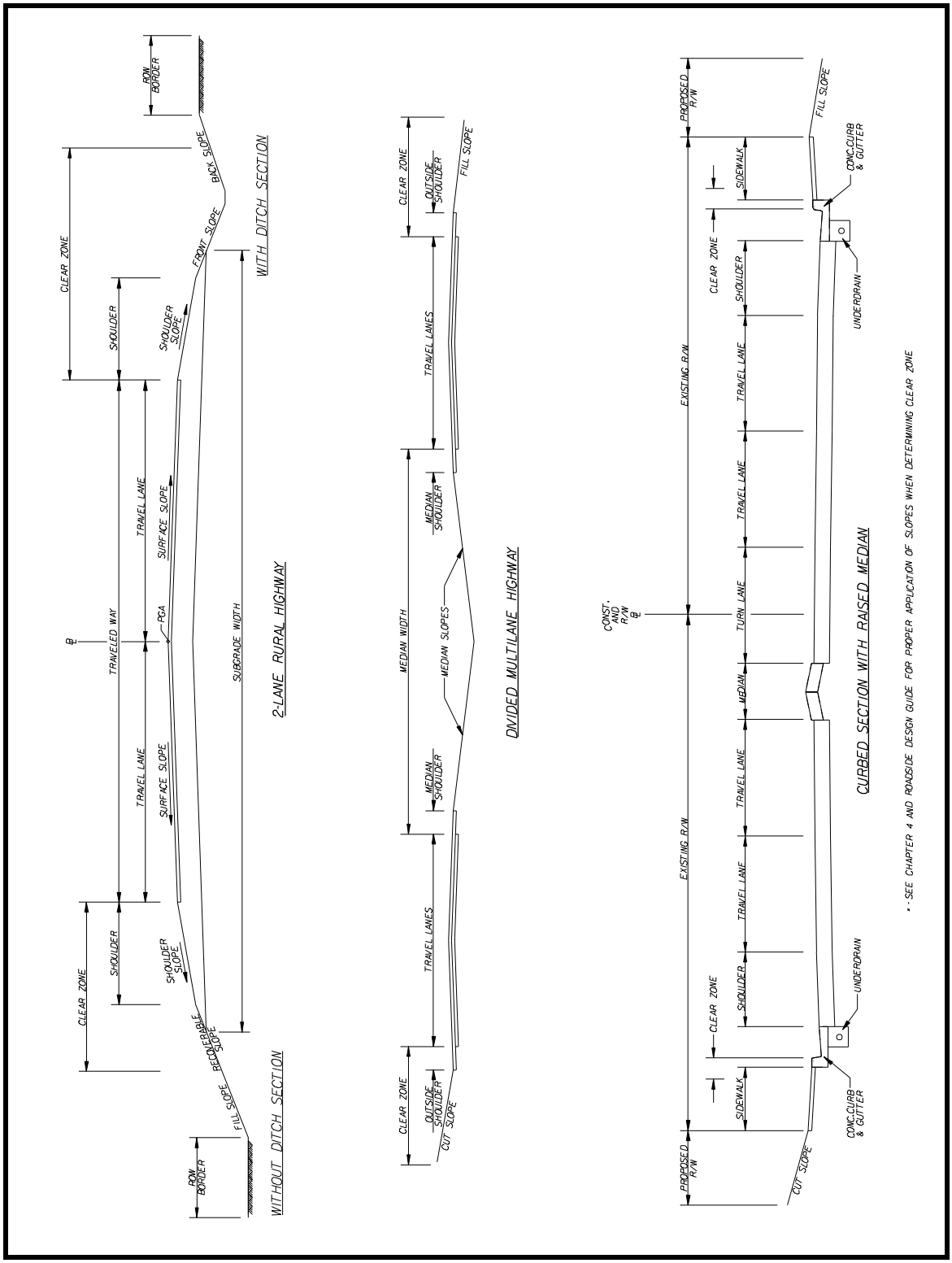
The designer's goal is to provide at least the minimum values, and preferably larger values, for these standards, regardless of traffic volumes, functional classification or any other consideration. These design elements are very closely related to traffic safety and should not be compromised.

A first step in determining the appropriate design standards is to establish a reasonable and realistic design speed. Since the majority of design controls are related to the design speed, this decision needs to be based on more factors than a roadway's functional classification and traffic volume.

The design speed selected should accommodate a high percentage of drivers, including the reasonable and prudent driver. Other considerations include topography, anticipated operating speeds, driver expectations, volume and mix of vehicles, the volume and type of non-vehicular traffic, driver familiarity, level of congestion reasonably acceptable to the motorists, and community values.

Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Figure 3-7
Typical Section Nomenclature



Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page 384, Exhibit 5-5;
- Collector Roads and Streets (Rural) — page 425, Exhibit 6-5;
- Collector Roads and Streets (Urban) — page 433;
- Arterials (Rural) — page 448, Exhibit 7-3;
- Divided Arterials (Rural) — page 455;
- Urban Arterials — page 472;
- Freeways — page 504.

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-6, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the

borne. A portion of the rounding (usually one half) can be considered part of the "useable" width. Where front slopes are steeper than 4:1, the rounding should occur outside of the useable shoulder width.

Well-designed and maintained shoulders are necessary on rural highways with any appreciable traffic volumes. Shoulders and their widths are an important consideration in establishing a project's design standards. The benefits of including a shoulder include:

- Providing a refuge when a driver makes an emergency or parking stop.
- Providing lateral recovery areas for vehicles that inadvertently leave the traffic lane.
- Providing improved sight distance in cut areas.
- Providing areas for maintenance operations, including snow removal and storage.
- Providing for alternative modes of travel by pedestrians, bicyclist, joggers, transit operations, etc.
- Structurally improving the service life of the pavement by increasing the stability of the roadway's base and surfacing materials at the edge of the through traffic lane.
- Providing the opportunity for improved subgrade drainage designs.

Section 3.2.1 discusses the selection of shoulder widths. Normally shoulder widths of 10 ft [3.0 m] are used on new construction projects for arterial highways with relatively high traffic volumes. Where truck traffic exceeds 250 DHV it is desirable to have a paved shoulder width of 12 ft [3.6 m]. AASHTO allows narrower shoulder widths on most roadways with lower traffic volumes. However, wider shoulders widths should be provided on these projects when practical. Where bicyclists and pedestrians are to be accommodated on the shoulders, a minimum useable shoulder width, clear of any rumble strips, of 4 ft [1.2 m] should be used. On highways with three or more lanes in each direction a median shoulder width of 10 ft [3.0 m] is desirable. This provides a refuge area for disabled vehicles without affecting roadway capacity and flow.

3.3.4 SURFACED SHOULDER WIDTH

The surfaced shoulder width is that part constructed to provide better all-weather load support than is afforded by natural soils or stabilized materials. The paved portion of the shoulder also protects the edge of the traffic lane pavement from deterioration and raveling. More discussion on shoulder surfacing is in Chapter Nine - Pavement Selection. Normally the shoulder's structural design, including surface material, is recommended by the Materials and Research Section.

3.3.5 SIDE SLOPES

Various cross section slopes are identified in Figure 3-7. Four of these slopes are described below.

- **Front Slope.** The slope extending outward and downward from the shoulder to the ditch line.
- **Back Slope**. The slope extending upward and outward from the ditch line to intersect the natural ground.
- **Fill Slope**. The slope extending outward and downward from the shoulder to intersect with the natural ground; it may include a ditch section.
- **Cut Slope**. The slope extending outward and upward from the shoulder, intersecting the ditch slope and then extending upward from the ditch back slope to natural ground.

It is generally desirable that these slopes be 6:1 or flatter. Often, from a practical standpoint, they must be steeper. There is a distinct relationship between the steepness of side slopes, operating speeds and the desirable widths of clear zones. Chapter Four discusses

the relationship of these slopes to establishing a roadway's clear zone. General criteria for side slopes are presented in Figure 4-4, in terms of both desirable slopes and maximum slopes. The desirable slopes should be provided wherever feasible.

3.3.6 HORIZONTAL CLEARANCE AND CLEAR ZONE

3.3.6.1 HORIZONTAL CLEARANCE

Horizontal clearance is the lateral distance from edge of traveled way to a roadside feature or object for a roadway with barrier curb. Roadways having curbed sections should be provided with a minimum horizontal clearance of 1.5 ft [0.5 m] beyond the face of curb, with wider offsets (if possible to the full clear zone width) provided where practical since most types of curbs provide little help in redirecting an errant vehicle. Please see the Green Book for more information regarding horizontal clearance and AASHTO's Roadside Design Guide for more information regarding the clear zone width. If the minimum horizontal clearance cannot be provided in curbed areas, then a design exception is required.

3.3.6.2 CLEAR ZONE

The clear zone is defined in AASHTO's Roadside Design Guide as "the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry." This border area includes any shoulders or auxiliary lanes. Adequate lateral clearance between the edges of traffic lanes and roadside obstructions has been shown to be a very important safety factor. Vehicles leaving the roadway should have a reasonable opportunity to recover control and return to the roadway without overturning or colliding with roadside obstacles such as trees, poles, headwalls or other large objects. The combination of a relatively flat slope and an obstacle-free roadside within the prescribed clear zone helps this situation.

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the safest available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-5), the Design Criteria Form (Figure 3-6) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
800 BAY ROAD
P.O. BOX 778
DOVER, DELAWARE 19903

CAROLANN WICKS, P.E.
SECRETARY

June 16, 2010

Dear Road Design Manual Holder:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual - June 2010 Revision

This is the formal issuance of the June 2010 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective June 28, 2010. You can download this revision from DelDOT's web site, http://www.deldot.gov/information/pubs_forms/manuals/road_design/index.shtml. The web site also includes the revision with changes highlighted for your reference. Changes have been made to a portion of Chapter 7 regarding islands and bicycle lanes.

Please replace the pages in your manual with the appropriate sheets. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely yours,

Kevin T. Canning
Quality Engineer

KTC:lob
cc: Natalie Barnhart, Chief Engineer
Michael Simmons, Assistant Director, Project Development, South Region
Drew Boyce, Assistant Director, Project Development, North Region
Linda M. Osiecki, Program Manager, Quality

DelDOT

Figure 7-7
Design Widths for Turning Roadways [Metric]

Radius on Inner Edge R [m]	Pavement width [m]								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 0.3 m Add 0.6 m			None Add 0.3 m			Add 0.3 m Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width; minimum width as under Case I			Deduct 0.6 m where shoulder is 1.2 m or wider		

Note:

Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination trucks.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Certain principles should be followed in the design of a channelized intersection, but the extent to which they are applied will depend on the characteristics of the total design plan. These principles are:

- Motorists should not be confronted with more than one decision at a time.
- Unnatural paths that require turns greater than 90 degrees or sudden and sharp reverse curves should be avoided.
- Areas of vehicle conflict should be reduced as much as possible. Channelization should be used to keep vehicles within well-defined paths that minimize the area of conflict.
- The points of crossing or conflict should be studied carefully to determine if such conditions would be better separated or consolidated to simplify design with appropriate control devices added to ensure safe operation.
- Refuge areas for turning vehicles should be provided clear of through traffic.
- Prohibited turns should be blocked wherever possible.
- Location of essential control devices should be established as a part of the design of a channelized intersection.
- Channelization may be desirable to separate the various traffic movements where multiple-phase signals are used.

7.3.3 ISLANDS

Design of islands is the principal concern in channelization. An island is a defined area between traffic lanes for control of vehicle movements. It may range from an area delineated by barrier curbs to a pavement area marked with paint.

Islands provide three major functions:

- **Channelizing islands**–designed to control and direct traffic movement, usually turning;
- **Divisional islands**–designed to divide opposing or same-direction traffic streams, usually through movements; and
- **Refuge islands**–to provide refuge for pedestrians and bicyclists.

Most islands combine two or all of these functions. Islands may be delineated or outlined by a variety of treatments, depending on their size, location and function. Types of delineators include: (1) raised islands outlined by curbs, (2) islands delineated by pavement markings, and (3) non-paved areas formed by the pavement edges–possibly supplemented by delineators on posts or other guide posts.

Islands should be sufficiently large to command attention, and to accommodate pedestrian refuge and pedestrian signal poles where they are needed. Curbed islands normally should be no smaller than 50 ft^2 [5 m^2] for urban streets and about 75 ft^2 [7 m^2] for rural intersections; however, 100 ft^2 [9 m^2] minimum is preferable for both. Triangular islands should not be less than 12 ft [3.6 m], preferably 15 feet [4.5 m], on a side before rounding the corners; those with five foot wide curb ramps, pedestrian refuge and pedestrian signal poles should have sides that are at least 15 ft [4.5 m] and preferably 20 feet [6.0 m] on a tangent side resulting in a minimum island area of 175 ft^2 [16 m^2]. Islands with pedestrian signals and curb ramps wider than five feet will have to be larger accordingly. Median islands narrower than 8 ft [2.4 m] from back of curb to back of curb cannot be mowed effectively; therefore they should be paved. Elongated or divisional islands should not be less than 4 ft [1.2 m] wide, 6 ft [1.8 m] if pedestrians are anticipated and 20 to 25 ft [6.0 to 8.0 m] long.

DelDOT has adopted general rules for the placement of islands. The first preference is to design the intersection with radii that accommodate the selected design vehicle path with-

out creating large open paved areas requiring the need for islands to direct traffic. The overriding criteria are operational efficiency and safety. In many locations the best way to accomplish this is by the use of islands. Curbed islands usually provide the better alternative to delineate the location, minimize maintenance and control drainage. Due to the higher speeds associated with rural areas, using the proper type of curb and offsets are important design considerations. Full shoulder width offset and P.C.C. Type 2 are the preferred design elements. In locations where it is necessary to place the island closer to the traveled way, a 5 ft [1.5 m] offset is to be maintained to accommodate through movement of bicyclists.

Urban intersections will normally have curbed shoulders, parking lanes or right turn lanes on the roadway cross section adjacent to the island location. In these locations there are several alternatives for placing islands depending upon the need to accommodate bicyclists and pedestrians as well as higher traffic volumes with more complicated traffic patterns. Normally, to accommodate bicyclists, the face of curbed islands will be offset a minimum of 5 ft [1.5 m] from adjacent curb lanes. In areas with high pedestrian traffic where there is a need to minimize the distance between refuge areas, the offset for bicycles may be reduced to 4 ft [1.2 m] from an approaching curbed section. In this case, it is preferable to offset the nose of larger islands 4 to 6 ft [1.2 to 2 m] from approaching curbed lanes to allow an errant vehicle to recover before striking the curb. To accommodate the variety of vehicle, pedestrian and bicycle movements in urban areas various offsets and configurations may be needed within the same intersection. The preferred curb type for islands is mountable P.C.C. Type 2.

Figures 7-8 and 7-9 show a typical preferred island layout for rural and urban conditions.

Figure 7-8
Typical Island Layout–Rural Areas

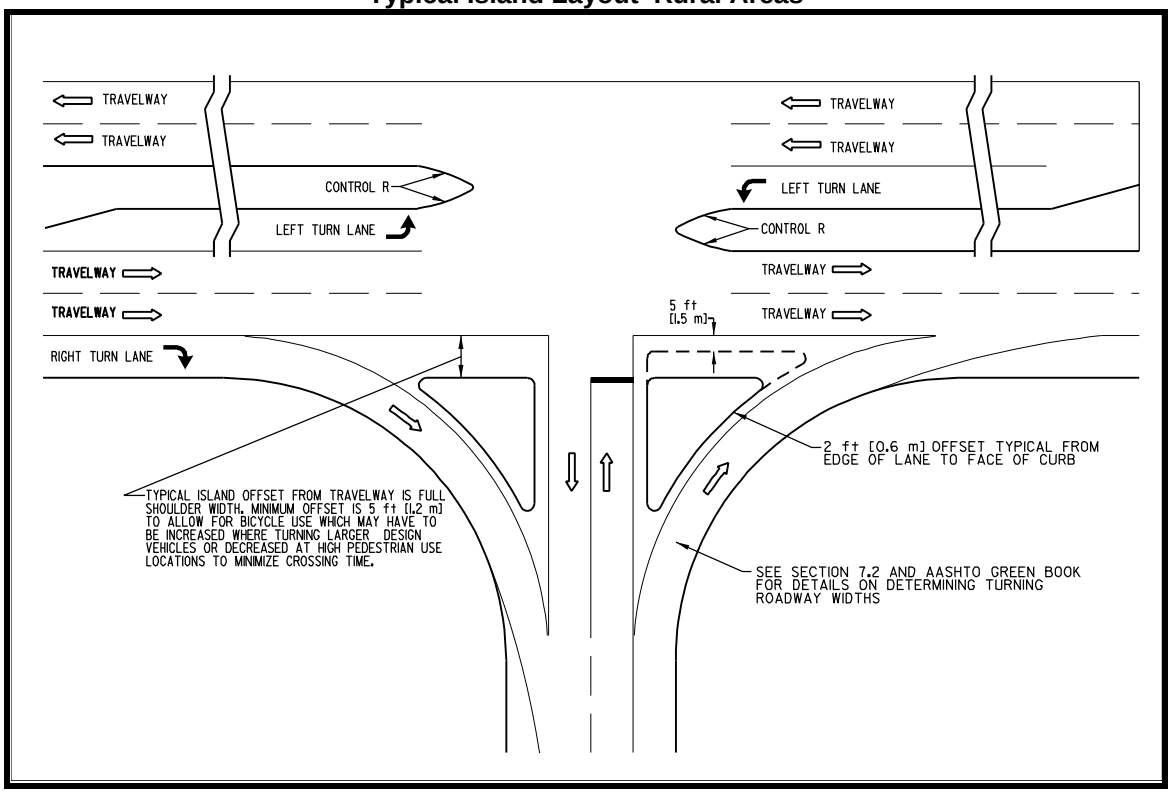
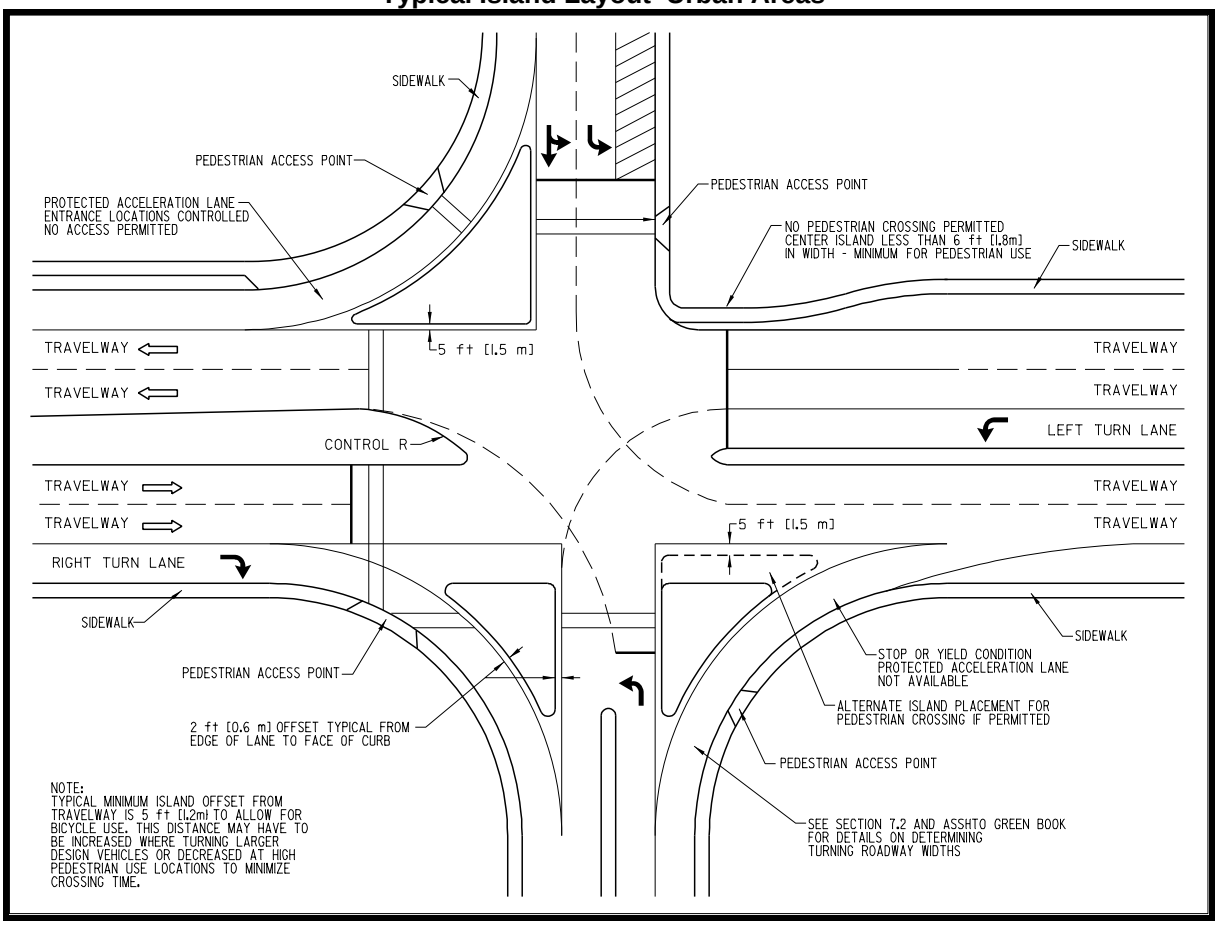


Figure 7-9
Typical Island Layout–Urban Areas



.../DGN/New Figure 7-09.dgn 06/12/2003 11:07:24 AM

7.4 SIGHT DISTANCE

The operator of a vehicle approaching an at-grade intersection should have an unobstructed view of the whole intersection and of a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions which is termed "approach sight distance". (See Figure 7-10.) The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and the resultant distances traversed during perception, reaction time, and braking. In addition to approach sight distance, sight distance is also provided to allow stopped vehicles sufficient view of the intersecting roadway to decide when to enter the intersecting roadway or to cross it, which is "departure sight distance." Both sight distances must be checked on all intersection designs based on the procedures set forth in the Green Book, pages 650 to 679.

7.4.1 MINIMUM SIGHT DISTANCE TRIANGLE

Sight triangles are areas of unobstructed sight along both roads at an intersection and across their included corner for a distance sufficient to allow the operators of vehicles approaching simultaneously to see each other in time to prevent collision at the intersection.

The length of the legs may vary based upon traffic volumes, design speeds, operating speeds and type of intersection traffic control.

Certain principles should be followed in the design of a channelized intersection, but the extent to which they are applied will depend on the characteristics of the total design plan. These principles are:

- Motorists should not be confronted with more than one decision at a time.
- Unnatural paths that require turns greater than 90 degrees or sudden and sharp reverse curves should be avoided.
- Areas of vehicle conflict should be reduced as much as possible. Channelization should be used to keep vehicles within well-defined paths that minimize the area of conflict.
- The points of crossing or conflict should be studied carefully to determine if such conditions would be better separated or consolidated to simplify design with appropriate control devices added to ensure safe operation.
- Refuge areas for turning vehicles should be provided clear of through traffic.
- Prohibited turns should be blocked wherever possible.
- Location of essential control devices should be established as a part of the design of a channelized intersection.
- Channelization may be desirable to separate the various traffic movements where multiple-phase signals are used.

7.3.3 ISLANDS

Design of islands is the principal concern in channelization. An island is a defined area between traffic lanes for control of vehicle movements. It may range from an area delineated by barrier curbs to a pavement area marked with paint.

Islands provide three major functions:

- **Channelizing islands**–designed to control and direct traffic movement, usually turning;
- **Divisional islands**–designed to divide opposing or same-direction traffic streams, usually through movements; and
- **Refuge islands**–to provide refuge for pedestrians and bicyclists.

Most islands combine two or all of these functions. Islands may be delineated or outlined by a variety of treatments, depending on their size, location and function. Types of delineators include: (1) raised islands outlined by curbs, (2) islands delineated by pavement markings, and (3) non-paved areas formed by the pavement edges–possibly supplemented by delineators on posts or other guide posts.

Islands should be sufficiently large to command attention, and to accommodate pedestrian refuge and pedestrian signal poles where they are needed. Curbed islands normally should be no smaller than 50 ft^2 [5 m^2] for urban streets and about 75 ft^2 [7 m^2] for rural intersections; however, 100 ft^2 [9 m^2] minimum is preferable for both. Triangular islands should not be less than 12 ft [3.6 m], preferably 15 feet [4.5 m], on a side before rounding the corners; those with five foot wide curb ramps, pedestrian refuge and pedestrian signal poles should have sides that are at least 15 ft [4.5 m] and preferably 20 feet [6.0 m] on a tangent side resulting in a minimum island area of 175 ft^2 [16 m^2]. Islands with pedestrian signals and curb ramps wider than five feet will have to be larger accordingly. Median islands narrower than 8 ft [2.4 m] from back of curb to back of curb cannot be mowed effectively; therefore they should be paved. Elongated or divisional islands should not be less than 4 ft [1.2 m] wide, 6 ft [1.8 m] if pedestrians are anticipated and 20 to 25 ft [6.0 to 8.0 m] long.

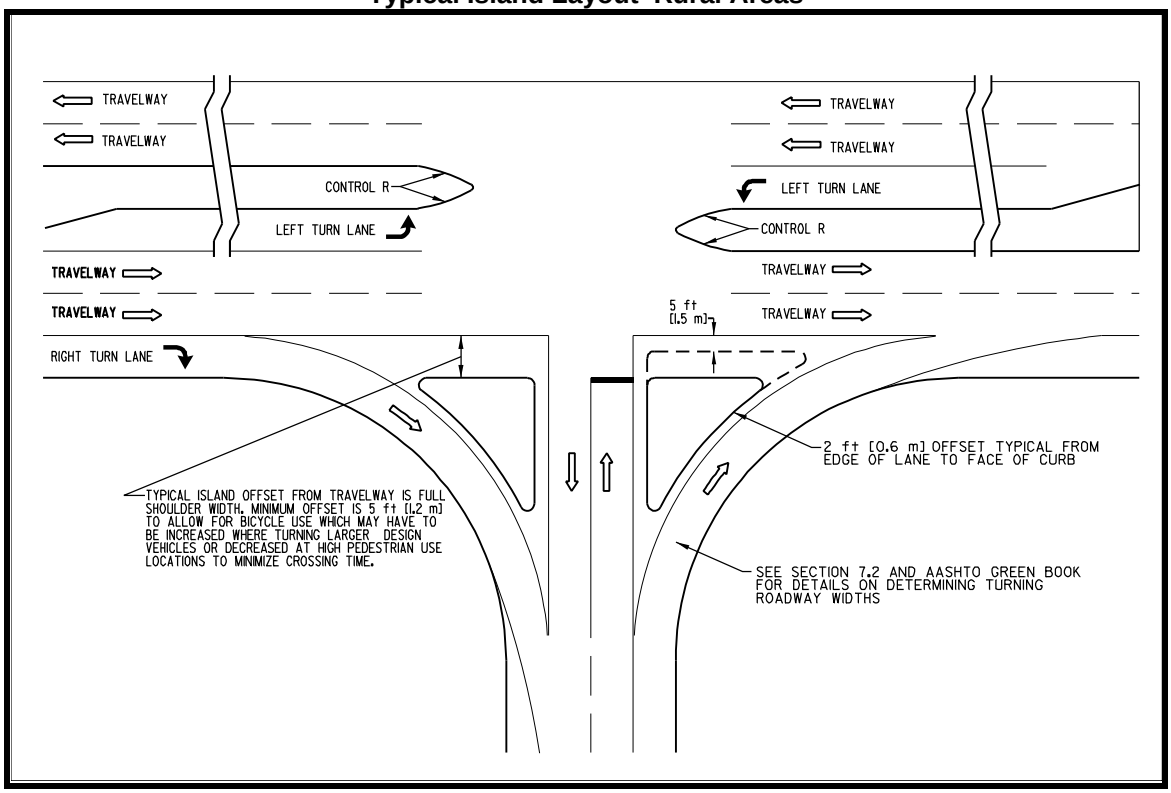
DelDOT has adopted general rules for the placement of islands. The first preference is to design the intersection with radii that accommodate the selected design vehicle path with-

out creating large open paved areas requiring the need for islands to direct traffic. The overriding criteria are operational efficiency and safety. In many locations the best way to accomplish this is by the use of islands. Curbed islands usually provide the better alternative to delineate the location, minimize maintenance and control drainage. Due to the higher speeds associated with rural areas, using the proper type of curb and offsets are important design considerations. Full shoulder width offset and P.C.C. Type 2 are the preferred design elements. In locations where it is necessary to place the island closer to the traveled way, a 5 ft [1.5 m] offset is to be maintained to accommodate through movement of bicyclists.

Urban intersections will normally have curbed shoulders, parking lanes or right turn lanes on the roadway cross section adjacent to the island location. In these locations there are several alternatives for placing islands depending upon the need to accommodate bicyclists and pedestrians as well as higher traffic volumes with more complicated traffic patterns. Normally, to accommodate bicyclists, the face of curbed islands will be offset a minimum of 5 ft [1.5 m] from adjacent curb lanes. In areas with high pedestrian traffic where there is a need to minimize the distance between refuge areas, the offset for bicycles may be reduced to 4 ft [1.2 m] from an approaching curbed section. In this case, it is preferable to offset the nose of larger islands 4 to 6 ft [1.2 to 2 m] from approaching curbed lanes to allow an errant vehicle to recover before striking the curb. To accommodate the variety of vehicle, pedestrian and bicycle movements in urban areas various offsets and configurations may be needed within the same intersection. The preferred curb type for islands is mountable P.C.C. Type 2.

Figures 7-8 and 7-9 show a typical preferred island layout for rural and urban conditions.

Figure 7-8
Typical Island Layout–Rural Areas



STATE OF DELAWARE
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SHAILEN P. BHATT
SECRETARY

October 3, 2011

Dear Road Design Manual Holders:

SUBJECT: The Delaware Department of Transportation 2004 Road Design Manual July 2011 Revision

This is the formal issuance of the July 2011 Revision to the Delaware Department of Transportation's 2004 Road Design Manual. This revision will be effective October 14, 2011. You can download this revision from DelDOT's website http://www.deldot.gov/information/pubs_forms/manuals/road_design/index.shtml. The website includes the revision with changes highlighted for your reference.

Please replace the pages in your manual with the appropriate sheets. If you should have any questions, please contact Linda Osiecki at (302) 760-2342.

Sincerely,

Barry Benton
Quality Engineer

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DelDOT

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slope for adequate cross drainage than is required for gravel or a coarse-textured, intermediate-type asphalt surface. The Department has adopted the cross-slope values shown in Figure 4-1 for standard practice on tangent sections of highways.

The range of cross slope values for projects other than "new construction" permits slight variations in the slope where the scope of work is primarily resurfacing. If surface drainage is a problem, cross slopes up to 2.5% on high-type pavements may be justified. However, new construction slopes should be provided wherever practical. For two-lane highways or multi-lane undivided highways, the cross slope normally goes downward both ways from a crown point at the highway center line.

On divided highways, each one-way pavement may be crowned separately, as on two-lane highways, or each may have a one-direction cross slope across the entire width of pavement-usually downward to the outer edge. Surface drainage on roadways with three or more lanes in one direction can cause problems if the pavement slopes uniformly in one direction at the rate of slope recommended for one- and two-lane roadways. The designer has two options for cross slopes on a three-lane roadway: (1) slope the inside lane to the median and the other two lanes to the outside, or (2) slope all three lanes to the outside and increase the slope of the outside lane.

A cross section with each roadway crowned separately, such as the first option above, has an advantage in rapidly draining the pavement during rainstorms. Disadvantages are that more inlet and underground drainage lines are required, and treatment of at-grade intersections is more difficult because of several high and low points on the cross section. Sections having no curbs and a wide depressed median are particularly well suited for this design. With a crowned section, cross slopes should not exceed 2% because the rollover effect, when changing lanes, is then 4%. "Rollover" is the algebraic difference between the two slopes.

Roadways that slope in only one direction are more comfortable to drivers because vehicles tend to be pulled in the same direction when changing lanes. This design is generally desirable for divided highways with a narrow curbed median. The cross slope of the third lane (outside lane) of a three-lane roadway where the cross slope is all one direction should be increased by 0.5% to 1.0% to improve surface drainage.

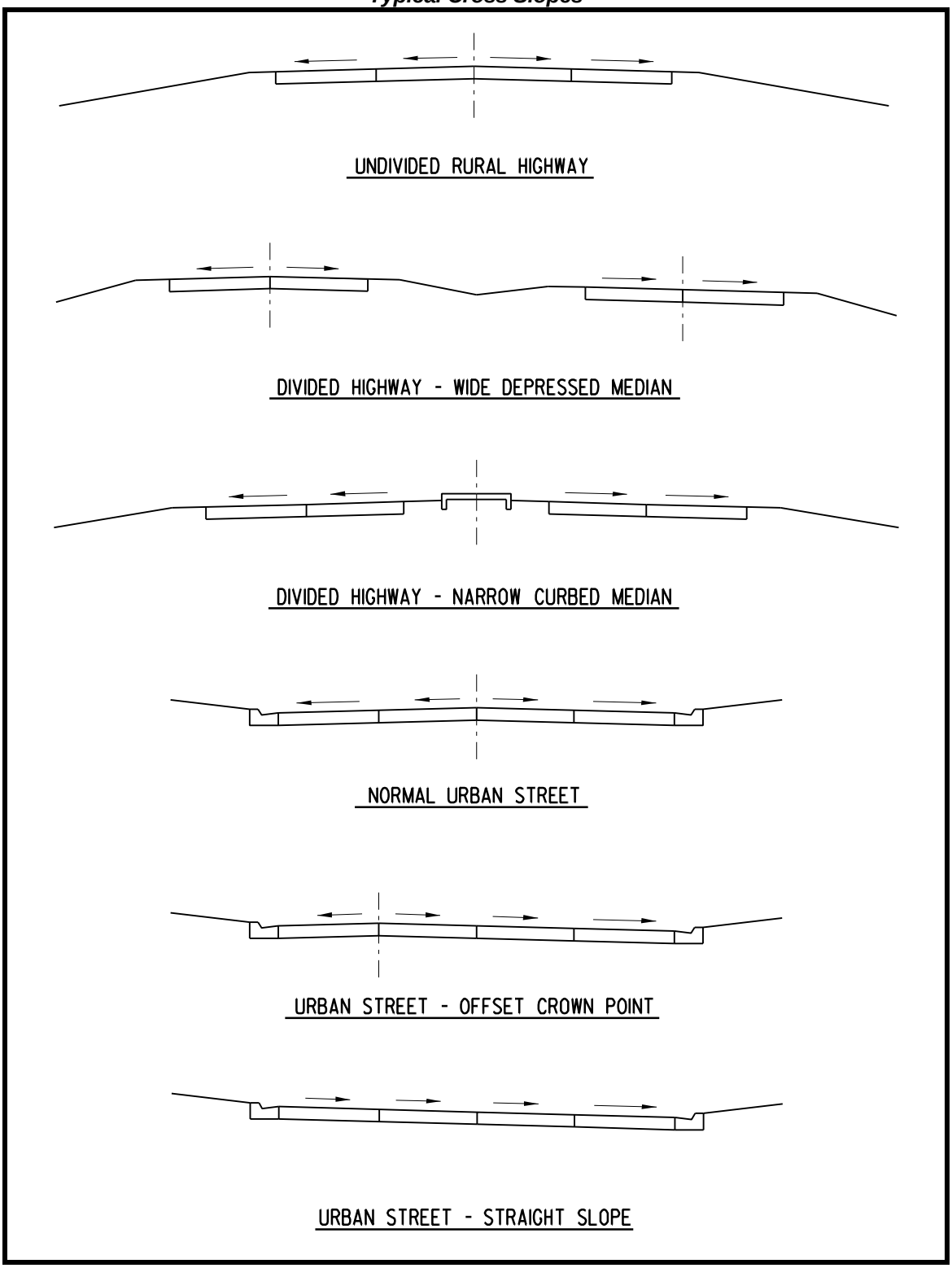
In the design of urban highways and streets, it may sometimes be found that adjacent property developments dictate that the curb on one side must be higher than the curb on the other. Two options are available. The cross slope can be in one direction for the full width of the street, or the crown point can be offset from the centerline toward the high side of the street. The latter option usually is preferable with the offset crown point corresponding to an edge of travel lane, out of the wheel path and with a maximum of 4% rollover.

Typical cross slope designs are illustrated in Figure 4-2.

4.1.5 SHOULDER CROSS SECTIONS

Shoulders should be flush with the roadway surface and should abut the edge of the traffic lane. All shoulders, including median shoulders on divided highways, normally should be sloped to drain away from the traveled way. However, in the case of a raised narrow median, the median shoulders may slope in the same direction as the traffic lanes, but consideration should be given to sloping the shoulders toward the median and providing inlets and underground drainage to alleviate problems with snow and ice. Slightly sloping shoulders steeper than the traffic lanes assure rapid surface drainage, reduce the chance of ponding, and minimize subgrade penetration of moisture through the edge joint. Paved shoulders normally should slope at a rate of 4%, and unpaved shoulders should be sloped at a rate of 6%.

Figure 4-2
Typical Cross Slopes



Special attention must be given to shoulder slopes in relation to superelevation on curves. Shoulder slopes that drain away from traffic lanes on the outside (high side) of a superelevated curve should be designed to avoid too great a cross slope break at the pavement edge. The rollover should not exceed 8%; differences greater than this tend to pull the vehicle toward the shoulder and may result in difficulty for the driver to regain control. For example, with a superelevation rate of 6% and a shoulder slope of 4%, the rollover would equal 10% which is unacceptable. The shoulder slope should be reduced to 2% but not less than 1% along the high side of the curve; this is acceptable since there is no stormwater discharge to the shoulder from the pavement and there is little opportunity for ponding or shoulder erosion damage.

Standard shoulder slopes should be used on the inside (low side) of superelevated curves unless the rate of superelevation exceeds the rate of normal shoulder slope. In this case, the shoulder slope should be the same as the superelevation slope.

4.1.5.1 GRASS SHOULDER

Vehicles often drift off the roadway and cause rutting of the shoulder edge. This is particularly noticeable at intersections and on the inside of sharp curves. Several options of grass shoulders are available. Vehicles may also park on the grass shoulder. Grass shoulders should be used only for special project needs. A grass shoulder shall not be used directly under a bridge overpass. Consider the following:

1. Use inground pavers, geogrid or a combination, which provide stability and allow grass to grow up through them. This is suitable for the area where parking is anticipated. Consider this where trash pick-up or mail delivery could rut the shoulder.
2. Provide a two foot wide paved shoulder adjacent to the grass shoulder, which may or may not be stabilized.
3. Provide a pavement edge line to determine the edge of the travel lane.
4. In a heavily salted area, alkaligrass may be used. Contact the Roadside Environmental Administrator.
5. The time of year for seeding and the length of time after seeding before the shoulder is subject to traffic are important. Phasing work should provide time for growth after seeding, and the shoulder should be protected until the grass is firmly established.

4.1.6 CURBS

Curbs are closely related to other surfacing cross section elements. They generally serve several purposes including drainage control, pavement edge delineation, delineation of pedestrian walkways, and control of entrances to roadside development. Curbs are used extensively on various types of urban highways and streets. In the interest of safety, curbs should be omitted on high-speed rural highways when the same objectives can be attained by other acceptable means. Curbs may be considered an obstruction, increase project cost and design effort. When using curbs, positive drainage of paved areas, particularly the traveled way, is necessary. This normally requires the installation of a closed drainage system with drainage inlets, positive outfalls and extensive ditching. Therefore, the need and use of curbs should be given appropriate study.

Curbs may be designed as a separate unit or integrally with the pavement structure. Separate curbs usually are a combination curb and gutter. Sometimes the curb is constructed alone without the gutter section.

The two general classes of curbs are barrier curbs and mountable curbs. Barrier curbs tend to, but do not always, prevent vehicles from crossing the curb line; mountable curbs permit such vehicle crossings without much difficulty. The types of curbs used most commonly are in DelDOT's *Standard Construction Details*. Refer to Chapter Ten for criteria for curb installations.

4.2 GRADING CROSS SECTION

The geometric elements of the grading cross section include the width and shape of the graded roadbed that consists of either suitable natural material or specified imported material. The top surface of the roadbed soil is defined as the subgrade. The pavement structure is placed on the prepared roadbed. The pavement structure includes any required selected subbase materials, base materials and the various layers of paving courses. The subgrade includes the various cut and fill slopes related to grading operations, including side ditches, to prepare a surface for constructing the pavement structure.

4.2.1 SUBGRADE CROSS SLOPES

The cross slope of the bottom of pavement box (top of subgrade) should parallel the cross slopes of the finished traffic lanes for the full width of the roadbed, including shoulders. This allows the pavement structure to drain through the porous material into side ditches or, if necessary, an underdrain system. The subgrade slope should not be broken to parallel the steeper finished shoulder slope. The parallel relationship between the subgrade and the finished traffic lanes applies to both normal crown slopes on tangent sections of highways and superelevated sections on curves.

4.2.2 SUBGRADE WIDTH

The design width of the subgrade should be shown on the roadway typical section(s). The total subgrade width should be the sum of the widths required for travel lanes, shoulders, median area, and any side slopes necessary to meet the subgrade.

For ease of computation and field staking, the design width may be rounded off to the nearest 1 ft [0.3 m]. This will result in a slope from the outside of the finished shoulder to the subgrade shoulder slightly different from the slope prescribed by the standards, but the variations will be negligible.

4.2.3 SUBGRADE WIDENING FOR GUARDRAIL

The subgrade should be widened at locations where guardrail is to be installed. The purpose is to provide the required horizontal clearance from the edge of the normal shoulder to the face of the guardrail, ensure the stability of the guardrail posts when placed in the embankment, and reduce maintenance.

Normally, 6 feet [1.8 m] of widening is required. This width includes 2 feet [0.6 m] from the normal shoulder line to the face of rail and 4 feet [1.2 m] behind the face of rail to a newly established edge of shoulder (the point of intersection of the front slope with the stabilized shoulder subgrade). Widening requirements are shown in DelDOT's *Standard Construction Details* for guardrail including special details for tapered flares for end treatments.

Refer to Chapter Ten for criteria for guardrail installations for various conditions relating to high embankments, non-traversable hazards and bridge ends.

4.2.4 SIDE SLOPES

A roadway's cross section includes side slopes as illustrated and identified in Figure 4-3. Side slopes are important in maintaining the stability of the roadbed and pavement structure as well as providing an area for the safety of errant vehicles. Side slopes are constructed in both fill (embankment) areas (those falling above the natural ground level) and cut areas (those falling below the natural ground level). As a general reference, slopes in embankment areas are commonly referred to as fill slopes or front slopes. When it is determined that no parallel ditch section is needed the front slope is graded to meet natural ground. In cut areas, side slopes are referred to as front slopes and back slopes, the back slope being necessary to bring the roadway cross section back up to meet the natural ground level. Ditch sections included as part of either fill or cut sections have a front slope, a ditch bottom with a defined shape and width, and a back slope. Criteria for rates of

these slopes (by road classes) are shown in Figure 4-4. The application of the criteria is very important in selecting a safe cross section. This application is discussed in this section; a full understanding of the concepts presented in AASHTO's *Roadside Design Guide* is critical to the proper application of the criteria.

All three slopes depend upon a lateral horizontal area measured from the edge of outside travel lane, called "clear zone". Consideration must be given to the lateral clear zone requirements when applying the criteria in selecting all side slope and ditch sections for the design cross section.

4.2.4.1 SIDE SLOPES WITHIN THE CLEAR ZONE

A roadway's "clear zone" is the total roadside border area, starting at the edge of the inside traveled way, that is considered available for safe use by errant vehicles. In addition to any shoulder area, the clear zone area may consist of a combination of a recoverable slope, a non-recoverable slope, a traversable slope, a clear run-out area and a critical slope. These slopes are defined as follows:

- A recoverable slope is flatter than 4:1 and an errant driver has a high probability of being able to recover control of the vehicle.
- Non-recoverable slopes are embankment areas with slope ratios from 3:1 to 4:1 on which the vehicle will continue to the bottom of the slope.
- A traversable slope has a slope ratio between 3:1 and 4:1. Slopes in this range, if properly graded and clear of obstructions, will not allow the driver to recover control and steer back onto the roadway but will permit the vehicle to slow down and stop safely.
- A clear runout area follows a non-recoverable slope and is graded, shaped and made free of hazards (traversable) wide enough to allow an errant vehicle to safely stop.
- Critical slopes have a slope ratio of 3:1 or steeper and will require barrier treatment to protect an errant vehicle

Please note that in the Department's presentation of slope ratios it uses horizontal to vertical while the *Roadside Design Guide* uses the ratio of vertical to horizontal, e.g. DelDOT's 4:1 versus the *Roadside Design Guide's* 1:4.

The "forgiving roadside" concept recognizes that motorists do run off the roadway and that serious accidents and injuries can be lessened if at least a traversable recovery area is provided. The concept calls for a clear, unobstructed, relatively flat roadside area providing drivers an opportunity to recover control if their vehicle accidentally leaves the pavement surface. It may not be possible or practical to provide an area with flat slopes large enough to permit the driver to regain control of the vehicle. Where these areas can not be provided every attempt should be made to have an appropriate area clear of obstructions. The desired width of a project's clear zone varies based on several factors: (1) operating speeds, (2) traffic volume, (3) the steepness of slopes, (4) changes in slopes, (5) horizontal curvature, and (6) the accident history.

Any decisions on clear zone width obviously will influence the geometrics of the cross-section design, including design of side slopes. Since funds available for roadway improvements are limited, designers must consider the benefits and costs of alternate design treatments to provide the optimum clear zone design for any specific location. The proposed improvements for some projects do not take into consideration the clear zone based on the scope of work, such as minor improvements projects like pavement rehabilitation.

Table 3.1 of the *Roadside Design Guide* was developed to determine suggested roadside recovery area or clear zone distances for selected traffic volumes and speeds. The numbers are not precise since they are based on limited empirical data extrapolated to provide information for a wide range of conditions. Keep in mind

site-specific conditions, design speeds, rural versus urban locations, project scope and practicality. The clear-zone distances from Table 3.1 for horizontal curvature may be modified by using Table 3.2. These modifications are normally considered where accident history indicates a need or a specific site investigation shows definite accident potential which could be significantly lessened by increasing the clear zone width in a cost effective manner.

For relatively flat and level roadways, the clear-zone concept is simple to apply. However, it becomes somewhat less clear when the roadway is in a fill or cut section where roadside slopes may be either positive, negative, or variable, or where a ditch exists near the traveled way. Consequently, these features must be discussed before a full understanding of the clear zone concept is possible.

A basic understanding of the clear zone concept is critical to its proper application. As previously mentioned, the numbers obtained from Table 3.1 of the *Roadside Design Guide* are based on limited empirical data and extrapolated to provide information for a wide range of conditions. Thus, the numbers represent a reasonable measure of the degree of safety suggested for a particular roadside, but they are neither absolute nor precise. In some cases, hazards outside the clear zone may require removal or shielding depending on the severity of the hazard, the projected ADT, projected construction costs, and the classification of the roadway. The selection of an appropriate clear zone distance amounts to reaching a compromise between balancing user safety, construction costs, land use and social impacts, environmental concerns and the many other constraints that influence project decisions. Appropriate application of the clear zone concept will often result in more than one possible solution.

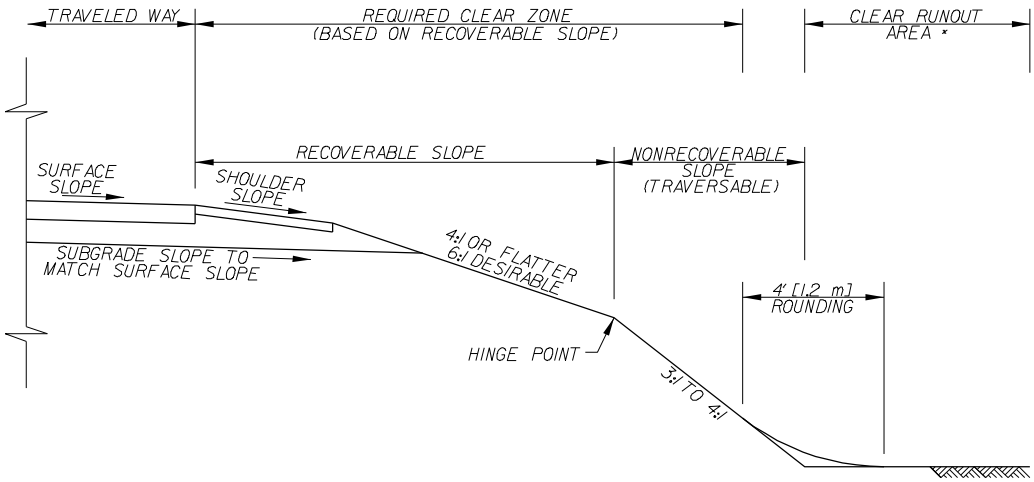
Chapter 3 of the *Roadside Design Guide* states: "The guidelines found in this chapter may be most applicable to new construction or major reconstruction." For other types of projects the guide recognizes that it may be not within the scope, not within the available funding, too environmentally disruptive, or impractical to achieve the recommended clear zone widths. Projects at this level are evaluated on an historical basis, identifying safety problems and obvious obstructions with emphasis placed on correcting these, if clear-zone related. TRB's *Special Report 214 Designing Safer Roads* should also be referred to when designing these types of projects.

In Delaware, experience has shown that the amount of lateral clear zone that should be provided varies from location to location. For each specific project a lateral clear zone is established by considering the following factors:

- Run-off-the-road accident experience,
- Design speed,
- Operating speed,
- Traffic volume,
- Steepness of side slopes,
- Profile grade,
- Horizontal curvature,
- Amount of roadside development,
- Sight distances,
- Level of improvement,
- Policy on removal and/or preservation of trees,
- Policy on installation of above ground utilities and
- Severity and location of the hazard.

After consideration of these factors and the clear zone requirements, the designer recommends a lateral clear zone or zones for each project. Clear zone widths generally will be uniform throughout the project except where widened for curvature. Lateral clear zone width decisions are an important design issue and are fully documented as described in Chapter 3.

Figure 4-3
Cross Section Side Slopes

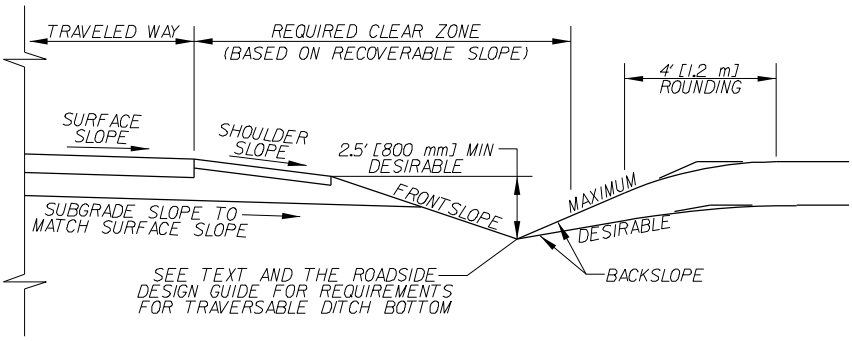


FILL SLOPES

* - THE CLEAR RUNOUT AREA IS ADDITIONAL CLEAR ZONE SPACE THAT IS NEEDED BECAUSE A PORTION OF THE REQUIRED CLEAR ZONE FALLS ON A NON-RECOVERABLE SLOPE. THE WIDTH OF THE CLEAR RUNOUT AREA IS EQUAL TO THAT PORTION OF THE CLEAR ZONE DISTANCE THAT IS LOCATED ON THE NON-RECOVERABLE SLOPE. THE CLEAR RUNOUT AREA SLOPE SHOULD BE RECOVERABLE OR AT LEAST TRAVERSABLE.

NOTES:

1. SEE TEXT SECTION 4.2.4.1 FOR DEFINITIONS OF CLEAR ZONE COMPONENTS.
2. A FILL SECTION USING A FRONTSLOPE CROSS SECTION THAT INCLUDES A NON-RECOVERABLE BUT TRAVERSABLE SLOPE WITHIN THE REQUIRED CLEAR ZONE IS COMMONLY REFERRED TO AS A "BARN ROOF DESIGN."



CUT SLOPES

Figure 4-4
Side Slope Criteria

Cut Slopes						
Road Class	**Front Slopes**		**Back Slopes**			
	Desirable	**Maximum**	**Desirable**	**Maximum** [1]		
				Depth of Cut		**Slope Ratio**
				US Cus-tomary	**Metric**	
Arterial	6:1	4:1	6:1	0 to 5 ft 5 to 10 ft 10 ft+	0 to 1.5 m 1.5 to 3 m 3 m +	4:1 3:1 2:1
Collector	6:1	4:1	4:1	0 to 5 ft 5 to 10 ft 10 ft +	0 to 1.5 m 1.5 to 5 m 3 m +	4:1 3:1 2:1
Local	6:1	4:1	4:1	0 to 5 ft 5 to 10 ft 10 ft +	0 to 1.5 m 1.5 to 5 m 3 m +	4:1 3:1 2:1

Fill Slopes						
Road Class	**Within Clear Zone**		**Outside Clear Zones**			
	Desirable	**Maximum**	**Desirable**	**Maximum**		
				Depth of Fill		**Slope Ratio**
				US Cus-tomary	**Metric**	
Arterial	6:1	4:1	6:1	0 to 5ft 5 to 10 ft 10 to 15 ft 15 ft +	0 to 1.5 m 1.5 to 3 m 3 to 5 m 5 + m	6:1 4:1 3:1 2:1
Collector	6:1	4:1	6:1	0 to 5 ft 5 to 10 ft 10 to 15 ft 15 ft +	0 to 1.5 m 1.5 to 3 m 3 to 5 m 5 m +	6:1 4:1 3:1 2:1
Local	4:1	3:1	4:1	0 to 3 ft 3 ft +	0 to 1.5 m 1.5 m +	3:1 2:1

Note:

Refer to the text and Tables 3.1 and 3.2 in the *Roadside Design Guide* for proper application of side slope design and clear zone requirements.

[1] The maximum back slope ratio outside the clear zone may be increased because of right-of-way restrictions.

In urban areas where curbs are often utilized, the space available for clear zones is generally restricted. In areas where barrier curbs are used, the clear zone shall extend to a minimum of 2 ft [0.6 m] beyond the face of curb, with wider clear zones provided where possible. In locations with mountable curbs, the clear zone width provided shall be as determined in *Roadside Design Guide*'s Table 3.1 as adjusted by Table 3.2.

If the clear zone width requirements, as determined by the above procedure, are not practicable because of local conditions or are inadequate because of specific safety problems, the designer should consider adjustments to the highway geometry or the installation of barriers. Refer to Chapter Ten for policies and criteria for barriers.

Utility poles are considered an obstruction and are not permitted within the clear zone without an engineering study, including accident history, and proper documentation. A design exception may be necessary. The use of breakaway utility poles can be considered as an alternative to moving or burying the utility where there is documented accident history.

4.2.4.2 FRONT SLOPES

Slopes within the selected clear zone must have a slope rate that is relatively flat. Normally a 6:1 slope (or flatter) should be used outward and downward from the edge of the finished shoulder to the outer limits of the lateral clear zone. Steeper slopes, up to a maximum of 3:1, may be used for low traffic volume roads and for conditions where flatter slopes would cause inordinately high costs. With slopes steeper than 4:1 the horizontal distance of the steeper slope cannot be used to meet the clear zone requirements.

For relatively low embankment heights, fill slopes extending outward and downward from the outer limits of the lateral clear zone to the natural ground normally should be the same.

Under conditions of high fills and/or right-of-way restrictions, steepening the fill slopes to a maximum of 2:1 beyond the clear zone may be considered, but they should be designed as flat as practical within the constraints of local right-of-way conditions. Slopes steeper than 3:1 cannot be mowed with conventional mowers. The cost of flattening slopes versus the cost of guardrail is also a consideration as guardrail is deemed a roadside obstruction and can be a continuing maintenance problem.

Slopes that parallel the traveled way can be recoverable, non-recoverable, traversable or critical. For recoverable slopes that are smooth and traversable with slopes of 4:1 or flatter, the suggested clear zone may be taken directly from the *Roadside Design Guide*'s Table 3.1, adjusted by Table 3.2 as necessary. Motorists who encroach on recoverable slopes can generally stop their vehicles or slow them enough to safely return to the roadway. Fixed object hazards, such as culvert headwalls, should not extend above the embankment either within the clear zone or beyond if the embankment is traversable to the bottom.

Slopes between 3:1 and 4:1 are considered non-recoverable slopes for which most motorists would be unable to safely stop or return to the roadway. It is very important that these embankments be traversable since a high percentage of encroaching vehicles will reach the toe of these slopes. The clear zone cannot logically end on the slope. Fixed object hazards should not be constructed along such slopes, and a clear runout area should be provided at the base of the slope. The runout area is a relatively flat clear area wide enough to allow the vehicle to stop. Figure 4-3 shows an example of such a clear zone. The clear zone width is the sum of the widths of the shoulder, any recoverable slopes, and the clear runout area, but excludes the non-recoverable slope.

Front slopes steeper than 3:1 are considered critical slopes and a driver will most likely lose control. If a slope steeper than 3:1 begins closer to the traveled way than the suggested clear zone for that specific roadway and the slope

cannot readily be flattened, a barrier may be warranted. Barrier warrants are included in Chapter Ten-Miscellaneous Design.

In developing the proposed roadway cross section, the design cross section may consider a shape that resembles and is referred to as a "barn roof" section, (see Figure 4-3). This design provides a relatively flat recovery area adjacent to the roadway for some distance, followed by a steeper traversable down slope. This cross section may be more economical than providing a continuous flat slope from the edge of the traveled way to the original ground line and is generally perceived as safer than a continuous steeper slope.

In cut sections, the slope extending outward and downward from the finished shoulder to the side ditch should desirably be 6:1. For low traffic volume roads and unusual local conditions, the front slopes may be as steep as 3:1. On federal-aid projects the "desirable" criteria shown in Figure 4-4 shall be used. It is desirable that the front slope extend outward far enough to provide a side ditch flow line elevation at least 2.5 feet [800 mm] below the elevation of the paved or finished shoulder. The purpose of this minimum ditch depth is that it will place the ditch bottom below a normal pavement box allowing any moisture trapped under the pavement to travel through a porous subbase into the ditch. In addition, when combined with the proper slope, it will provide a depth that allows for some temporary ponding but will quickly remove any roadway runoff before it saturates the pavement structure.

4.2.4.3 BACK SLOPES

Back slopes extending upward and outward from side ditches to intersect the natural ground desirably should be 6:1 for principal arterials, but may be slightly steeper for lower classes of roads.

Under conditions of deep cuts and/or right-of-way restrictions, steepening the back slopes to a maximum of 2:1 may be considered, depending on the depth of cut and the class of road. Recommendations for slopes in rock cuts will be made for individual projects based on studies of local conditions.

When a roadway is in a cut section, the back slope may be hazardous depending upon its relative smoothness and the presence of fixed object hazards. If the slope is traversable (3:1 or flatter) and obstacle-free, it may not be a significant hazard. However, steep back slopes or those with obstacles such as rock cuts within the clear zone may require shielding if they cannot be flattened or the obstacle removed. Warrants for barriers are discussed in Chapter Ten.

4.2.4.4 TRANSVERSE SLOPES

Common obstacles along roadsides are transverse slopes created by median crossovers, drainage structures, driveways and intersecting side roads. These are generally more critical to errant motorists than front slopes or back slopes because they are typically struck by run-off-the-road vehicles travelling parallel to the roadway and impact the feature head on. Transverse cross slopes of 6:1 or flatter are suggested for high-speed roadways, particularly for the section of the embankment that is located immediately adjacent to traffic. This slope can then be transitioned to a steeper slope as the distance from the traveled way increases.

Embankment slopes (including the ends of any drainage structures) of 10:1 are desirable; however, their practicality is limited by width restrictions and the maintenance problems associated with long tapered pipe ends. Embankment slopes that are steeper than 6:1 may be considered for urban areas or for low-speed facilities. Safety treatments of drainage structures are discussed in Chapter Six-Drainage.

4.2.5 ROADSIDE DITCHES

The two principal functions of roadside ditches (hydraulically defined as open channels) are: (1) to drain water from the subgrade and (2) to collect surface water either from the roadway

surface or adjacent roadside areas and remove it before entering the subgrade. Moisture in the subgrade and the frequency and magnitude of pavement loads are the most destructive forces to the roadbed and pavement structure. In addition, roadside ditches are an important element in reducing the environmental impact of a project on the adjacent landscape. Ditch designs can play a major role in managing stormwater runoff, removal of sediment, controlling erosion, and reducing the impact of roadway pollutants on watercourses.

Insofar as practical, ditch cross sections should be traversable within the clear zone. Figures 4-5 and 4-6 show preferred front slopes and back slopes for basic ditch configurations. Cross sections that fall within the shaded area of each figure are considered to have traversable cross sections. Ditch sections that fall outside the shaded area are considered less desirable; their use should be limited where high-angle encroachments, such as the outside of relatively sharp curves, can be expected. Ditch sections outside the shaded area may be acceptable for projects with one or more of these characteristics: restrictive right-of-way, rugged terrain, low traffic volume low, operating speed, or projects involving resurfacing, restoration, or rehabilitation, particularly if the ditch bottom and back slopes are traversable and free of any fixed objects. If practical, ditches with cross sections outside the shaded areas and in vulnerable locations may be re-shaped and converted to a closed system or shielded with traffic barriers.

Side ditches are particularly important to control surface drainage through cut sections in order to maintain the design integrity of a freely draining pavement structure. If the excavated material is of adequate quality it usually is used in the construction of adjacent fill sections. Figure 4-3 indicates that ditches may not be required at the toes of fill sections to drain the subgrade. However, in addition to the previously discussed environmental concerns, there may be other reasons to provide ditches to control runoff at the toes of fills to carry the flow to natural drainage channels thereby minimizing real or perceived damage to adjacent properties.

The two commonly used geometric configurations for side ditches are trapezoidal and v-ditch.

4.2.5.1 TRAPEZOIDAL DITCH

The preferred design is a ditch that is trapezoidal in shape with relatively flat front slopes and back slopes and a wide, flat bottom. (See Figure 4-5.) The general configuration of the trapezoidal ditch section does graphically show sharp breaks at the intersection points. However, constructing these breaks in the field is not always practical and they are normally graded in a more rounded shape making this ditch type more easily traversable than most other shapes.

4.2.5.2 V-DITCH

The V-ditch is a less desirable ditch design. Safety features are reduced because of the sharp break in the slope between the front slope and back slope. (See Figure 4-6.) This type of ditch section is more easily constructed and requires less right-of-way. However, it is not the best choice for traversability or maintenance.

4.3 MEDIANS

Medians are provided on divided multi-lane highways to provide a separation of opposing traffic lanes, a recovery area for out-of-control vehicles and an area for emergency stops. Besides these safety benefits, medians also can provide space for:

- Left-turn lanes,
- Snow storage,
- Collecting surface drainage,
- Refuge for pedestrians at crosswalks,
- Installation of traffic control devices, and
- Adding future lanes.

Figure 4-5
Trapezoidal Ditch Section

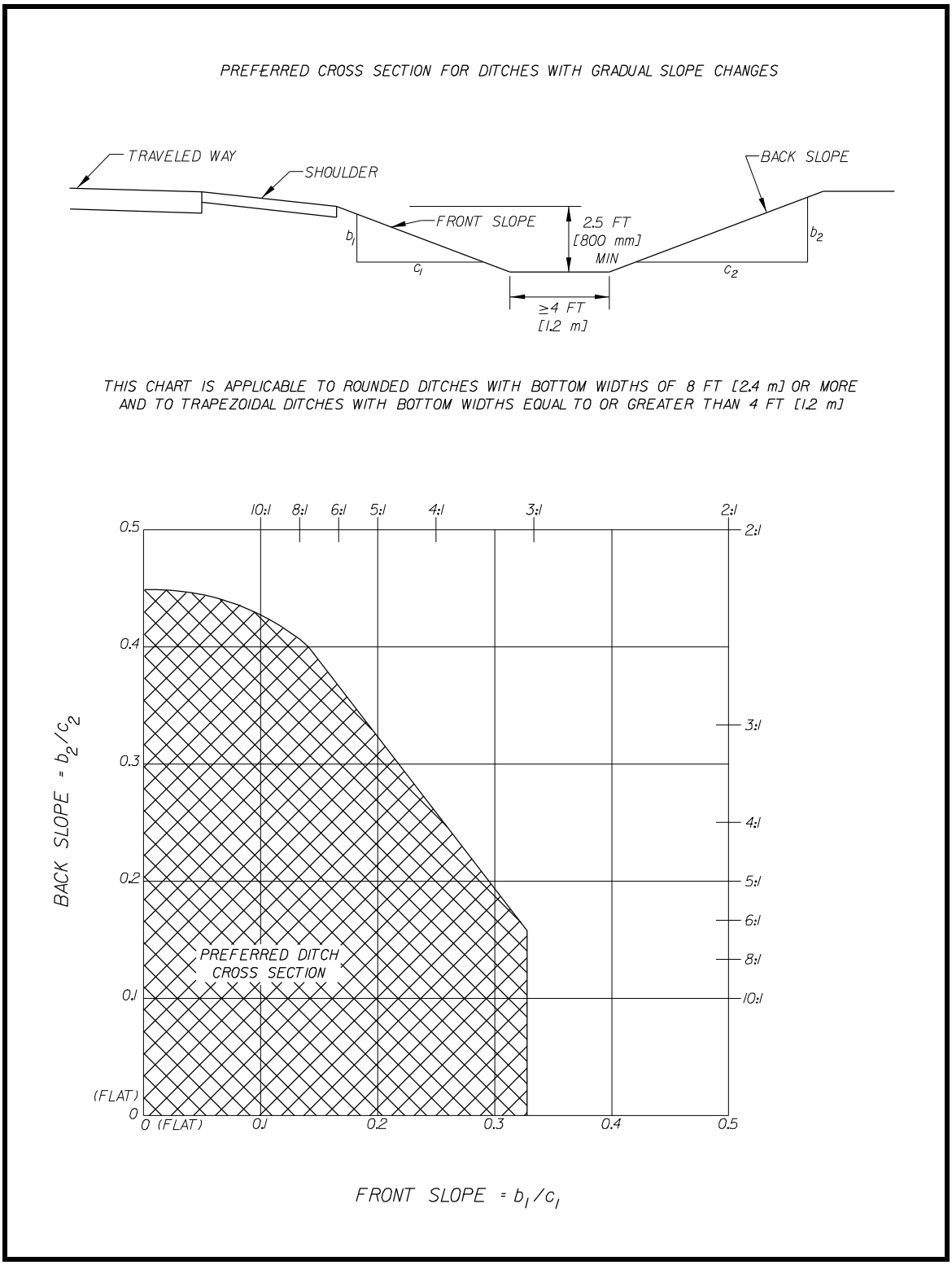
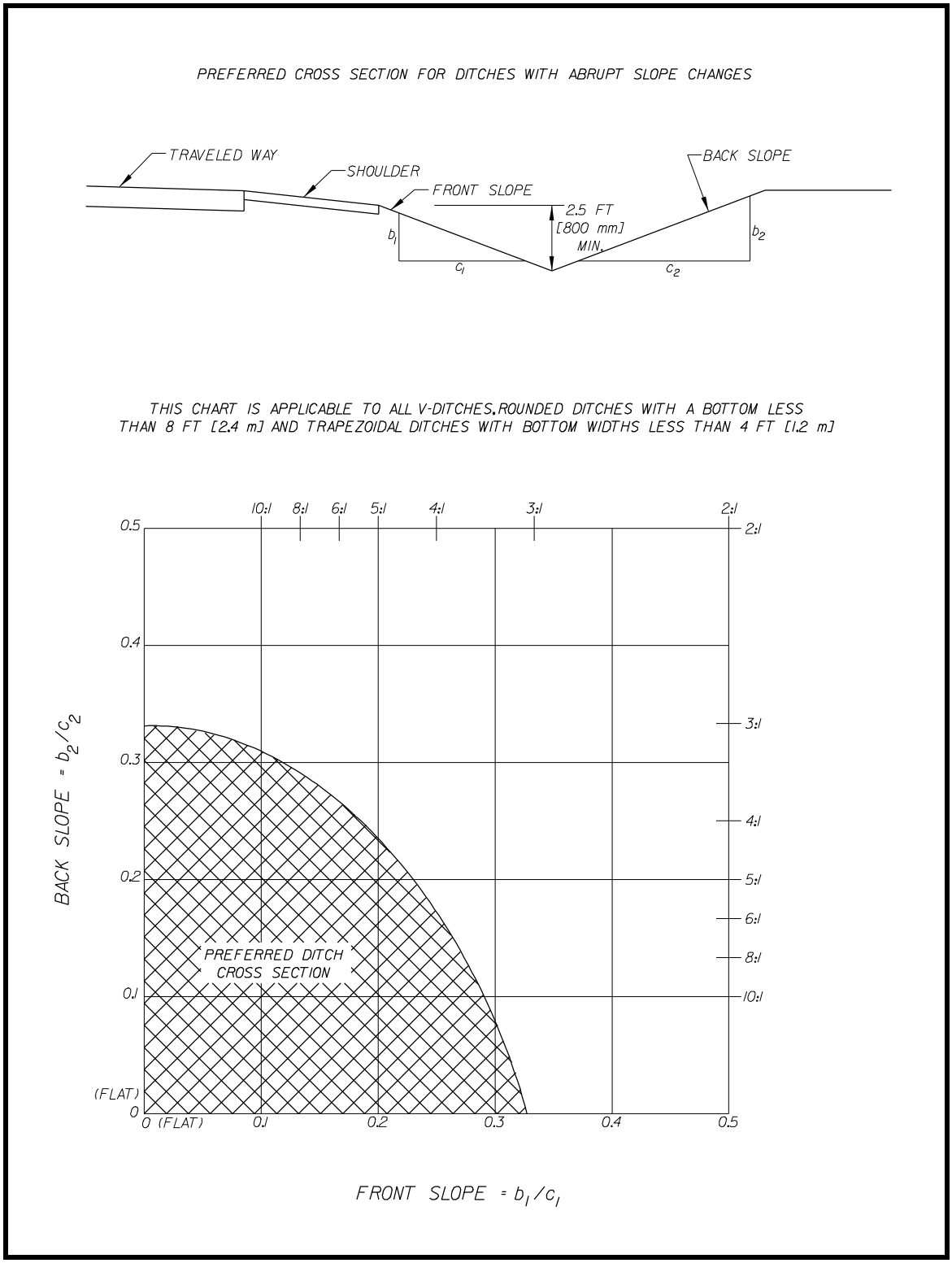


Figure 4-6
V-Ditch Section



Median widths are always measured between the inside edges of opposing travel lanes. Medians operate best when they are highly visible during the day or night and are at a width that provides for the predominant usage. There are three basic types of medians:

- Flush medians,
- Curbed (raised) medians, and
- Depressed medians.

The use of medians in providing for U-turn movements, auxiliary lanes and intersection design is further discussed in Chapter Seven-Intersections.

4.3.1 FLUSH MEDIANS

Flush medians consist of a relatively flat paved area separating the traffic lanes with only painted stripes on the pavement. This type is generally used only for lower-speed urban arterials. Painted medians need frequent repainting to maintain their visibility at night and under inclement weather conditions.

To accommodate painted left-turn channelization, flush medians should be at least 16 ft [4.8 m] wide and desirably 18 ft [5.4 m]. Flush medians should be either slightly crowned to avoid ponding of water in the median area or slightly depressed (with median drains) to avoid carrying all surface drainage across the driving lanes.

4.3.2 CURBED MEDIANS

Curbed raised medians are most commonly used on lower-speed urban arterials. They have the same basic advantages and characteristics of flush medians except the separation is more clearly defined than for painted lines, do not need frequent repainting and are more easily seen at night and during inclement weather.

Typical widths of raised medians range from 4 to 22 ft [1.2 to 4.1 m]. A raised median of 4 to 6 ft [1.2 to 1.8 m] in width with a paved surface may be used under restricted conditions on urban streets, but they have limited advantages. Although they provide a positive separation between opposing traffic and an opportunity to collect drainage, they offer no opportunity to introduce left turn lanes, are too narrow to provide a desirable pedestrian refuge and do not adequately serve as an area for installing traffic control devices.

The absolute minimum median width is 12 ft [3.6 m] for introducing left-turn lanes on low-speed arterial streets with restricted conditions and minimal truck use. Any size truck (as well as many passenger car drivers) could not use this lane without infringing on the adjacent travel way. A median width of 16 ft [4.8 m] is the normally accepted minimum in urban areas to adequately serve a mix of drivers and vehicles without having erratic movements. This width provides for a 10 ft [3.0 m] turn lane and a 6-ft [raised median]. This width does not provide any curb offset so there will be a tendency for drivers to shy away from the median into the adjacent travel way.

The two preferred urban median widths, where frequent left turns are to be accommodated with a diverse traffic mix, are 20 ft [6.0 m] or 22 ft [6.6 m]. A 20 ft [6.0 m] median width allows for a 12 ft [3.6 m] left turn lane, 2 ft [0.6 m] clearance from the edge of traffic lanes to the face of the curbed island, and a 4 ft [1.2 m] wide island to provide space for traffic control devices. However, in high pedestrian use areas, the preferred width is 22 ft [6.6 m] that will allow for a 6 ft [1.8 m] raised median for pedestrian refuge.

4.3.3 DEPRESSED MEDIANS

Depressed medians are most commonly used for high-speed expressways, freeways and rural arterials. Depressed medians are uncurbed grass areas with flat slopes drained by open ditches and flush drainage inlets. Normally, the widths of depressed medians are considerably greater than for either flush medians or raised medians. Smoother traffic operations and improved traf-

fic safety are observed advantages of wide, depressed medians.

Designing a relatively narrow depressed median creates problems. The result is that the longitudinal drainage ditch in the center of the median is too shallow, or the transverse slopes from the roadways to the ditch are too steep.

Median side slopes of 6:1 or flatter, for a distance of at least 30 ft [9 m] from the edge of the traffic lanes, are preferred. Other median slopes (for median crossovers, ditch blocks, etc.), that might be in the path of an out-of-control vehicle, should be 6:1 as a minimum and preferably 10:1 or flatter as a safety feature.

A width of 40 ft [12.0 m] or more for depressed medians permits adequate drainage design with flat slopes. A median width of at least 50 ft [15.0 m] can safely store a school bus. Wider medians are desirable where right-of-way permits allowing for the placement of a median bridge pier or overhead sign structure without the need for barrier protection. Wider medians should also be considered where there is a potential for adding travel lanes in the median to meet future traffic demand. Also see the *Green Book* pages 460 and 461 for further discussion on this subject.

Where flat longitudinal slopes on the roadway are encountered, the cross slopes of the median may be varied to increase the longitudinal slope of the median ditch. For example, the cross slope may be kept very flat (10:1 or flatter) at the upper end of the drainage area and steeper (6:1) at the lower end.

4.3.4 MEDIAN BARRIERS

For divided highways with large traffic volumes and high operating speeds, a wide, depressed median is the best choice. Under some conditions this is not practicable, and a flush or raised median must be provided. But in this case, some type of physical barrier must be placed in the median to prevent out-of-control vehicles from crossing into opposing traffic lanes.

Several types of physical median barriers can be designed. Refer to Chapter 10 and the *Roadside Design Guide* for criteria for median barriers.

4.3.5 MEDIAN OPENINGS

Refer to Chapter 7 for the design of median openings and channelization for left turns.

Figure 7-7
Design Widths for Turning Roadways [Metric]

Radius on Inner Edge R [m]	Pavement width [m]								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 0.3 m Add 0.6 m			None Add 0.3 m			Add 0.3 m Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width; minimum width as under Case I			Deduct 0.6 m where shoulder is 1.2 m or wider		

Note:

Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination trucks.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Certain principles should be followed in the design of a channelized intersection, but the extent to which they are applied will depend on the characteristics of the total design plan. These principles are:

- Motorists should not be confronted with more than one decision at a time.
  - Unnatural paths that require turns greater than 90° or sudden and sharp reverse curves should be avoided.
- Areas of vehicle conflict should be reduced as much as possible. Channelization should be used to keep vehicles within well-defined paths that minimize the area of conflict.
- The points of crossing or conflict should be studied carefully to determine if such conditions would be better separated or consolidated to simplify design with appropriate control devices added to ensure safe operation.
- Refuge areas for turning vehicles should be provided clear of through traffic.
- Prohibited turns should be blocked wherever possible.
- Location of essential control devices should be established as a part of the design of a channelized intersection.
- Channelization may be desirable to separate the various traffic movements where multiple-phase signals are used.

7.3.3 ISLANDS

Design of islands is the principal concern in channelization. An island is a defined area between traffic lanes for control of vehicle movements. Islands may be delineated or outlined by a variety of treatments, depending on their size, location and function. Types of delineators include: (1) raised islands outlined by curbs, (2) islands delineated by pavement markings, and (3) non-paved areas formed by the pavement edges–possibly supplemented by delineators on posts or other guide posts.

Islands provide three major functions:

- **Channelizing islands** control and direct traffic movement, usually turning;
- **Divisional islands** divide opposing or same-direction traffic streams, usually through movements; and
- **Refuge islands** provide refuge for pedestrians and bicyclists.

Most islands combine two or all of these functions. Triangular channelizing islands, used for roadways, subdivision street entrances or commercial driveway entrances, serve all three functions where there are pedestrian facilities.

Islands should be sufficiently large to command attention, and to accommodate pedestrian refuge and pedestrian signal poles where they are needed. Curbed islands normally should be no smaller than 50 ft^2 [5 m^2] for urban streets and about 75 ft^2 [7 m^2] for rural intersections; however, 100 ft^2 [9 m^2] minimum is preferred for both. Triangular islands should not be less than 12 ft [3.6 m], preferably 15 feet [4.5 m], on a side before rounding the corners; those with curb ramps, pedestrian refuge and pedestrian signal poles should have sides at least 15 ft [4.5 m] and preferably 20 feet [6.0 m] on a tangent side resulting in a minimum island area of 175 ft^2 [16 m^2]. Median islands narrower than 8 ft [2.4 m] from back of curb to back of curb cannot be mowed effectively; therefore they should be paved. Elongated or divisional islands should not be less than 4 ft [1.2 m] wide, 6 ft [1.8 m] if pedestrians are anticipated and 20 to 25 ft [6.0 to 8.0 m] long.

DelDOT has adopted general rules for the placement of islands. The first preference is to design the intersection with radii that accommodate the selected design vehicle path without creating large open paved areas requiring the need for islands to direct traffic. The overriding criteria are operational efficiency and safety. In many locations the best way to accomplish this is with islands. Curbed islands usually provide the better alternative to delineate the location, minimize maintenance and control drainage. Using the proper type of curb and offsets are important design considerations. Full shoulder width offset and mountable P.C.C. Curb, Type 2, are the preferred design elements. Typically, the island is offset from the traveled way the full width of the

shoulder or turn lane. This offset may be reduced where necessary as determined by a traffic study to only five feet to accommodate bicycles under the following conditions:

- Urban, suburban and developing areas where, due to queue lengths and congestion, traffic needs to be discouraged from using the shoulder to pass on the right.
- Commercial driveway entrances or streets leading up to an urban, suburban or developing intersection to prevent illegal shoulder traffic before the deceleration lane. Here, the island also offers protection to the vehicle entering the highway and prevents a car crossing the highway entering the business or side street from being involved in an angle crash.
- Crosswalks where pedestrian refuge and shortening the length of the crossing is needed, particularly with signalization.

In areas with high pedestrian traffic where there is a need to minimize the distance between refuge areas, the offset for bicycles may be reduced to 4 ft [1.2 m] from an approaching curbed section; in this case, it is preferable to offset the nose of larger islands 4 to 6 ft [1.2 to 2 m] from approaching curbed lanes to allow an errant vehicle to recover before striking the curb. To accommodate the variety of vehicle, pedestrian and bicycle movements in urban areas, various offsets and configurations may be needed within the same intersection.

Figures 7-8 and 7-9 show a typical preferred island layout for rural and urban conditions.

No matter what the island configuration:

- Positive drainage must be provided for the safety of vehicles and pedestrians.
- The corners of the island shall be flush with the pavement, tapering back to full height at a slope of 4:1 for triangular channelizing islands and 10 feet [3.05 m] for median islands (where the sleeve for the "Keep Right" sign would be placed) for snow plowing operations and errant vehicles.
- The corners of islands not offset the full width of the shoulder adjacent to the roadway shall be delineated with tubular delineators.

Figure 7-8
Typical Island Layout–Rural Areas

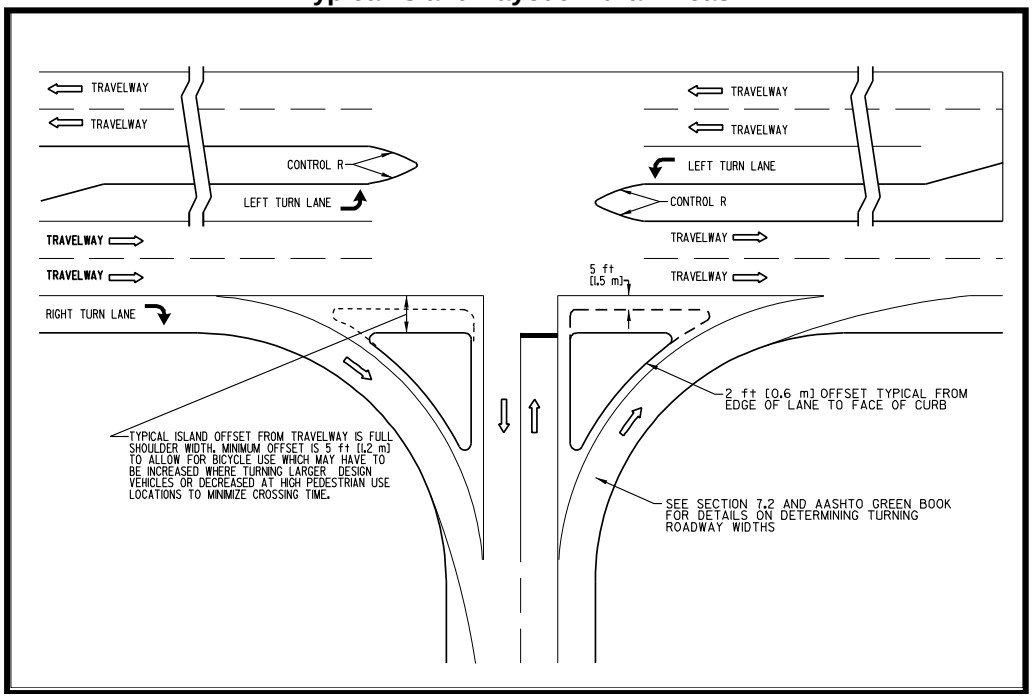
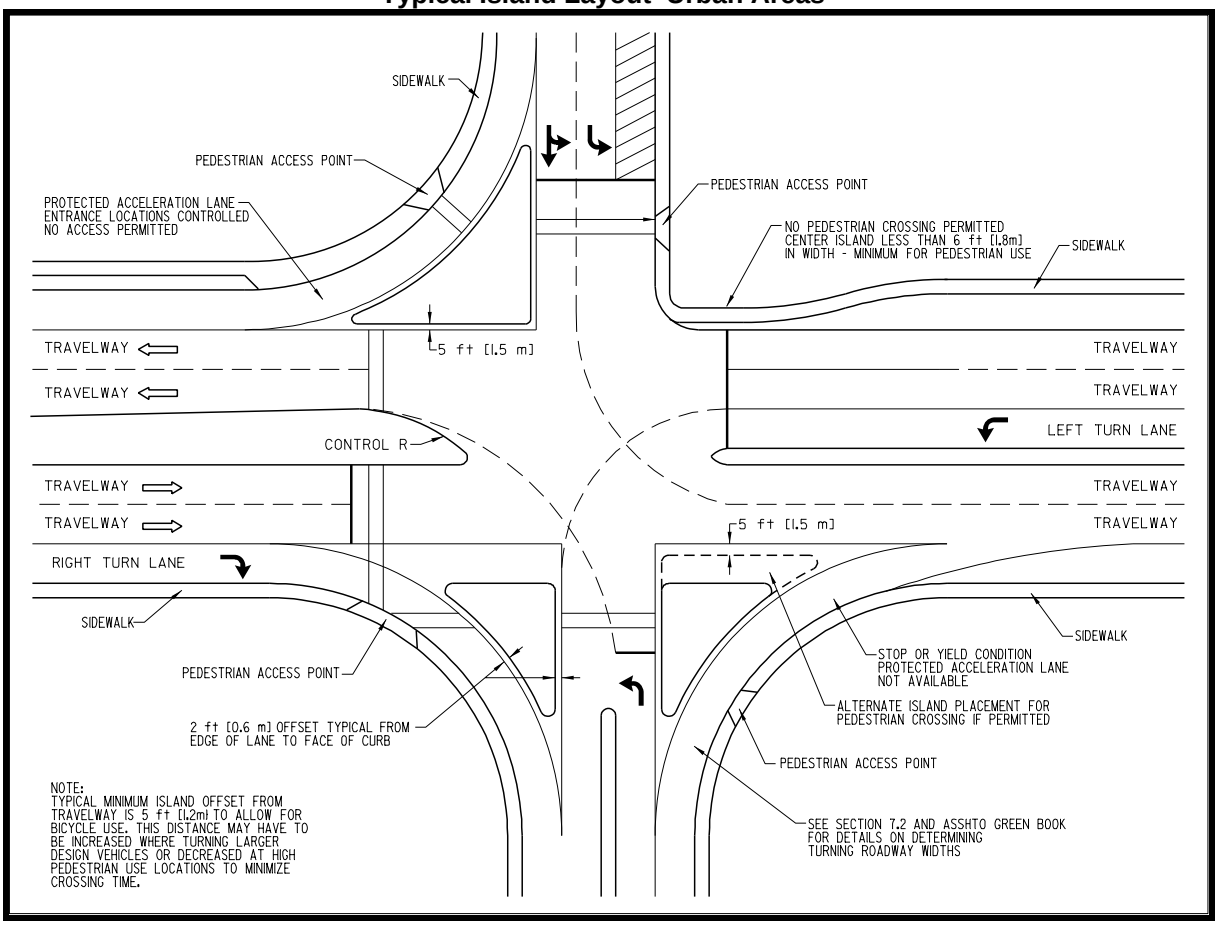


Figure 7-9
Typical Island Layout–Urban Areas



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7.4 SIGHT DISTANCE

The operator of a vehicle approaching an at-grade intersection should have an unobstructed view of the whole intersection and of a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions which is termed "approach sight distance". (See Figure 7-10.) The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and the resultant distances traversed during perception, reaction time, and braking. In addition to approach sight distance, sight distance is also provided to allow stopped vehicles sufficient view of the intersecting roadway to decide when to enter the intersecting roadway or to cross it, which is "departure sight distance." Both sight distances must be checked on all intersection designs based on the procedures set forth in the Green Book, pages 650 to 679.

7.4.1 MINIMUM SIGHT DISTANCE TRIANGLE

Sight triangles are areas of unobstructed sight along both roads at an intersection and across their included corner for a distance sufficient to allow the operators of vehicles approaching simultaneously to see each other in time to prevent collision at the intersection.

The length of the legs may vary based upon traffic volumes, design speeds, operating speeds and type of intersection traffic control.

Chapter Nine

PAVEMENT SELECTION

This chapter discusses the general criteria, procedures and responsibilities for structural design of highway pavements. In addition there is information given on the various types of pavements, pavement rehabilitation techniques, and other factors that enter into pavement design and final pavement selection.

For the purposes of uniform and consistent design practices, the Department has adopted the criteria and procedures as set forth in the *AASHTO Guide for Design of Pavement Structures 1993*. This chapter briefly reviews the concepts and criteria used. Reference should be made to the AASHTO Guide for more detailed information on design procedures, if needed.

The design procedures include the determination of total thickness of the pavement structure as well as the thickness of the individual components using input parameters specified in the DARWIN 3.01 computer program. Provision is made for the design of equivalent alternate pavement sections, with the selection primarily a function of availability of materials, comparative costs, constructibility, and availability to traffic.

The discussion and explanatory material presented here are intended to give the designer a general understanding of pavement design concepts, alternative paving treatments, and a basic understanding of the information contained in a soil survey and pavement design report.

9.1 DESIGN RESPONSIBILITY

The design of pavement structures has some jointly shared responsibilities between the Materials and Research (M&R) Section and the responsible Project Development section. However, the primary responsibility for structural design and final recommended pavement sections is that of the M&R Section.

9.1.1 SOIL SURVEY/PAVEMENT EVALUATION REQUEST

There are several elements in the design process that need to be accomplished before a pavement section recommendation can be requested from the M&R Section. The project handoff package will describe the project scope. If the intent is to construct new pavement or replace the existing pavement then a soil survey, pavement design and pavement type recommendation will have to be requested.

The following should be available when requesting borings for a soil survey from DelDOT's Geotechnical Engineer and/or corings of the existing pavement from DelDOT's Pavement Design Engineer:

- preliminary surveys,

- survey plans with the location of the investigation, including the road name and state maintenance road number, and any predetermined locations marked,
- existing right-of-way verified,
- Right-of-entry to trespass, if needed,
- Purpose of the investigation in order to determine what pertinent information is required from the cores or the borings.
- If a boring is required, the depth of the boring. If the designer is unsure of depth to sample, contact the Geotechnical Engineer for guidance.

Coring and boring requests are typically processed within 30 days. If a coring and boring request is received simultaneously, they will be processed concurrently. If the boring request is received after the coring request has been completed, it will be treated as a different request and will be processed within 30 days of receipt.

For a pavement design, the following information is required with the request:

- Design year traffic data (AADT)
- Design year truck percentages
- Weight group pattern of trucks
- Directional split
- Existing pavement structure (if applicable)
- Subsurface investigation report (if applicable)
- Description of any existing pavement deterioration

Copies of any existing corings, borings, or subsurface condition reports should be provided to the Pavement Design Engineer when sending the pavement design request. The Pavement Design Engineer may elect to have additional corings, borings, or other investigations performed to ensure the existing conditions are known and considered when performing the pavement design. All information, including the design, will be forwarded to the designer.

For projects designed by a consultant, the design consultants must:

- Develop the subsurface investigation plan to have drilling and inspection services provided under their direction.
- Notify the DelDOT Project Manager of the scope of the subsurface investigation plan and provide a boring plan sheet(s) to the Department. The DelDOT Project Manager will forward the boring plan to the Materials & Research (M&R) Geotechnical Engineer
- Specify to M&R what soil testing is requested.
- Inform the Geotechnical Engineer at least 24 hours in advance when drilling and soil sampling is to begin and provide the name and phone number of the consultant who is to receive the M&R soil test information.
- Arrange to have the soil samples and copies of the field logs brought to the M&R lab within 24 hours after obtaining samples.

The Geotechnical Engineer shall:

- Perform, or arrange for, all soil testing required for the project as requested by the design consultant.
- Furnish completed boring logs and laboratory test summaries to the design consultant with a copy to the DelDOT Project Manager.
- Provide the DelDOT Project Manager regular weekly updates on the progress of the drilling/testing programs.
- Inform the DelDOT Project Manager when problems arise and when the testing programs are completed.

Many projects are initiated with the intent of extending the service life of the existing pavement section. Rather than rebuilding the entire pavement, improvements are made in the riding quality, skid resistance and limited structural improvements through various rehabilitation methods. For projects of this type a pavement

evaluation is requested. A pavement evaluation includes an extensive pavement survey noting pavement conditions, drainage and major distress problems. The initial evaluation will determine which rehabilitation method(s) should be considered. Based upon the alternatives being considered, it may be necessary to conduct detailed measuring and testing, including coring and sampling, and accurately determining structural clearances to any overhead structures. The final report received by the designer will contain most if not all this collected field data and will include the recommended rehabilitation method. Much of the field data collected can be a very valuable tool in designing the project.

9.1.2 SOIL AND PAVEMENT DESIGN REPORT

The M&R Section performs the soil sampling and pavement coring. This field data is tested in the laboratory and the results documented in the form of a Soil Survey. This report includes the soil profile data summary showing the sample location, depth, soil profile description, soil classification and any remarks. The soil samples are tested and a summary of this analysis is included in the report.

Based upon the subgrade soil characteristics, the report may provide recommendations for muck excavation, limits of special fill, need for underdrains, grade adjustments, embankment construction, the use of geotextiles or other special construction considerations.

An important part of the report is the pavement design portion. Using the AASHTO Guide, the soil survey data, and past experience, the recommended pavement type and thickness by components is included in the report. Pavement sections for shoulders and turn lanes are also included.

The report may provide an alternate design including at least one rigid pavement and one or more flexible alternatives. Project Development and M&R will meet and mutually agree on which pavement to use on the project. If necessary, the M&R Section may perform an economic and life cycle cost analysis or other studies in making the final determination and recommendation for pavement type and section. For many projects, only one pavement design recommendation is made.

The Soil and Pavement Design Report should be evaluated thoroughly and considered as the design progresses. In the design process significant changes in the proposed profile or alignment can dramatically affect the pavement design and need to be discussed with M&R, perhaps even before such decisions are finalized. In addition the proposed construction sequencing, methods of construction and maintenance of traffic plan can dramatically affect whether the recommended pavement materials can be placed in a timely manner and meet the in-place performance quality necessary for the intended service life.

9.1.3 PAVEMENT SELECTION

As mentioned in Section 9.1.2, there may be several choices of structurally equivalent pavements. The choice of pavement, particularly on reconstruction and new construction is a major decision and needs to be approved prior to proceeding with the design.

The factors considered in making the final decision on pavement type are quite varied from empirical to subjective and may include several of all of the following:

- Project scope—as initiated,
- Cost to construct or rehabilitate the pavement,
- Available project funding,
- Construction sequencing as it relates to controlling through and local traffic,
- Construction sequencing as it relates to serving commercial areas,

- Construction sequencing as it relates to maintaining quality control of pavement construction,
- Availability of work areas for the paving equipment,
- Projected future traffic control and other costs to perform maintenance, restoration or rehabilitation
- Minimum desirable service life,
- Performance of similar pavements under similar soil conditions and traffic loadings,
- Geotechnical design problems created by the depth of pavement structure, which could increase drainage costs,
- Effect on underground utilities, and
- Effect on existing vertical clearances.

9.2 PAVEMENT TERMINOLOGY

The pavement section is treated as a structural element consisting of several different materials of varying depths and supporting strengths. Knowledge of the following definitions and the terminology as shown on Figure 9-1 is needed to understand the pavement design and rehabilitation concepts in this chapter.

- Base Course—the layer or layers of specified or select material of designed thickness placed on a subgrade to support a surface course
- Bituminous Concrete—A designed combination of dense graded mineral aggregate filler and bituminous cement mixed in a central plant, laid and compacted while hot.
- Bonded Overlay—An overlay of concrete placed over a Portland cement concrete pavement.
- Flexible Pavement—A pavement structure of bituminous concrete that distributes loads to the subgrade and depends on a firm continuous subgrade, aggregate interlock, particle friction, and cohesion for stability.
- Grinding—Patterns cut into a concrete pavement with closely spaced diamond blades to restore pavement smoothness and skid resistance.
- Grooving—Patterns cut in asphalt or concrete pavements to promote surface drainage to reduce wet weather hydroplaning.
- Micro-Surfacing—A polymer modified cold-mix paving system consisting of a mixture of dense-graded aggregate, asphalt emulsion, water, and mineral fillers.
- Modulus of Subgrade Reaction (k)—A value used in rigid pavement design that is the ratio of the load in pounds per square inch [in kilograms per square mm] on a loaded area of the subgrade or subbase divided by the deflection in inches [mm] of the subgrade soil or subbase, psi/in [kPa]. Typically, k is adjusted for potential loss of support due to subbase erosion.
- Open Graded Mix—A special mix, containing aggregate that resists polishing, placed on the surface course to drain surface water, improve skid resistance, and reduce hydroplaning.
- Rigid Pavement—A pavement structure that distributes loads to the subgrade, having as one course a Portland cement concrete slab of relatively high bending resistance.
- Pavement Milling—The use of carbide cutting teeth mounted on a rotary drum to chip off as much 3 to 4 inches [75 to 100 mm] of asphalt concrete surface.
- Pavement Structure—A combination of subbase, base course and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed
- Recycling—Salvaging and processing portions of existing pavement for use in construction of new pavement structures.
- Resilient Modulus (M_R)—A measure of the essential properties of untreated subgrade soils through determination of dynamic elastic modulus under conditions that represent a

reasonable simulation of the physical conditions and stress states of subgrade materials beneath flexible pavements subjected to moving loads.

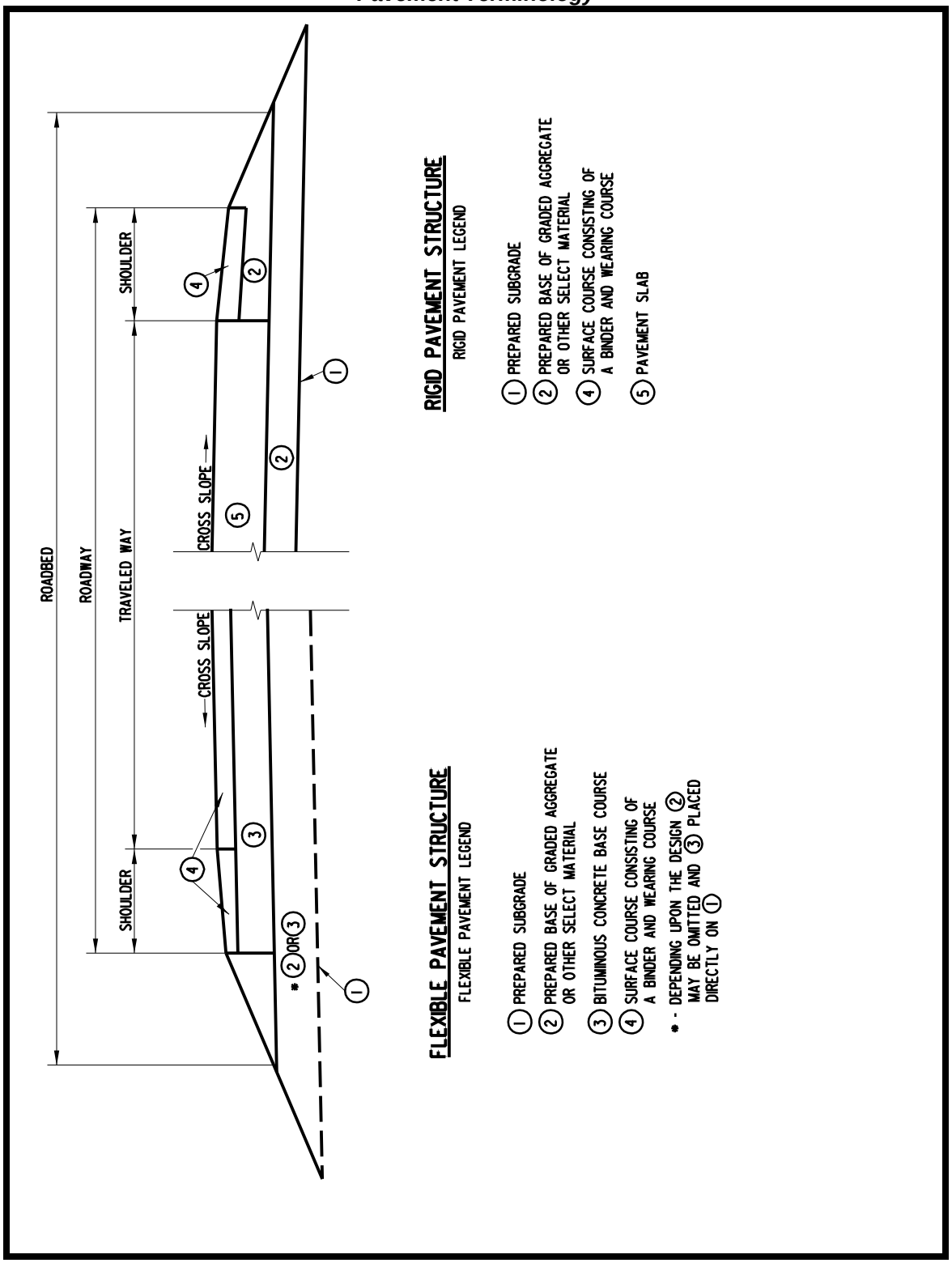
- Roadbed Material—The material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.
- Roadbed—The graded portion of a highway between top and side slopes prepared as a foundation for the pavement structure and shoulders.
- Select Material—A suitable native material obtained from a specified source such as a particular roadway cut or borrow area having specified characteristics to be used for a specific purpose.
- Subbase—The layer or layers of specified or select material of designed thickness placed on a subgrade to support a base course (or in the case of rigid pavements, the Portland cement concrete slab).
- Subgrade—The top surface of a roadbed soil upon which the pavement structure and shoulders are constructed.
- Superpave—An asphalt pavement with a laboratory design mix that provides superior performance.
- Surface Course—One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called "wearing course."
- Ultra-Thin-Whitetopping—An overlay of concrete less than 4 inches [100 mm] thick placed over an asphalt base, usually 2 to 3.5 inches [50 to 90 mm].
- Whitetopping—A concrete overlay of an asphalt pavement of 4 inches [100 mm] or more.

9.3 PAVEMENT DESIGN FACTORS

Pavement design methodology for both new pavements or rehabilitation of existing pavements consider several, if not all, of the following factors:

- Pavement design life,
- Pavement performance,
- Traffic volume and vehicle class,
- Roadbed soil,
- Materials of construction,
- Temperature changes,
- Drainage,
- Reliability,
- Life-cycle costs, and
- Shoulder design

Figure 9-1
Pavement Terminology



Of these factors, the most influential factors in determining a pavement's required structural strength are the characteristics of the underlying roadbed material, the projected traffic volumes and the percentage and weight of vehicles in the traffic mix using the facility over the expected design life of the pavement. The following sections briefly discuss the factors considered.

9.3.1 PAVEMENT DESIGN LIFE

Each pavement design has a selected design life. Roadway cross section elements and other components of the project such as the pavement structure are expected to remain structurally sound for a designated period of time defined as design life. Although the roadway cross section may become operationally obsolete or the pavement distressed and in need of restoration or rehabilitation, they have not reached the end of their design life but rather have reach the end of a condition defined as service life. Not until they need complete replacement are they considered to have reached the end of their design life.

9.3.2 PAVEMENT PERFORMANCE

The goal of a pavement design is to produce a pavement that when placed will perform functionally and structurally while maintaining its safety characteristics for at least the selected service life.

Functional performance of a pavement identifies how well a pavement will serve the user. The characteristics identified are riding comfort and ride quality. This concept is called serviceabilty-perfomance and provides a means to measure functional performance. In the pavement design procedure, this factor is expressed in terms of the present serviceability index (PSI). PSI is a measurement of roughness and distress of a pavement during the service life of a pavement. Therefore, a reliable method of measuring roughness and maintaining and updating historical performance data is an integral part of pavement design. The major factors influencing the loss of serviceability are traffic, age, and environment.

The structural performance of a pavement relates to its physical condition; including occurrence of cracking, faulting, raveling, or other conditions which would adversely affect the load-carrying capability of the pavement or would require maintenance.

A pavement's safety performance primarily relates to its ability to provide adequate skid resistance during its service-life but also can be affected by its ability to maintain a smooth and rut free surface. Age, traffic, physical properties of materials used to construct the pavement and environmental conditions influence a pavement's safety performance.

9.3.3 TRAFFIC

Traffic volumes using a facility, in particular the number and weight class of trucks, is a major factor in determining how strong a pavement structure must be. In the design procedure, traffic data is reduced into axle loads, axle configuration, and number of applications of these loads. The result is a design number representing the damage done to the pavement caused by the effect a single axle carrying a load on the pavement over its design-life. For the design calculations, traffic data is converted into 18-kip [80 kN] equivalent single axle loads or ESAL's. Since it is one of the more important design considerations, accurate traffic data will ensure that a pavement's design life will be attained and the pavement sections selected will not be over or under designed.

9.3.4 ROADBED SOIL

The pavement structure rests on a graded and compacted roadbed either of suitable natural material or on specified imported material. The roadbed soil has measurable material characteristics that are used in the pavement design. This measurement is defined as a soil's resilient modulus (M_R) and is a measure of the elastic

property of soil. The resilient modulus is used directly for designing flexible pavements but must be converted to a modulus of subgrade reaction (k-value) for the design of rigid pavements or composite pavements. The resilient modulus is also a soil property used in analyzing multilayered material systems for predicting roughness, cracking, faulting, rutting, and other potential distresses.

The value of a roadbed's resilient modulus is dependent on how well the roadbed soil is placed in conformance with the specified compaction parameters. For most projects, the material is to be placed in accordance with the *Standard Specifications* with no special treatment. However, the Soil and Pavement Design Report may indicate that there is anticipated difficulty with the existing roadbed soil meeting the design M_R value. For soils that are excessively expansive the report may recommend these soils be covered by select material sufficiently deep enough to reduce or eliminate the expansive affect of the natural material. Other solutions may include the adding of an admixture to reduce the water content or the use of a geotextile.

One of the more difficult soils encountered on projects are those having a large organic content. These materials are extremely compressible, unstable and frequently non-uniform in properties and depth. These soils are the most complicated and expensive to deal with in order to provide an adequate roadbed. Small, shallow or localized deposits are most often excavated and replaced with suitable material. Deeper and more expansive areas involve more detailed geotechnical design, more complicated construction techniques and costs. Treatments other than complete removal are more time dependent allowing for the slow consolidation and removal of excess moisture. Methods available include surcharge embankments for preconsolidation of the underlying material usually involving sand drains which allow the water to rise to the surface and be removed. The M&R Section is responsible for identifying and designing the most economical method treating this type of problem area.

Underdrains (a system of perforated pipes to collect and transmit the water to an outfall site) are recommended for use on all roadway projects to adequately address drainage and removing water from the roadbed. If site conditions indicate that underdrains may not be required, contact the M&R Section to initiate further investigation.

The soil and pavement condition survey will normally identify roadbed drainage problem areas or soils highly susceptible to expansion or loss of strength with increase in water content. When either of these conditions exist, the M&R Section may recommend additional work and/or materials to address the existing conditions.

Another type of material encountered in constructing roadbeds is classified as cohesionless (sandy) soil and is much more difficult for the contractor to place and compact; it is readily displaced under the load of the equipment. To stabilize this type of soil it may be necessary to blend granular material or add a suitable admixture. Wet clay soils may also be encountered. Because of high moisture content this type of soil is unstable and cannot be compacted. Long periods of dry weather and exposure to the air are required to reduce the water content. To reduce the time necessary to reuse these materials, the recommendation may be to add a suitable admixture that hastens drying or cover the area with a more suitable select material. Removing the material and replacing it with suitable material allowing construction to continue is an option. The material may be used in areas that don't require compaction or moved to an available site for air-drying and reuse at a later time.

9.3.5 PAVING MATERIALS

Depending upon materials that comprise a pavement, the pavement structure is identified as either a flexible or rigid pavement. Combining these two types of paving materials in a pavement structure as a subbase or surface course results in a composite pavement.

9.3.5.1 FLEXIBLE PAVEMENTS

Flexible pavements consist of a prepared roadbed with a subbase of graded aggregate or bituminous concrete and a bituminous concrete base with a surface course. Properly preparing a uniform roadbed compacted to the prescribed density is especially important for providing the design support value necessary for flexible pavement to perform as designed.

The subbase course usually consists of a compacted layer of granular material or an untreated graded aggregate. If additional support strength is needed or the roadbed soil is questionable either of these materials may be treated with an admixture. In some instances, the subbase may be recommended to be a freely draining-highly permeable material to provide a means for water to migrate from under the pavement structure to the side slopes or to an underdrain system. The subbase also prevents intrusion of fine-grained roadbed soils into the base course, minimizes the damaging affects of frost action, and provides a working platform for construction equipment.

The base course is a specified depth of bituminous concrete that is primarily designed to provide the structural strength needed to support and distribute the projected traffic loads.

The surface course is a bituminous concrete mixture placed as the upper course and is usually constructed on a base course. The surface course provides some structural strength. However, the major functions of the surface course are to provide a smooth riding surface that resists distress, minimizes the amount of water that may penetrate the lower more porous layers, provides and maintains its skid resistance for selected service life. To meet these requirements, the surface course mix must have the optimum gradation of aggregate and percent of bituminous binder to prevent raveling, provide durability, resist fracture, and remain stable under traffic use and adverse climate changes.

9.3.5.2 RIGID PAVEMENTS

Rigid pavements consist of a prepared roadbed, a layer of subbase material and a Portland cement concrete pavement slab. The subbase may be either stabilized or unstablized. Although concrete pavements can span failed subgrade and subbase areas easier than flexible pavements, they are still susceptible to damage and failure from excessive moisture in the support structure.

The subbase of a rigid pavement structure consists of one or more compacted layers of granular or stabilized material placed between the roadbed and the rigid slab. The subbase functions to:

- Provide uniform, stable, and permanent support,
- Increase the modulus of subgrade reaction,
- Minimize the damaging effects of frost action,
- Prevent pumping of fine-grained soils at joints, cracks, and edges of the rigid slabs, and
- Provide a working platform for construction equipment.

The pavement slab is composed of Portland cement concrete, longitudinal tie steel, load transfer devices between slabs, and joint sealing materials.

Joint sealing is a critical element in the long-term performance of a rigid pavement. Proper joint sealing prevents infiltration of water under the slab that reduces the support strength of the subgrade and reduces pumping action between slabs caused by the transfer of moving traffic loads between joints. The vertical movement of the slabs eventually erodes the subgrade material through any unsealed joints allowing a slab to break or settle.

There are two types of joint sealants in use, liquid sealants and preformed elastomeric seals. Liquid sealants include a wide variety of materials including hot-poured rubber, elastomeric compounds, and polymers. The materials are

placed in the joint in a liquid form and allowed to set. Preformed seals are extruded neoprene seals having internal webs that exert an outward force against the joint face. The size is determined by the amount of joint movement anticipated.

Transverse joints are needed in a rigid slab to form cracks at desired locations that can be controlled and sealed. Joints can be keyed, butted or tied. Transverse joints are sawed or formed to a depth of one quarter to one third the slab thickness. Timing the saw cutting operation to the curing of the concrete is critical.

Load transfer devices are used between slabs and usually consist of smooth round steel dowels. The dowels should distribute the load stresses without over stressing the concrete surrounding it, offer little restraint to longitudinal movement of the joint, be mechanically stable under the loads and load frequencies expected during the design period, and should be resistant to corrosion from moisture and road salts.

Materials used in the design of shoulders can be either flexible or rigid. Differences in material types and the subbase combined with unexpected wheel loads along the pavement edge can cause joint problems. With proper care and attention, this potential problem can be minimized. Solutions include widening the full depth pavement slab, using tied concrete shoulders, properly sealing the joint, and ensuring compatibility between subbase materials.

9.3.6 TEMPERATURE CHANGES

Temperature changes affect (1) the creep properties of asphalt concrete, (2) thermal-induce stresses in asphalt concrete, (3) contraction and expansion in Portland cement concrete, and (4) freezing and thawing of the roadbed soil. Temperature differences between the top and bottom of concrete slabs create uneven stresses on the slab and can be of concern. Temperature and a poorly drained pavement structure or subgrade, although normally not a concern in Delaware, can combine to create freeze-thaw cycles that rapidly deteriorate the pavement.

9.3.7 DRAINAGE

Keeping the pavement subgrade and soilbed dry is a major design consideration. Excessive moisture combined with increasing traffic and load applications will inevitably lead to premature pavement distress. Water can enter the pavement structure from many directions including a permeable pavement surface, unsealed or poorly sealed joints, surface cracks, high water tables, and even local springs. If water is trapped within the pavement structure, pavement performance will be affected through loss of support due to erosion of any granular material and loss of material strength.

Addressing the problem areas that allow water to enter the pavement structure is difficult to prevent and expensive to correct. In fact it is a shared responsibility. The pavement designer should recommend pavement designs that use dense non-permeable surface courses to reduce surface infiltration, specify underlying material courses that freely drain, and a pavement structure that is strong enough to resist the effects of the traffic loads and water. The roadway designer must ensure proper pavement crown, drainable grade lines, proper ditching and other adequate drainage systems to remove water quickly.

9.3.8 RELIABILITY

Reliability is a method to determine the probability that a particular pavement design will perform as desired during its design life. In selecting the appropriate level of reliability, the pavement designer relates the projected level of usage to the risks involved if a thinner pavement section is recommended.

For facilities with higher importance to the transportation system a pavement design considers the traffic disruption caused by closing or restricting traffic flow due to higher levels of distress, maintenance, and rehabilitation associated with an inadequate or a marginal initial

pavement structure. For lesser facilities, it may be perfectly acceptable and even economical to use a reduced thickness.

9.3.9 LIFE-CYCLE COSTS

Life-cycle costs are costs and benefits that accrue during a pavement's complete life cycle. These costs include the initial construction costs, maintenance costs, rehabilitation costs, resurfacing costs, maintenance of traffic costs, salvage or residual value and user costs.

As a part of the pavement selection decision, particularly when more than one pavement design or rehabilitation procedure is proposed, an economic comparison may be needed. Two methods are detailed in the ASSHTO Guide to determine life cycle cost comparisons, net present worth and equivalent uniform annual cost. Whichever method is used, it is essential that the analysis periods be of equal length.

9.3.10 SHOULDER DESIGN

The inclusion of a shoulder adjacent to the main pavement structure improves pavement performance. The AASHTO guide does not provide a design method for determining the shoulder section. The M&R Section recommends a shoulder section that is compatible with the proposed mainline pavement section, has good constructibility and has performed well in the past. Shoulders are usually designed to carry 10 percent of the projected Average Daily Traffic (ADT).

9.4 DESIGN FOR NEW CONSTRUCTION OR RECONSTRUCTION

Constructing a pavement section is one of the most costly items on new construction and reconstruction projects. In making a selection on the type of pavement to construct it is usually necessary to analyze alternate pavement types and combinations of various support materials. In performing the analyses, the AASHTO procedure requires the pavement designer to provide input in several categories:

- Design variables,
- Performance criteria,
- Material properties for structural design,
- Structural characteristics, and
- Reinforcement variables.

9.4.1 DESIGN VARIABLES

A set of design criteria is established for each project including the pavement's expected service life (performance period) and projected design life (replacement). The traffic projection for the cumulative expected 18-kip [80 kN] equivalent single axle loads (ESAL) during the analysis is determined. The level of reliability is selected and any detrimental environmental factors identified. There is a heirachy of application of these variables with the most important roadways assigned the most stringent and detrimental values to a pavement's performance.

9.4.2 PERFORMANCE CRITERIA

Performance of a pavement is measured by its serviceability to the expected users. The concept is to design a pavement, which at the end of the proposed performance period will still have a predefined minimum level of serviceability (PSI). The terminal level of serviceability is selected based on the lowest index the user will tolerate, or as defined in a pavement management strategy before rehabilitation, resurfacing or reconstruction becomes necessary. DelDOT typically uses a terminal PSI of 2.5 or 3.0, which varies based on functional classification and use.

9.4.3 MATERIAL PROPERTIES FOR STRUCTURAL DESIGN

For the design of flexible pavements, roadbed materials are characterized based on their effective elasticity or resilient modulus, M_R. Their

resilient modulus is determined for periods of stress and moisture conditions simulated for the primary moisture season.

For rigid pavement designs, an effective modulus of subgrade reaction (k-value) is developed. The k-value is directly proportional to roadbed soil resilient modulus. However, the effective design k-value is also dependent upon the effects of the characteristics of the subbase.

Another important input is the compressive strength of the materials composing the pavement structure. In the design of rigid pavements, the modulus of rupture (flexural strength) is determined using the mean value as tested at 28 days. This value is specified in the Standard Specifications and verified as being consistently met or exceeded through laboratory records.

In flexible pavement design, the layer coefficient method is used. Each structural layer is assigned a layer coefficient value which is the relationship between the Structural Number (SN) and thickness of the layer. (See Section 9.5.5.) It is an empirical means to represent the relative ability of the material to function as a structural component of the pavement.

9.4.4 PAVEMENT STRUCTURAL CHARACTERISTICS

Drainage is an important element in the ultimate performance of all pavements. The designer assigns a factor that represents the expected quality of a project's drainage system to minimize moisture intrusion into the pavement structure.

For flexible pavements, the layer coefficients are modified to reflect the expected quality of drainage and percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation.

For rigid pavement design, the level of drainage is addressed through the use of a drainage coefficient. This coefficient represents the quality and effectiveness of drainage systems and the percent of time during the year the pavement structure would be subjected to moisture levels approaching saturation.

For rigid pavements, another structural factor used is its load transfer ability. Rigid pavements have the ability to distribute loads across discontinuities, such as joints or cracks and small voids under the slab. Load transfer devices, aggregate interlock, and the presence of tied longitudinal joints and concrete shoulders influence this value.

Another structural factor applied to rigid pavement designs is a loss of support value. This factor accounts for the potential loss of support arising from subbase erosion and/or differential vertical movement.

9.4.5 REINFORCEMENT VARIABLES

Rigid pavements may be reinforced with joints, unreinforced with joints or continuously reinforced with no joints. The steel reinforcement is used to control transverse cracking.

Joint spacing is particularly important in the performance of plain jointed pavement. The spacing is usually much closer than in reinforced pavements to control cracks due to temperature and moisture enduced stresses. The slab spacing selected must also minimize joint movement thus protecting the aggregate interlock value of the joint. DelDOT has a standard spacing of 20 feet [6 m] for 12 inch [305 mm] Portland cement concrete pavement. (See the Standard Construction Details.)

9.5 STRUCTURAL DESIGN FOR FLEXIBLE PAVEMENTS

A flexible pavement structure may consist of three layers, designated as a subbase course, a base course, and a surface course. A flexible pavement system distributes the load by particle-to-particle-contact by interlocking, friction, and cohesion through its thickness. The surface

course usually consists of a binder course and a surface course.

The concept of all pavement design procedures is to determine the required structural thickness based on the projected traffic loading. Using this as a basis, the most economical and constructable combination of materials is determined. For flexible pavements, this consists of determining the required thickness of the pavement's subbase, base, and surface courses.

9.5.1 SUBBASE COURSE

The subbase course is the portion of the flexible pavement structure between the subgrade and the base course. The subbase insulates the base and surface courses from frost penetration, provides a drainage medium, and a layer resistant to erosion and erosion of fine material into the subgrade. The subbase usually consists of a compacted layer of granular material, which may be either treated or untreated. Subbase materials are either a Soil Cement Base Course, Graded Aggregate Base Coarse or Borrow Type A.

This course has a less stringent specification requirement for strength, plasticity, and gradation. If roadbed materials are of high quality, the pavement design report may recommend that the subbase layer be omitted. If the roadbed materials are relatively poor quality, the design procedure will indicate that a substantial thickness of pavement is required. In this case, alternate designs are usually provided with and without the use of a subbase. The selection of an alternate may then be made on the basis of availability and relative costs of materials suitable for the base and subbase. By using less expensive materials in the lower layer of a flexible pavement structure, the use of a subbase course is often the most economical solution to construction of pavements over poor roadbeds.

9.5.2 BASE COURSE

The base course is the portion of the flexible pavement structure immediately beneath the surface course. The base course is the primary load-spreading layer. It has to be strong enough to withstand the shear stresses produced by the wheel loads, and be incompressible and rigid enough to distribute the load over the underlying materials.

Base course material typically consists of a Graded Aggregate Base Course or Bituminous Concrete Base Course. A graded aggregate base course may be crushed stone, crushed slag, crushed or uncrushed gravel and sand, or other combinations of these materials. These materials may also be used treated with suitable stabilizing admixtures such as Portland cement or asphalt. Base course specifications are generally more stringent than for subbase materials in requirements for strength, plasticity, and gradation.

9.5.3 SURFACE COURSE

The surface course of a flexible pavement consists of a wearing course and a binder course which are mixtures of mineral aggregates and bituminous materials, placed as the upper courses and usually constructed on a base course. In addition to providing a portion of the pavement structural support, the surface course must also be designed to resist the abrasive forces of traffic and weather. They are designed to be dense enough to minimize surface water penetrating the pavement. The proper aggregate selection will provide a non-polishing-skid-resistant surface, resist rutting and provide a smooth and uniform riding surface. The success of a surface course depends to a considerable degree on obtaining a laboratory mixture with the optimum gradation of aggregate and percent of bituminous binder. Open graded mixtures that provide good surface drainage and skid-resistance are available for use on high-speed, high-traffic facilities.

9.5.4 STRUCTURAL NUMBER (SN)

The structural number is an abstract number expressing the structural strength of pavement required for a given combination of the effective resilient modulus, M_R, of the roadbed soils, the total equivalent 18-kip [80 kN] single-axle loads (ESALS), the design serviceability loss, and the standard deviation and reliability factors. The AASHTO Guide provides a nomograph for determining this value. The required SN must be converted to an actual thickness of surfacing, base and subbase by means of appropriate layer coefficients representing the relative strength of the material to be used for each layer.

A design equation is used to solve for the total required SN for the entire pavement structure. By solving the equation with the effective resilient modulus value representative of the roadbed soil, an SN for the entire pavement structure is obtained represented by the general equation:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

Where:

a_1, a_2, a_3 = layer coefficients representatives of surface, base, and subbase course, respectively;

D_1, D_2, D_3 = actual thickness, in inches [mm], of surface, base and subbase courses, respectively (Open-graded surface courses are excluded from this calculation.) and,

m_2, m_3 = drainage factors modifying base and subbase layer coefficients.

The nomograph allows the pavement designer to determine the SN and provides a means through assigning a reliability factor (R) to incorporate some degree of certainty into the validity of the design process. In addition, it allows for assigning a standard deviation factor, which accounts for the variance in the projected traffic capacities and their reliability.

The SN equation does not have a single solution since many combinations of layer thickness will satisfy the equation. However, the pavement designer must use past experience, consider cost effectiveness and construction and maintenance constraints in order to avoid an impractical design.

The design procedure allows for doing a life cycle cost analysis based upon planned rehabilitation. This allows the designer to analyze the tradeoffs between thickness designs of the initial pavement structure and any subsequent overlays.

The procedure allows for considering the adverse effects of changing environmental conditions. The objective is to perform an iterative process to determine when the combined serviceability loss due to traffic and environment reaches the terminal level.

9.5.5 LAYER COEFFICIENTS

The layer coefficient expresses the empirical relationship between the SN and thickness, and is a measure of the relative ability of the material to function as a structural component of the total pavement structure.

To design a flexible alternative, the structural number over the roadbed soil is computed. Then, the structural numbers for the subbase and the base layers are determined. Using the differences between these values, the maximum thickness of any layer can be computed.

The SN for any combination of courses is determined with the design equation, the layer coefficients and the proposed thickness of each course. Alternate designs can be prepared by varying the thickness. The computed SN should equal or exceed the required SN determined from the nomographs in the AASHTO Guide.

The layer coefficients per 1 in [25 mm] of material have been established for various types and classes of flexible pavement, base course, and subbase.

Figure 9-2
Layer Coefficients

Material Type	Layer Coefficient
Type C Surface Mix	0.40
Type B Binder Course	0.40
Bituminous Concrete Base Course	0.32
Soil Cement	0.20
Graded Aggregate Base Course	0.14
Select Borrow	0.08

9.5.6 MINIMUM LIFT THICKNESS

Although the equations allow for a great number of thickness variations, there are the practicalities of constructing and maintaining a facility, which must be considered. Depending upon the material being placed, there are minimum and maximum limits in the placement depth that are practical for the available equipment to compact and are economical.

Minimum lift thickness for hot-mix is recommended to be three times the nominal aggregate size in the mix. Figure 9-3 shows the practical maximum and minimum lift thickness (compacted) that are to be applied to the materials normally used in constructing a flexible pavement section.

9.5.7 TEMPORARY PAVEMENTS

It is not practical to attempt to follow the formalized AASHTO procedures for design of temporary pavements such as needed for detours during construction. Variations in speed and ease of placement as well as the anticipated required service life of the detour significantly affect the economic justification for the structural design.

When temporary pavements are needed designers should closely coordinate with the M&R Section in the development of a practical pavement design based on knowledge of local conditions and engineering judgment.

Figure 9-3
Lift Thickness

Type of Material	Minimum Lift Thickness	Maximum Lift Thickness
Type C Surface Mix (9.5 mm)	1-¼ inches [30 mm]	2 inches [50 mm]
Type B Binder/Base	2-¼ inches [60 mm]	3 inches [75 mm]
Bituminous Concrete Base Course	3 inches [75 mm]	6 inches [150 mm]
Graded Aggregate Base Course	4 inches [100 mm]	8 inches [200 mm]
Soil Cement	4 inches [100 mm]	6 inches [150 mm]
Select Borrow	4 inches [100 mm]	8 inches [200 mm]
Open Graded	1 inch [25 mm]	1 inch [25 mm]

9.6 DESIGN FOR RIGID PAVEMENTS

Rigid pavements consist of a Portland cement concrete slab on a subbase course. The design procedure consists of developing an effective modulus of subgrade reaction based on subbase treatment and thickness, determine the slab thickness, allowing for any stage construction, adjusting for adverse environmental conditions, determining type of joints, joint sealant, and the required reinforcement.

9.6.1 SUBBASE—EFFECTIVE MODULUS OF SUBGRADE REACTION

Before the slab thickness can be determined, it is necessary to determine the strength (modulus), of the material on which the slab will be supported. This is done by determining an effective modulus of subgrade reaction (k), of the soilbed and subbase.

The effective k-value is dependent upon several factors including the roadbed soil resilient modulus, the type of subbase, the thickness of subbase, the potential of loss of support due to erosion of the subgrade and in northern New Castle County whether there is rock underlying the proposed pavement.

The subbase used in a rigid pavement structure consists of one or more compacted layers of granular material, graded aggregate or a stabilized material such as bituminous concrete. This material is placed between the subgrade and the rigid pavement. The subbase provides several very important functions:

* Provides uniform, stable and permanent support,
* Increases the effective modulus of subgrade reaction (k),
* Prevents pumping of fine-grained soils at joints, cracks and edges of the pavement,
* Reduces cracking and faulting, and
* Provides a working platform for construction equipment, especially the paver.

9.6.2 PAVEMENT SLAB THICKNESS

After developing the effective k-value the process of selecting the optimum slab and subbase thickness can begin. Past experience, economics, equipment limitations, ease of construction, and other subjective factors influence the final recommended section(s).

The AASHTO Guide provides a nomograph which provides the slab thickness based on inputting the k-value, the estimated future traffic, the reliability factor to be achieved, the standard deviation, the design serviceability loss, the concrete elastic modulus, the concrete modulus of rupture, the load transfer coefficient, and the drainage coefficient.

9.6.3 JOINTS

Joints are a very important part in assuring a rigid pavement will perform as intended. They allow for the stresses created by the expansion and contraction of the concrete during curing and during seasonal temperature changes. They also are used to facilitate construction.

The Department's Standard Construction Details for Construction provide details for locating and constructing the required rigid pavement joints. The following is a general discussion on the need, use and treatment of joints.

9.6.3.1 JOINT TYPES

The three types of joints used in constructing a rigid pavement are expansion, contraction and construction. Expansion joints provide space for the pavement to expand, preventing buckling of the slabs.

Contraction or weakened plane (dummy) joints provide relief for the tensile stresses caused by the effects of temperature, moisture and friction. Without these joints to control cracking, the slabs would crack randomly.

Construction joints are required to facilitate construction. Although used at the end of a day's pour, they are particularly dictated by the width of the paving machine and the pavement thickness.

Joints may be developed by sawing, forming, or with inserts. When sawing joints, timing is very important to prevent uncontrolled cracking and will vary during the day depending upon the slab temperature, curing conditions, and the concrete mix.

9.6.3.2 JOINT GEOMETRY

Joint geometry refers to the spacing and layout of the joints.

Transverse and longitudinal contraction joint spacing is dependent upon local conditions of materials (coarse aggregate type) and the environment, whereas expansion and construction joints are dependent on layout and construction methods.

Contraction joints are spaced to prevent intermediate natural cracking due to thermal changes and subbase friction created by the movement of the slabs. The type of joint sealant and slab thickness affects their spacing.

Expansion joints are usually used at structures and where pavement types change. They are expensive, complex to construct, and have not performed well in the past. Therefore, they are used only where absolutely necessary.

The spacing of construction joints is a function of the daily construction activities and equipment. They are used when equipment breaks down and at the end of the day's pour. Longitudinal construction joints are placed at lane edges to maximize pavement smoothness and minimize load transfer problems

The width of the joint is a function of the slab length, movement due to opening and closing by temperature cycles and concrete shrinkage. More movement expected at the joint affects the quality and cost of the joint sealant used.

The depth of a joint is selected to ensure that the slab will crack where intended.

9.6.4 REINFORCEMENT DESIGN

Concrete pavements inherently crack. Depending upon the slab length and depth selected, it may be necessary to provide steel reinforcement within the pavement slab. The purpose of the reinforcement is not to prevent cracking but to control the crack width. Excessive cracking allows for moisture intrusion into the subgrade which is the leading cause for distress in the slab.

Stresses leading to cracking are temperature and moisture related contraction of the slab. These stresses are resisted by the subbase as friction and shear between it and the slab. The result is tensile stresses that are minimum at the mid-point of the slab. To resist these stresses and limit the crack width, reinforcement is installed. The AASHTO Guide provides methods for designing the necessary reinforcement for both jointed reinforced concrete pavement and continuously reinforced concrete pavements.

9.7 PAVEMENT DESIGN FOR REHABILITATION OF EXISTING PAVEMENTS

9.7.1 REHABILITATION CONCEPTS

Bituminous concrete pavements deteriorate because of climatic conditions, age, and traffic. Transverse and longitudinal shrinkage stresses occur due to temperature changes. Over time, material problems can develop causing surface problems with stripping, raveling, weathering and bleeding of the asphalt. Repeated traffic loadings eventually cause fatigue cracking allowing moisture into the subbase causing loss of subgrade support leading to pavement cracking or failure.

For composite pavements with both concrete and asphalt as components of the pavement structure, the most prominent problem is reflective cracking from joints and cracks in the concrete base. This is caused by a combination of underlying slab movement due to temperature changes and heavy loads crossing the joints and cracks. The primary resulting distress is spalling of the asphalt as well of the concrete if severe enough.

Concrete pavements react differently depending whether or not they are reinforced. Over time and load applications each reacts different-

ly. The only reinforcement in non-reinforced pavements is tie bars across the longitudinal joints to keep the slab from separating and dowels at the transverse joints to transfer loads across the joints. If dowels are not used, the design depends only upon aggregate interlock to transfer these loads. With loss of subgrade support, cracking of the slab can occur at almost any location. However, more common problems occur at joints. Once the joints are no longer free to move, spalling, buckling, and random slab cracking can result. If joint sealant is lost, aggregate interlock is lost, or cracks become too wide, pumping of the subgrade within the travel lane and the shoulder can occur. This leads to erosion of support material and faulting and cracking

Reinforced concrete pavements perform much like non-reinforced pavements. They usually have longer joint spacing and internal reinforcement to resist the larger tensile stresses. There are fewer joints to relieve stresses; when their free movement is restricted, rapid faulting of the pavement can occur. In addition, normal shrinkage, thermal curl, and load applications cause cracks in the slab that over time grow in width, allowing moisture and road salt to infiltrate. Corrosion of the reinforcing mesh occurs. As the loads are repeated, pumping begins leading to faulting and spalling of the pavement.

Continuously reinforced concrete pavements deteriorate under heavy truck loading and are also adversely affect by moisture under the slab. This type of pavement is more complicated to construct and deterioration will occur due to inadequate consolidation, poor vertical steel placement, and inadequate steel overlap.

Because of the need to maintain the pavement systems now in place, several rehabilitation strategies have been developed to address the various problems that may affect a pavement's performance. The objective is to extend the pavement’s service life. The alternative methods of rehabilitating a pavement range from a simple overlay to complete removal and replacement. When developing a rehabilitation strategy a combination of the following alternatives are considered:

1. Resurfacing to provide structural capacity and/or serviceability either using concrete or asphalt,
2. Replacing or restoring malfunctioning joints,
3. Pavement subsealing prior to resurfacing or as a part of concrete restoration,
4. Grinding rigid pavements to restore smoothness.
5. Removing and replacing deteriorated materials,
6. Reworking or strengthening bases and subases,
7. Recycling existing material,
8. Improving the subdrainage or adding underdrains,
9. Joint and crack sealing,
10. Full depth pavement repair,
11. Partial depth pavement repair, and
12. Cracking and seating.

M&R and other team members develop the most effective strategy through a detailed pavement evaluation. This evaluation is normally prepared as a part of the Department’s long term pavement rehabilitation program and will be a part of the project initiation data. The three steps for determining the preferred strategy are to (1) determine the cause of the pavement distress, (2) develop a list of possible solutions that cure and hopefully prevent reoccurrence, and (3) recommend the preferred solution. The preferred solution includes an analysis of funding, traffic control problems, minimum desirable service life, utility conflicts, clearances to overhead structures, available materials and equipment, contructibility, future maintenance and reliability based on past performance.

9.7.2 TYPES OF DISTRESS

Although furnished at the project initiation stage, each project usually will have a project scoping meeting and preliminary field review prior to beginning detailed design. The field review will include checking the pavement for condition and any damage that may have occurred since the last pavement survey to confirm the proposed strategy is still valid. The following discussion is a brief description of the various conditions the designer may observe depending upon the type of pavement being evaluated.

9.7.2.1 ASPHALT PAVEMENTS

Asphalt pavements usually have the following major distress conditions: (1) alligator or fatigue cracking, (2) longitudinal and transverse cracking, (3) depressions, and (4) rutting.

Alligator or fatigue cracking is caused by repeated traffic loadings. They are a series of interconnecting cracks caused by fatigue failure of the bituminous concrete surface. The crack starts at the bottom of the asphalt surface or the stabilized base due to high tensile stresses and propagates to the surface as a longitudinal crack. After repeated load applications, a network of these cracks form that look like chicken wire or the skin of an alligator. This type of cracking does not occur in asphalt overlays over concrete—only in high load areas and is considered a major structural distress. Pattern cracking not in high load areas is called block cracking.

Longitudinal cracks are parallel to the pavement's centerline or paving laydown direction. They may be caused by: (1) a poorly constructed paving lane joint; (2) shrinkage of the bituminous concrete surface due to low temperatures or hardening of the asphalt; or (3) a reflective crack caused by cracks beneath the surface course, including cracks in Portland cement concrete. Transverse cracks are perpendicular to the pavement's centerline; they are caused by (2) or (3) above and are not usually load-related.

Depressions are localized pavement surface areas that are slightly lower than the surrounding pavement. Depressions are most noticeable during and after a rain. If deep and large enough, depressions may cause hydroplaning or an unpleasant ride. Depressions may be initially built into the pavement by the paving operation or as a result of settling of the surface support structure.

Rutting is a surface depression in the wheel paths. Usually, there is uplift along the sides of rutted areas. Rutting is the result of permanent consolidation or lateral movement of any of the pavement layers or subgrade due to traffic loads. Rutting may also occur because of plastic movement of the asphalt due to high temperatures, poor design mix or inadequate compaction during construction.

In addition to the major distresses, the pavement survey may indicate surface corrugation areas, joint reflection cracking, lane and shoulder drop off, lane and shoulder separation, patch deterioration, polished aggregate, potholes, raveling, and weathering.

If necessary, the M&R Section may determine that field observations are not adequate or need to be supplemented to identify underlying problems. In that case nondestructive testing (NDT) will be conducted. NDT is used to:

- Evaluate the in-situ (in-place) structural capacity of the pavement,
- Evaluate the capacity of joint and load transfer, and
- Detect the presence of voids under the pavement.

9.7.2.2 CONCRETE PAVEMENTS

Jointed concrete pavement may have the following distresses: (1) pumping, (2) longitudinal cracking, (3) spalling of transverse or longitudinal joints, or (4) Alkali-Silica-Reactivity (ASR).

Pumping is the ejection of material through joints or cracks, caused by the deflection of the slab under moving traffic. In some poorly drained pavements particularly in superelevated sections, water may bleed through the joints and cracks after a rain or continuously if large amounts of water are present.

Longitudinal cracks are caused by a combination of heavy load repetition, locking of load transfer devices, thermal and moisture stresses, and curing shrinkage stresses. Cracks that are spalling and/or faulting are considered a major structural problem.

Spalling of cracks and joints is the cracking, breaking or chipping of the slab edges within 2 ft (0.6 m) of the joint or crack. Spalling usually does not extend the full depth of the slab, but intersects the joint or crack at an angle. Spalling is a result of one or a combination of the following: (1) excessive stresses at poorly sealed or cleaned joints and cracks which allow incompressible material to accumulate preventing the pavement from expanding, (2) disintegration of the concrete, (3) weak concrete overstressed by repeated loading, and (4) a poorly designed or placed load transfer device.

Continuously reinforced concrete pavements usually show punchout and patch distress. Punchout is the loss of aggregate interlock between closely spaced cracks. The cracks fault, spall and under load applications the steel reinforcement ruptures causing concrete pieces to punch down. This type of distress is considered a major structural problem. Due to the difficulty in patching continuously reinforced pavements the failure of previously constructed patches can be anticipated.

ASR is another type of distress commonly found in Delaware. The pavements exhibiting the most severe ASR were generally constructed in the 1980's. ASR is the reaction between the alkalis (sodium and potassium) in Portland cement and certain siliceous aggregates. The product of this reaction is a thermodynamically metastable. The gel in the presence of water absorbs it, and causes expansion and cracking of the concrete. Once an ASR pavement has been identified, one possible solution is to apply lithium treatments. Lithium treatment will not repair the concrete, but will slow the further progression of ASR. Normally the pavement will have to be removed and replaced. Overlaying ASR concrete with a standard hot-mix overlay may trap moisture in the slab and accelerate the ASR causing premature failure of the overlay. However, hot-mix overlays with water proofing properties may minimize further deterioration for several years. M&R will recommend the most suitable solution when encountering ASR pavements.

In addition, other pavement distresses that may be observed in a field review are blow ups, corner cracks, depressions, durability "D" cracking, lane and shoulder drop-offs, lane and shoulder separation, patch deterioration, popouts, and staining of the pavement due to subgrade drainage problems.

Concrete pavements can also display roughness caused by irregularities in the pavement surface that adversely affect the ride quality, safety, and vehicle maintenance costs. Roughness is measurable based on the multi-frequency of waves, wavelengths and amplitudes. Roughness can be built into the pavement when constructed or develop over time due to traffic, climate, and other factors. Equipment is used to measure the roughness and a profile developed showing the vertical movement between the trailer axle and body. The results are reported in in/mile or m/km for the International Roughness Index (IRI). The roughness survey identifies areas where severe roughness exists and needs correction. Data provided can be used in developing a PSI which estimates the user's subjective assessment of the pavement condition. Surveys taken before and after a project can be used to document the benefits of the work to the traveling public.

9.7.3 DRAINAGE SURVEY

As emphasized throughout this chapter, the presence of moisture is a primary cause of distress or failure of all pavements. Therefore, a drainage survey is an important component of pavement evaluation. Moisture conditions are caused externally by the climatic conditions and internally by the properties of the materials composing the pavement structure. The severity of damage caused by excessive moisture will influence the decision on which rehabilitation strategy to select.

Since moisture problems can exist in any layer of the pavement structure, more than visual observations may be needed. Cores and even nondestructive deflection testing may have to be conducted. It is necessary to determine which material is responsible for the moisture-related damage and if an economical rehabilitation to correct the problem is to be initiated. Not identifying and correcting the problem could lead to a failed project. A valuable tool in this evaluation is the as-built plans.

In addition to determining if the pavement structure is freely draining and moisture resistant, the entire roadway section should be evaluated including:

1. Are the ditchlines free of standing water? If not, how high does it stand and will it infiltrate the pavement structure?
2. Are ditchlines and pavement edges clear of the type of growth that would indicate excessive moisture?
3. After a rain, is water standing in the joints or cracks. Is there pumping, is there standing water adjacent to the pavement or on the shoulder?
4. If there are drainage outlets including underdrains, are the outlets clear, at the proper elevation, and working?
5. Are drainage inlets clear and cross slopes adequate to remove the water from the pavement surface?
6. Are joint and crack sealants in good condition and preventing surface water infiltration?
7. Are there signs of pumping, such as pavement discoloration or the presence of fine material at joints or pavement edges?

Recommending drainage improvements to the pavement structure can be a very expensive item and should be carefully evaluated and documented.

9.7.4 RESTORATION

Restoration of a pavement includes the work required to return the pavement's level of serviceability for a designated time period. Frequently, some level of restoration is performed prior to an overlay or resurfacing. Work undertaken to restore pavements is quite varied and includes:

- Full-depth repair of jointed concrete pavement
- Full-depth repair of continuously reinforced concrete pavement,
- Patching with bituminous mixtures,
- Partial-depth spall repair,
- Slab stabilization and slab jacking,
- Diamond Grinding, grooving and cold milling,
- Pressure relief joints,
- Load transfer restoration,
- Joint and crack sealing,
- Surface treatments,
- Subdrainage, and
- Shoulder improvements.

9.7.4.1 FULL-DEPTH REPAIR

One of the most expensive restoration alternatives used on all types of pavements is full-depth repair. To limit the repair areas and the

potential of overruns, the specific distress or distresses to be addressed should be clearly established both during the design and construction phases. Depending upon the severity, distresses that may lend themselves to full-depth repair are blow-ups, corner breaks, durability "D" cracking, ASR, excessive spalling, and loss of load transfer. Intermediate working cracks may also have to be repaired by full-depth replacement or a working joint. Not addressing areas that need full-depth repair prior to an overlay could result in continued deterioration and premature failure of the overlay.

The major considerations to ensure satisfactory performance using this rehabilitation approach are:

1. Joint design,
2. Selection of the repair areas and their boundaries,
3. Preparation of the repair area,
4. Placement and finishing of the repair material,
5. Joint sealing material and its installation, and
6. Curing time and traffic control.

Usually the full-depth repair material is the same as the adjacent pavement. However, funding, traffic control or other reasons may require that concrete pavement be repaired with bituminous material. Using materials of different physical properties and characteristics can cause several problems. Differentials in expansion and contraction lead to pushing, shoving and humping requiring frequent milling to maintain rideability. The bituminous patch is more compressible and may allow excessive opening of remaining joints in the concrete pavement resulting in spalling, pumping, and faulting. In overlaying the patched pavement, there may be an increase in reflective cracking from the underlying joints The patch design may not have an equivalent structural strength. The difference in initial cost and possible future maintenance problems may not make using dissimilar materials the best solution.

9.7.4.2 PARTIAL-DEPTH REPAIR

Some distresses within the upper third of the slab in concrete pavements lend themselves to partial-depth repairs. Partial-depth repairs may or may not be more cost effective than full-depth depending upon the size, location, number, materials used, lane closure time and production limitations.

The distresses applicable to partial-depth repair are:

1. Spalls due the use of certain types of joint inserts,
2. Spalls caused by joint movement locking due to failed sealant and subsequent intrusion of incompressible material,
3. Spalls caused by misplaced dowels or other load transfer devices, and
4. Localized areas of scaling.

The M&R Section should be contacted for details on partial-depth patches as they require the use of better designs and construction techniques to be successful.

9.7.4.3 SLAB STABILIZATION AND SLAB JACKING

Although not that frequently used, voids under pavements can be filled, restoring the support strength of the pavement structure. The loss of support is caused by erosion of the subbase and/or subgrade by pumping or, in severe cases, movement of freely flowing water. Slab stabilization does not increase a pavement's structural strength, correct depressions or faulting and other distresses.

A slab that has settled may be raised through a process called slab jacking, also known as mudjacking. This procedure which injects water-soil-cement slurry (i.e. grout or mud) through holes drilled into the pavement slab under high pressure lifting the pavement back into its origi-

nal profile is also used infrequently. A process using expanding polyurethane foam may also be used. Another alternative is the injection of a flowable fill that consists of a mixture of fly ash and water that expands and hardens as it dries out. If not correctly done, all three of these methods can crack the slab.

9.7.4.4 DIAMOND GRINDING, GROOVING AND PAVEMENT MILLING

Diamond grinding is used to restore a concrete pavement's smoothness using closely spaced diamond-impregnated blades. Diamond grinding is effective in: (1) removing joint and crack faulting, (2) remove wheel path ruts, (3) correct joint unevenness caused by slab warpage, (4) removing areas of roughness and (5) restoring transverse drainage. Diamond grinding does not correct the problem that created the distress and may have to be use in conjunction with other rehabilitation techniques. Subsealing, slabjacking, full-depth, and partial depth repairs should be completed before grinding.

Grooving of concrete pavements is used to reduce hydroplaning by improving surface drainage and improving surface friction on curves or polished aggregate surfaces.

Pavement milling is used to remove asphalt surfaces as much as 3 to 4 inches [75 to 100 mm] in depth using carbide-cutting teeth mounted on a rotary drum. If used on concrete pavements, pavement milling leaves an extremely rough surface and spalled joints. Therefore, unless an overlay is included in the work, it should not be used on concrete.

Pavement milling is a very effective rehabilitation technique addressing several problems including:

1. Restoring the curb line of asphalt pavements,
2. Restoring cross slope and correcting inlet drainage problems,
3. Improving the friction resistance of asphalt pavements,
4. Removing asphalt overlays on concrete pavements,
5. Providing a good bonding surface on concrete for overlaying,
6. Removing material as part of a recycling project, and
7. Restoring pavement smoothness.

All three of these restoration techniques are cost effective, relatively quick and proven to perform well but do not improve the structural integrity of the pavement.

9.7.4.5 PRESSURE RELIEF JOINTS

As concrete pavement ages, there can be a net increase in its length. These increases can result from: poorly sealed joints and cracks filled with incompressible material, pumping of the base material into the joints and cracks, or the use of expansive or reactive aggregates in the initial mix.

Because pressure relief joints can adversely affect contraction joints, they are not usually installed unless there are major blow-up problems or the pavement has expanded to the point of shoving bridge abutments. These type of joints provide no load transfer and are also subject to closing through the intrusion of incompressibles, eventually becoming part of a pavement distress problem.

Pressure relief joints are full-depth and full-width saw cuts at mid-slab and are 2 to 4 inches [50 to 100 mm] in width. The joint must be cut across all lanes within the same time period to relieve the compressive stresses and is usually done at night under reduced traffic flow.

9.7.4.6 LOAD TRANSFER RESTORATION

Poor load transfer between joints can cause spalling, rocking, pumping, faulting, and corner breaks. Doweled joints are effective in providing

load transfer but under repeated load applications can work loose and lose this capability.

Whether or not joints and cracks have lost their load transfer ability, is determined through field observation and measurements conducted in the cooler parts of the year or day. The Falling Weight Deflectomator (FWD) may be used to test for load transfer capability at joints.

There are two techniques used to restore load transfer, both of which require the installation of dowels. Slots can be cut in the pavement, if the adjacent pavement is sound, and the dowels installed. More frequently and, usually a better alternative, load transfer is restored through full-depth repair.

9.7.4.7 JOINT AND CRACK SEALING

One of the easiest, cost effective and proven techniques in maintaining serviceability and restoring concrete and asphalt pavements is sealing and resealing joints and cracks. Inadequate or failed sealant allows free water and incompressibles to enter the pavement structure causing erosion of pavement support and blow-ups.

Joint sealants are constantly being improved and may be field poured—self-leveling, hot-poured, cold-poured, preformed compression seals, or field-poured—nonself-leveling sealants.

The performance of a sealant depends primarily upon the proper preparation of the joint to receive the sealant. The factors include the shape of the reservoir created for the sealant, the bonding ability of the sealant to the sidewalls, the surface preparation, the cleanliness of the surface, the dryness of the surface and the properties of the sealant. In addition, it is necessary to predict the amount of movement expected in the crack or joint to customize the type of sealant to fit the distresses encountered and techniques to be implemented.

Cracks occur randomly and are irregular in dimension and direction. Usually cracks do not experience the faulting and movement that happen at joints and sealing is not as tightly controlled. If the field survey indicates the width of cracks are large enough to cause large movements, then the cracks are sealed and treated similar to joints.

9.7.4.8 SURFACE TREATMENTS

The use of surface treatments or seal coats is a very effective way to rehabilitate. The technique is to apply asphalt and/or aggregate to a roadway surface at a depth of less than 1 inch [25 mm].

Surface treatments are classified by their composition and include:

Asphalt Chip Seal (Surface Treatment)—one or more applications of asphalt and stone chips.

Open-Graded Friction Course— the asphalt and aggregate mix is designed to be freely flowing to remove surface water thus reducing hydroplaning. In addition, it provides some improvement in skid resistance.

Micro Surfacing—a process using a moving pug mill which mixes latex rubber with an asphalt emulsion, aggregates and other additives that is placed on the surface.

Slurry Seal—a diluted emulsion mixed with a sand-size aggregate in a special mixer, and squeegeed onto the pavement. The thickness is usually less than one half inch [12 mm].

9.7.4.9 SUBDRAINAGE IMPROVEMENTS

The long-term presence of water within the pavement structure is largely responsible, directly and indirectly, for many of the distress and performance problems, which are found in the pavement systems. A pavement survey and evaluation includes a very detailed study of the drainage and subdrainage within the entire roadway cross section. Considerable design effort and cost are involved when correcting this

type of problem. Unless the rehabilitation program involves complete pavement reconstruction, improvements are limited to design and construction of longitudinal drains. Any modifications to or additions of transverse drains or drainage blankets would be limited to projects involving complete pavement replacement. However, the addition of longitudinal perforated underdrains to collect and outlet excess moisture in the pavement structure is a feasible and cost effective option.

9.7.4.10 SHOULDER IMPROVEMENTS

Shoulders are evaluated much like the mainline pavement and are include in the distress survey, drainage survey, traffic survey, structural evaluation, and subgrade and materials evaluation. Shoulders provide not only safer traffic flow but also give lateral support for the mainline pavement structure.

Shoulders can be rigid or flexible and distress similar to mainline pavements. Many of the same rehabilitation techniques are used. Reviewing the existing cross section can be very valuable. If the initial shoulder pavement design was not compatible with the mainline pavement structure, it may be a major contributor to its failure.

Two of the most common problems which occur are lane/shoulder joint separation which allows water into the subbase, and blockage of water draining out of the mainline subbase which is usually of more granular and higher quality. The material used in shoulder construction also may be of a different thickness. All of these affect the interaction between the two pavement structures.

Other distresses found in shoulders are pumping, fatigue cracking, lane/shoulder drop-off, frost heaving and differential shoulder support.

9.7.5 RECYCLING

Recycling is the term used to describe the process, which uses existing pavement materials to construct new pavements. The primary purpose is to conserve natural resources. In some cases, there may also be a net cost savings.

9.7.5.1 RECYCLING RIGID PAVEMENTS

Eventually, all pavements reach the end of their useful and/or structural life and must be reconstructed. This occurs when there is little or no structural life left due to extensive cracking, extensive slab settlement or heave, extensive joint deterioration requiring excessive full-depth repair, extensive concrete deterioration due to poor durability, and failure to meet geometric design standards.

Two methods of recycling are: to break the pavement slabs into smaller sections and leave them in place (rubblizing) as a base for resurfacing; or to break and remove the pavement slabs to an area for crushing and reuse. The crushed pavement material may be reused as aggregate in untreated dense graded aggregate bases for Portland cement concrete surfacing, asphalt concrete surfacing, fill, filter material, and as a drainage layer for edge drains.

Each method of recycling has its own cost and feasibility studies that need to be conducted before making a selection. Both methods involve the use of vibratory, hydraulic, pneumatic, or diesel chisels or hammers to demolish the existing pavement. Common factors to consider are: is the resulting recycled material actually reusable; are the underlying soils adequate to support an upgraded pavement; and are there shallow utilities (older gas, water or sewer lines such as vitrified clay or cast iron) that are sensitive to the equipment pounding and vibration and may rupture during the demolishing process.

The pavement may be demolished, removed, crushed and used for other construction purposes. This involves hauling the material to a crushing plant and having an electromagnet remove any reinforcement. The crushed material and salvaged steel would be available for reuse. The coarse-aggregate-sized particles resulting from the crushing process have a good shape and an-

gularity, high absorption and may have a lower specific gravity than virgin aggregates. For reuse of this material the concerns are the possible contaminants in the rubble including reinforcing steel, dowel bars and baskets, admixtures, chemical substances such as deicing salts, sea salt, oil, joint sealant material and soil.

9.7.5.1.1 RUBBLIZATION

Rubblization is initially the most expensive pavement recycling method, however, a selective use of it can be very effective in cost and construction time.

When rubblizing plain or reinforced concrete pavement, the rubble is leveled and left in place as the base for a new pavement. Maintaining traffic is a consideration as there must be enough area for the equipment to operate as well as a protective barrier. Even on a four lane facility this will require a median shoulder strong enough to temporarily carry full traffic loads or possibly a detour.

The rubblized surface essentially remains at the same elevation as the existing pavement. Therefore, matching existing curbs, drainage structures, intersecting roadways, and driveways is more complicated and time consuming. In addition, it may become difficult to meet height standards for guardrail and other appurtenances, as well as maintain vertical clearances for structures.

Rubblization is for total reconstruction. Rubblization reduces the structural value of a PCC pavement to a stone base. It requires a thick overlay, typically 11 inches [280 mm] of asphalt or 10 to 12 inches [255 to 305 mm] of concrete. It is a major reconstruction technique and therefore it should be used only when the pavement has reached the end if its service life, as indicated by severe deterioration, ASR, or severe freeze and thaw damages, etc.

Rubblization can be used when other concrete pavement restoration methods will not work. Thoroughly evaluate the existing condition of PCC pavements. Concrete pavement restoration (CPR) techniques, such as diamond grinding, patching or sealing, should be ruled out before the use of rubblization is specified. Pavement Management and M&R should be consulted in the selection of reconstruction/restoration technique.

Rubblization cannot be used over a subgrade demonstrating widespread instability or of poor condition. Many concrete distresses result from poor subgrade support conditions. Rubblizing a pavement destroys the concrete slab's bridging action, causing problems to become more pronounced. The poor support condition could be due to poor soils, poor drainage or high moisture content. If the problem is widespread, rubblization cannot be used. Thorough subgrade investigation is essential for the successful application. Contact M&R for subgrade investigation.

Treat rubblized PCC pavement as a subbase. Although rubblization provides the benefits of an in-place recycle opportunity and an interlocked stone base, it has challenges as well. As for all subbase materials, gradation and density are two important factors, but the control of these two factors is more difficult since it is in-place recycling.

The pavement breaker may be powerful, but as energy dissipates through the depth of the slab, it produces smaller pieces at the top and larger pieces at the bottom. A soft subgrade or the reinforcement in the slab only compounds the difference in sizes. The recommended requirement is for the top pieces to be 3" maximum and 12" maximum at the bottom. Acceptance of a larger size will increase the probability of future reflective cracking. A good density is achieved through the interlocking and good compaction.

With good control of gradation and density, it is reasonable to expect a good fatigue resistance performance of an asphalt overlay, which is a major controlling factor for a flexible pavement service life.

Rubblization is a balancing act. The size of rubblized concrete can be controlled. A larger size will provide a stronger support (thus a better structural value), but it increases the likelihood of reflective cracking (thus a reduced service life). The designer needs to balance these two in the pavement design. A long term performance evaluation to validate this design value may be necessary.

Install a drainage system. An adequate subgrade drainage system is essential for rubblization and future performance of the pavement. Rubblization cannot be successfully done over a wet subgrade. The drainage system should be in place at least two weeks prior to the rubblization. In a special case, if the drainage system cannot be installed prior to actual rubblization, then a time limit should be specified to have the drainage system installed immediately following rubblization.

Use a test section, not a second pass of the breaker. A second pass over a rubblized area does not enhance the quality of rubblization, and it could cause more damage. Test sections should be done to calibrate all rubblization variables (machine related-velocity, frequency, pressure or force, shoe size and conditions related-concrete condition or state of distress, thickness and subgrade conditions). The objective is to achieve the required sizes of rubblization both at the top and bottom of the PCC pavement for a good service life.

Selection of Rubblization Equipments and Production Rates - Among the different types of equipment for breaking the pavement, two frequently used types are resonant pavement breakers (a low impact, low-amplitude, high frequency vibration to the slab) and multiple head breakers (12 to 16 drop hammers mounted laterally in pairs with half of the hammers in a forward row and the remainder diagonally offset in a rear row). The multi-head breaker rubblizes a full lane width in a single pass with a production rate of about one lane mile per shift per day, while a resonant breaker may take up to 20 passes for a full lane width with a production rate of about ½ miles per shift per day.

The multi-head breaker may cause damages to the subgrade. the resonant breaker may produce concrete size too small at the top to meet the design requirement. Unless the equipment selection is specified on the plan, the specification will allow the use of either breaker.

A concrete overlay can be used. Rubblization was developed to eliminate reflective cracks in the asphalt overlay. Engineers still have an option of using concrete overlay, considering cost, service and construction time. Consult M&R for pavement design options.

Soft spots need to be repaired. Original PCC pavement could bridge the soft spot and this soft spot will show up after rubblization. Any depression one inch or greater in depth from the immediate surrounding area should be examined to see if it is due to poor underlying subgrade before the application of filler aggregate as required by the specification. A bearing capacity failure during rubblization could cause depression on one area and heave in other areas.

The repair of soft spots is necessary not only for a long term performance of the pavement but also for a good working platform for paving operations.

Pavements with delamination type cracks should not be rubblized. Horizontal cracks hinder the rubblization process by absorbing energy and decreasing the effective depth of rubblization.

Survey and set a new profile. Although rubblization does not significantly change the existing grade, simply specifying a few inches over the existing grade may not be adequate. The existing PCC pavement may not have adequate cross slope, or it is distressed due to poor subgrade which might have resulted in an irregular profile. Comparing survey results with the original geometry could provide clues on distress or subgrade conditions.

The design service life should be 20 years or more; this should be noted on the plans. Rubblization with an overlay is major reconstruction. It should not be used as a short-term fix.

9.7.5.2 SURFACE RECYCLING OF BITUMINOUS PAVEMENTS

Surface recycling is the process that either reworks or removes and replaces a limited portion, usually about one inch [25 mm], of the asphalt surface. The distresses addressable through this process include: raveling, flushing or bleeding, low surface friction, weathering, poor drainage profile, shallow rutting, minor corrugations and block cracking. It may also be used to correct problems in the profile grade line.

The major advantage for selecting this rehabilitation method is the minimal amount of work involved. There are two processes. The first is hot surface recycling involving heating, scarifying, remixing, and repaving recycled material. Other surface recycling methods include hot pavement removal using a heater-planer or cold milling using a rotary drum equipped with closely spaced carbide teeth.

The material to be reused has a rejuvenating agent and soft asphalt added to restore the pavement consistency, viscosity or penetration. DelDOT does not typically use this method for recycling surface courses.

9.7.5.3 IN-PLACE RECYCLING OF BITUMINOUS PAVEMENTS

In-place recycling is a process in which the pavement surface is ripped up or pulverized to a depth greater than 1 inch [25 mm]. The material is cold worked and reused as an aggregate base. The recycled material may be further strengthened by the addition of admixtures such as asphalt, lime, cement or fly ash. The recycled material will perform similar to new stabilized material upgrading the structural capacity, correcting surface distresses and mixture problems in the asphalt pavement, and correcting base course problems such as gradation, moisture and density. Correction of profile grade problems can be made with this material.

9.7.5.4 HOT-MIX RECYCLING OF BITUMINOUS PAVEMENTS

Hot-mix recycling is the process in which all or some of the pavement structure is removed, reduced to the required size, and mixed hot with added asphalt cement at a central plant.

This process is used to correct surface roughness, cracking, rutting, surface friction, raveling, inadequate structure, and inadequate traffic capacity. It should be remembered that the underlying cause for structural inadequacy would not be corrected using this process.

9.7.6 RESURFACING

The most common type of rehabilitation for existing pavements is resurfacing. Resurfacing can correct many common distresses and add additional strength to the pavement structure.

Problems encountered with overlay projects include inadequate thickness to correct surface problems when the problem is actually structural, inadequate repair of the deteriorated areas, unanticipated increasing traffic loadings, and not addressing reflective cracking.

9.7.6.1 TYPES OF OVERLAYS AND THEIR FUNCTIONS

Overlays can be of asphalt or Portland cement concrete. There are several variations of these overlay techniques, which are designed for specific applications.

The most commonly used overlay is dense-graded, hot-mixed asphalt concrete, which may be used on existing asphalt or Portland cement concrete pavements.

Portland cement concrete overlays are designed specifically for the type of existing

pavement and may be unbonded, partially bonded or fully bonded.

In analyzing overlay alternatives, the designer should look at how extensive are the proposed pre-overlay repairs, structural adequacy of the existing pavement, subdrainage, existing distress conditions to be corrected, future traffic loadings, traffic control problems, clearances to overhead structures, and overall costs.

Asphalt overlays of rigid pavements are used both to correct surface problems and to improve the structural strength. Thicknesses greater than 2.5 inches [65 mm] are needed to provide improvements in strength. The greater the overlay thickness, the higher the possibility of rutting if compaction is not controlled but a significant reduction in reflective cracking can be expected.

Reflective cracking occurs in overlays due to thermal cracks in flexible pavements and joints in rigid pavements. Low temperatures, temperature cycles and traffic loads cause movement in the existing pavement leading to stresses reflecting through the overlay.

Another method to reduce reflective cracking in asphalt overlays of rigid pavements is to fracture the underlying slabs into pieces 2 to 3 feet [600 to 1000 mm] in size. After cracking the slab, a heavy roller is used to ensure the slabs are fully seated before the asphalt overlay is placed.

Reflective cracking leads to increased infiltration of water into the pavement structure that rapidly deteriorates the overlay. This creates potholes and other distresses. Treatments to minimize reflective cracking include the use of reinforcing fabrics, stress-relieving interlayers of rubber-asphalt with aggregate chips, crack-arresting interlayers of large aggregate bituminous material to blunt the cracks, and extensive pre-overlay repairs including determining existing joint and major crack locations, sealing them and saw cutting after paving.

Portland cement concrete overlays of existing rigid pavement may involve extensive pre-overlay repairs, are more difficult to construct and have added initial traffic control costs. These overlays may be fully bonded, partially bonded and unbonded.

Fully bonded concrete overlays require that the existing pavement be in good condition and that a complete and permanent bond be achieved. Fully bonded overlays are thinner and range from 2 to 4 inches [50 to 100 mm] in thickness.

Partially bonded concrete overlays require repair and/or replacement of damaged slabs. Surface cleaning by sweeping and either water blasting or sand blasting is necessary to achieve as much bond as possible. These overlays are usually between 5 to 7 inches [25 to 175 mm] in thickness.

Unbonded concrete overlays are used to improve the structural capacity of a rigid pavement in poor condition. A bond breaking and leveling course is placed between the existing pavement and overlay to assure there is no bonding and to absorb any movement in the base slab to prevent cracking of the overlay. Unbonded overlays are thicker and range from 8 to 10 inches [200 to 250 mm] in thickness.

All types of overlays normally require a fairly detailed design analysis. The analysis involves determining the initial construction and pavement information, a pavement distress survey, existing layer properties, future traffic analysis, existing pavement subgrade properties, overlay design properties, determining effective structural capacity, future overlay structural capacity, desired remaining service-life, and required overlay thickness design.

Chapter Ten

MISCELLANEOUS DESIGN

This chapter contains guidelines and criteria for the design of roadway elements that do not logically fit within the general categories of information covered in other chapters.

10.1 CONTEXT SENSITIVE DESIGN

Context sensitive design is a term used to identify a design process that balances the design features of a project with:

- User safety,
- Transportation system needs,
- Accessibility and mobility,
- Preservation of historic sites and districts,
- Natural and man-made environmental concerns,
- State and local economic needs, and
- Preservation of community values

Context sensitive design recognizes that the application of uniform engineering standards to specific transportation projects is not always possible. Transportation facilities are not constructed through consistent, uniform settings. In reality, landscapes are constantly changing and preventing the easy application of engineering standards. AASHTO and this manual respond to this problem by developing different sets and ranges of standards to apply to the most common types of landscapes encountered. There are standards for rural and urban environments, different types of terrain, and varying functional classes of highways. However, the designer still finds it difficult to always meet these variable standards. Historic sites, sensitive environmental areas, such as wetlands and natural areas and community concerns along with the mixing modes of transportation such as pedestrians, bicycles, and public transit, within the same right-of-way often conflict with the routine application of established engineering standards.

The context sensitive design approach is used to develop projects that achieve protection of community, historic, and environmental values while utilizing scientifically developed engineering standards that will result in a legally defensible, safe facility for the user. This process allows the designer to address community concerns while meeting the intended transportation needs, minimizing adverse impacts and enhancing the project area. Context sensitive design requires careful listening and understanding of the values placed on the project area by citizens, agencies, special interest groups, and local governments. It requires thoughtful and often creative design solutions.

10.1.1 TYPES OF PROJECTS

There are many types of projects in the Department's annual Transportation

Improvement Program. Projects are developed to preserve the integrity of the current system, to restore or increase the capacity of the system, to improve the safety of the system, to maintain suburban streets, and address other concerns in communities and municipalities.

For the highway system, the types of improvements used to expand capacity are new construction on new alignment and reconstruction of an existing roadway in-place, involving adding lanes and other capacity improvements. They also cost the most and have the greatest social, economic and environmental affects on adjacent areas. Projects on new alignment often do not have the land use restrictions or the historical, operational and safety problems associated with improving an existing roadway. This means that higher design standards can be attained. In contrast, reconstructing an existing roadway in-place to address capacity, safety and operational problems is difficult without introducing flexibility into the design. Since significant geometrical changes are not normally viable, historical data, such as traffic growth, traffic patterns, and types and frequency of accidents, become an important part of the decision-making process in assessing and correcting existing problems. Because of the difficulties in making changes, particularly in vertical and horizontal geometry, this analysis will determine if not meeting the highest design criteria is an acceptable alternative.

There are other types of projects ranging from restoring and/or maintaining a facility's capacity to resurfacing existing roadways to maintain their rideability. Many of these projects lend themselves to flexibility in choice of design options.

Ensuring that a project will be designed within the context of the community meeting the expectations of both the motorist and the community at large is the result of an early, continuous and meaningful public involvement process. Through a continuous public involvement process, the designer is assured that the purpose and need of the project is fulfilled, the needs of the community are understood and addressed, and new or additional issues do not arise during the final design or construction phase.

10.1.2 DESIGN STANDARDS

Each project should begin with determining the applicable design standards as published by AASHTO and this manual. In the context sensitive design environment, it is recognized that there are limitations, constraints, community values and other factors that require the designer to look beyond the full standards for a workable solution. Meeting the full standards for an entire project may not be possible but design alternatives that combine multiple substandard dimensions simultaneously are not acceptable. Arbitrarily lowering a project design speed to solve a problem in a sensitive area is also not an acceptable alternative. All major geometric values are based on design speed and lowering this value would preclude the use of appropriate standards where they are attainable. Where a non-standard element, dimension or approach is used, the potential adverse operational affects should be mitigated by wider paved areas, appropriate advanced signing, increased vertical or horizontal sight distance or other treatment.

The selected design criteria should properly reflect driver safety, desires, expectations, comfort and convenience. Of course there are many contraints, including terrain, land use, roadside and community effects, and cost considerations. A project is designed based on accepted design criteria, practices, guidelines, and standards. Appropriate dimensions and values are selected to produce a facility of a given quality, provide a reasonable degree of safety, and consistent expectation (standard design) for the user. Transportation Research Board’s Special Report 214

Designing Safer Roads is a valuable resource document when evaluating the various design parameters in reaching a safe and flexible set of design standards meeting project intent.

10.1.3 OPERATIONAL CONSISTENCY

Although, designs may not meet all of the applicable design standards, they can provide design/operational consistency. Ensuring the continuity of designs means that a motorist can travel to any state, city or town and, depending upon the situation encountered, react in the same driving manner e.g. all ramps are signed uniformly and have deceleration lanes. In responding to the many issues that each project faces, there is a need for flexibility in the design process, while maintaining uniform design practices. Flexibility is achieved by recognizing outstanding issues and making changes while recognizing the tradeoffs incurred. Flexibility in design should not unduly compromise the user's safety. Each project is unique and has its own community values, social, economic, and environmental constraints. Context sensitive designs recognize and address those unique elements that preserve or enhance community values.

AASHTO design standards do have a measure of flexibility, usually stating a maximum and minimum value. Many of these values were established many years ago with assumed conservative variables, some derived theoretically and others empirically. Design features that fall outside normal design criteria and accepted practice should be documented, if necessary, with a formal "Design Exception". The key to minimizing liability is documentation of major design decisions in terms of safety, capacity, route compatibility, time to construct, environmental, historic, and aesthetic considerations, and construction costs throughout the project development process. Many design options have operational experience from past projects, including accident history.

10.1.4 DESIGN CRITERIA

Design criteria have historically provided consistency in the quality, appearance and operational performance of the highway system. They are based on data available for successful operational performance, as well as theoretical modeling and field-tested research. AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book) and periodically issued supplemental reports and guidelines catalogue this data for use by designers. The Green Book contains nationally accepted design criteria for new construction or reconstruction projects that are generally applicable for all 50 states.

This manual utilizes criteria contained in the Green Book to define design standards to address transportation needs and community values as envisioned in this State. Design data in the Green Book is intentionally developed to be conservative because highway alignments are of a permanent nature, and projects are initially expensive to construct and very difficult and expensive to correct if there are safety problems. The public expects the design professional to design it right the first time, meeting the users' operational needs and safety expectations.

Since the criteria is conservative in many areas with a significant margin of safety, there is room for creative yet safe designs when a designer has to look beyond the criteria to meet a constraint or issue that arises. Again, documentation of accepted changes in design criteria is essential.

10.1.5 DESIGN CONTROLS

Design controls include: functional classification, design speed, traffic volumes, traffic mix, terrain and location. The design team, working with the public, can identify constraints that will require flexibility in the design criteria.

10.1.5.1 FUNCTIONAL CLASSIFICATION

A facility's functional classification and design speed are two major factors in setting the design criteria. The functional classification groups streets and highways based on the type of trips, mix of traffic, accessibility to the facility and the role of the facility in the total transportation system. The functional classification system establishes a heirachy based on the level of service to be provided for the users. This service ranges from the high level of service provided by the interstate system to local streets. Local streets provide a low level of service for through traffic movements but support safe local access and address community mobility needs.

The AASHTO Green Book relates the functional classification to design criteria, particularly design speed and geometrics. As a result of variances of design criteria within the functional classification system, there are overlapping ranges of values. This allows flexibility for choosing a design that is most appropriate within the determined functional classification. Land use is very dynamic, thus changing the character and use of a facility leading to a functional classification change. At the same time there may be strong support for the preservation of the existing character of a facility. The development group will become aware of this during the public involvement phase of both the planning and/or design processes.

10.1.5.2 SPEED SELECTION

In design, there are three speed concepts used: running speed, operating speed and design speed. The difference between these three values can be significant. Running speed is the actual speed of a vehicle over a specified section of highway. For evaluating road-user costs and benefits, the average running speed is used. This value varies during the day based on the traffic volume which, depending upon the roadway characteristics may have to be field measured at several locations to truly reflect the average running speed. Operating speed is the speed at which drivers are observed operating their vehicle during free flow conditions. The 85th percentile of the distribution of observed speeds is used to statistically describe the operating speed associated with a particular location or geometric feature. Design speed is a selected speed used to determine the various geometric design features of the roadway.

Design speed usually controls the design features. Selecting a design speed is not a simple task. Using a design speed as high as practical may ensure that drivers can drive as fast as comfort level will permit but may not be the best method in determining the design speed. In practice, design speed is selected to accommodate a high percentile of drivers.

The determination of a design speed is affected by many factors including: the capabilities of the drivers and their vehicles, physical characteristics of the roadway and its roadsides, the weather, the presence of other vehicles, and speed limitations. Selecting a higher design speed imposes more design constraints. Selecting a design speed based on an artificially low operating speed that does not meet the expectations of a high percentage of drivers can significantly degrade the safety of the facility.

Design speed should be logical with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of highway. The design speed chosen should be consistent with the speed a driver is likely to expect on the facility being driven. A lower design speed should not be selected where physical conditions are such that drivers are likely to travel at high speeds. Drivers adjust their speed not to the importance of the facility, but to their perception of the physical limitations and traffic flow. See Section 2.3.3 and Section 3.2.1 for more discussion

on the selection of an appropriate design speed.

Proposing an alternative design speed for a project or a segment of the project is not recommended for inclusion in the design exception process. Instead, individual geometric features should be evaluated within the selected design speed and addressed as exceptions if necessary. As discussed previously, most design features have an acceptable range of values that will meet driver expectation and provide acceptable driver safety.

Using the posted speed as the design speed is also not recommended. The design speed should be a minimum of 5-mph [10 km/h] over above the posted speed. Design speed usually approximates the 85th-percentile speed value as determined by observing a sizable sample of vehicles, but is not the highest speed that might be used by drivers unless reasonably enforced.

10.1.5.3 TRAFFIC CONSIDERATIONS

The importance placed on the operational and safety characteristics of a project is in direct proportion to the traffic volume and composition. Experience shows that the greatest contributor to the risk of an accident is traffic volume. Thus the volume of traffic may be a primary factor in decisions on design exceptions.

The designer uses two traffic volumes, current and projected in setting design controls. Depending upon the type of facility, perceived need, and existing traffic, a five or ten-year projection may be the control. New construction and reconstruction projects where increased capacity is the goal are designed to meet the needs of a projected 20-year traffic volume. Traffic forecasts are based on technical analysis, policy consensus (the State's vision of the future transportation network), anticipated type and intensity of land use, future economic activity, and other factors. Large variances with one or many of the factors used in the modeling process to project future traffic can and have occurred in the past. Some projects have exceeded their traffic projections in a very short time span and become functionally obsolete, some have never reached their projected traffic volumes and others have taken longer then the projected year to reach assigned design volumes. Context sensitive design recognizes the inherent limitations and uncertainties associated with long term traffic forecasting.

For setting design features, the forecasted daily traffic volume is further refined to develop an appropriate design hourly volume, DHV. The DHV initially selected is the 30th highest hourly volume of the year. Exceptions may be made on roads with high seasonal traffic fluctuations or other conditions, where a different volume may need to be used. Available projection techniques lead to estimated volumes. Selection of a design hour volume should reflect other constraints and controls reflecting the project's purpose and need. High DHV’s usually result in wider lanes, more lanes, and greater costs. All of these tend to increase disruption to the community and conflict with other important issues.

Another design control reached through analysis of current and forecasted traffic is the predominant type of driver and vehicle expected to use the facility. With the perception that safety will be greatly enhanced, design controls are conservatively established regarding the capabilities of the driver to react physically and mentally to conditions that may be encountered. The result of this analysis is the selection of a design vehicle. Projects involve a variety of adjacent land uses, types of intersections and alternative modes of transportation and may have several design vehicles. Each design vehicle has physical and operational characteristics that affect the design controls. These include acceleration and deceleration capabilities, ability to climb steep grades, sweep-path dimensions of turning vehicles, and height of the driver's

eye. There is a range of design vehicles and associated characteristics. There is no mandated design vehicle. The choice(s) should be made with the knowledge of the tradeoffs involved and input from the community and public while being cost and space effective.

10.1.5.4 LEVEL OF SERVICE

How the user perceives a design to meet the traffic operational conditions for the selected design hourly volume is defined as the level of service. Level of service can be quantified in terms of running speed, travel time, freedom of movement, traffic interruptions, comfort and convenience. Different types of highways have different attainable or expected levels of service. The various elements that make up each type of highway depend upon the selected level of service. Rural highways are expected to operate at higher levels of service and facilities in urban areas at lower levels with acceptable congestion. The level of service selection in context sensitive design recognizes the need to tailor the level of service to other design controls and constraints, within the context of the project's purpose and need.

10.1.6 HIGHWAY GEOMETRIC ELEMENTS—DESIGN AND SAFETY CONSIDERATIONS

Geometric design controls the horizontal, vertical, and cross sectional elements throughout the project including intersections. Selection of geometric criteria should recognize the characteristics of the potential users and the physical and operational characteristics of their vehicles. Using absolute design criteria values can lead to conflicts with community values and local constraints. In developing a context sensitive design, it may be necessary to design using the flexibility that lies within the standards while maintaining acceptable and predictable safety and operational characteristics.

10.1.6.1 HORIZONTAL ALIGNMENT

Horizontal alignment is comprised of tangents and circular curves used to adjust the alignment laterally to meet land use and terrain constraints. Horizontal curvature is a controlling feature of a design. Curve selection is based on the project's design speed and resulting superelevation rate. The transition sections between tangents and circular curves are also important elements in establishing horizontal alignment. AASHTO criteria in the Green Book are based on the amount of friction needed for the vehicle to track properly and the comfort perceived by the driver at the design speed under poor pavement conditions. Current design tables for friction are based more on driver tolerance than what is actually necessary to prevent loss of control.

Knowing that the assumptions in establishing the relationship of design speed to curvature are driver comfort and poor pavement condition will allow the designer to determine the risks in accepting a non-standard design. Reliable data on actual speeds, accident types and probable causes, and roadside conditions along the curve area will help the designer in this decision-making process. Studies show that safety is related to curvature and length of curves on higher speed facilities. On lower speed streets, there is little indication that there are safety affects associated with marginal curvature. However, where there is a predominance of higher profile vehicles with higher centers of gravity there is an increased need to meet or exceed the criteria.

In considering the safety risk of a design exception for horizontal curvature, AASHTO suggests evaluating the following factors:

- Risk is proportional to traffic volumes increase;
- Risk may generally be acceptable if the effective or nominal speed of the proposed curve is within 5 to 10 mph

[10 to 15 km/h] of the appropriate design speed of the curve;

- For roads with significant truck traffic, the risk increases due to a truck's tendency to overturn at lower speeds than passenger cars and the acceptable difference between design speed and operating speed is less;
- Risk varies proportionally with the length of curve;
- Risk of accidents within a curve is a function of the curve geometry, cross section, sight distance, and/or presence of intersections and driveways within the curve.

In providing a context sensitive design, the use of a range of curvature may be needed to mitigate various issues that arise on a project. Although it is desirable to use one minimum value for a design radius, it may not be practical. Substandard alignment decisions will require written documentation. Inclusion of other design elements to mitigate any adverse safety impacts should be considered. These elements could include additional signing, increased delineation, improved superelevation, lane widening, improving or widening the shoulder through the curve, improving roadside safety, increased clear zone, relocating or closing intersections or driveways within the curve, or providing higher surface friction pavements.

10.1.6.2 VERTICAL ALIGNMENT

The vertical alignment design elements are grade and sight distance. Grades are related to terrain and functional classification. Stopping sight distance governs vertical curvature.

Long or continuous steep grades can contribute to operational problems and safety risks. Flat grades can cause drainage problems leading to increased maintenance costs and wet-weather safety problems. Combining steep grades with sharp horizontal curvature creates a high-risk safety condition.

10.1.6.3 SIGHT DISTANCE

Sight distance is a function of the roadway alignment and cross section provided. Sight distance is the distance ahead for which a line of sight is provided for the driver under certain design conditions and assumptions. There are four types of sight distance—stopping sight distance, intersection sight distance, passing sight distance and decision sight distance. Stopping sight distance and intersection sight distance usually control the design.

Passing sight distance needs to be considered on two lane facilities as it affects operational quality and capacity but does not necessarily directly impact safety. Decision sight distance may be desirable in situations where there is a need to provide the driver with more time to make movement decisions.

Stopping sight distance criteria are based on operational models that reflect a driver's ability to see an object at given height above the roadway and react appropriately to avoid a collision. Sight distance can be limited by the vertical alignment of the road but also can be affected by a combination of horizontal alignment and sight obstructions beyond the edge of pavement. The model assumptions include size and placement of the object, the height of the driver's eye, driver perception time and braking reaction. The model applies to all levels of traffic volumes and all highway types equally. Since there are several assumptions that are affected by the evolution of the vehicle fleet, the age of the driving population and research on driver behavior, the range of acceptable stopping sight distance values is intended to be conservative. The model is not based on directly measurable safety values, the probability of a driver encountering the assumed conditions, or the severity of such an encounter. In other

words, the design values for stopping sight distance and vertical curvature are unrelated to direct measures of safety, and generally provide substantial margin of safety against the actual risk of a crash. Sag vertical curve values are based on night operations where the controlling feature is the headlight beam on the pavement ahead.

The designer can assess the risk of a location with limited stopping sight distance using the following guidance:

- The risk of a sight distance restriction is related to the traffic volume exposed to it.
- The risk of a sight restriction is greater where other features such as intersections, narrow bridges, high volume driveways, or sharp curvature occur within the sight restriction.
- Where no high-risk features exist within the sight restriction, nominal deficiencies may not create undue risk of increased accidents.
- Horizontal restrictions such as buildings, signs, tree lines, etc. affect all types of vehicles equally.
- When possible, designers should use shorter sag vertical curves in favor of providing the longest crest vertical curves possible.

Lower ranges for stopping sight distance may be applicable for reconstruction projects where speeds and volumes are observable, accident records are available and the constraints of providing the desirable value may be cost prohibitive and adversely impact adjacent land uses.

10.1.6.4 CROSS SECTION ELEMENTS

Cross section elements of a facility are the most visible and have the greatest physical affect on constructing or reconstructing a roadway. These include the lane and shoulder widths, median treatment, border areas, side slopes and ditch sections. In selecting these elements one must consider traffic volume, traffic mix, transportation modes, functional classification, available right of way and adjacent land use. Cross sectional elements become increasingly important as the severity of the alignment increases. Traffic operation and safety are very closely related to lane widths, shoulder availability, shoulder width and shoulder surface type.

The use of wider lane widths should be considered on higher speed facilities or those with high truck volumes. Lane widths are also influenced by horizontal curvature, as vehicles tend to move off-track and require room to avoid on-coming vehicles. In urban areas, narrower lanes may be considered as speeds are lower and the driver expects to encounter normal congestion and slower travel speeds.

Shoulders are a very important element of the roadway cross section. The shoulder serves as a part of the clear zone, improves capacity, accommodates drainage, provides an area for disabled vehicles, enables collision avoidance maneuvers, provides structural support for the traveled way and accommodates bicyclists and pedestrians. Where a full shoulder can not be provided, the designer should attempt to provide as wide a shoulder as possible that meets functional requirements.

The area adjacent to the roadway plays a major role in a facility's safety particularly with respect to run-off the road accidents. In applying context sensitive design philosophies, treatment of this area is affected by the established clear zone, costs, available right-of-way and probable impact on safety. Depending upon the roadside design provided, encroachment may not be a problem. When designing a project, overall safety benefits may be achieved by providing spot roadside improvements that may not necessarily be applicable throughout the length of a project.

Ideally a facility's roadside area should be a wide, firm, flat, hazard-free area that will permit the errant driver to recover and safely return to the highway. Early in the design process the designer normally establishes an area outside the travelway (edge of running lane) termed "clear zone." The effort and cost required in providing this area varies significantly from location to location. Deciding how much clear area to provide is proportional to the design speed, the projected traffic volume, historical accident data and the cost.

The most serious and obvious hazards are fixed objects that when impacted produce a sudden or instantaneous deceleration. These would include man-made objects, such as utility poles, overhead sign structures, buildings, retaining walls, large drainage inlets and outlets, exposed headwalls, and utility and traffic control boxes. Some objects only present a serious hazard at high-speeds while others are potentially serious hazards at any speed. The latter may require a more durable and effective barrier system.

The most hazardous natural fixed objects are trees. In context sensitive design, rather than clear the entire right-of-way, a designer needs to identify those trees most likely to present a significant hazard to the driver. Tree removal is a sensitive environmental and community issue. This perception must be considered when identifying trees to be removed. Where possible an analysis of accident history and visual evidence may indicate if there is a tree safety problem. If there are numerous trees, removing selective trees may not improve safety. Isolated trees lying close to the roadway present the most probable safety risk and should be removed or protected. Community interest, speed, volumes, and past operational history would all affect this decision.

Natural and created topography does play an important role in roadside design. Created topographical features enter into the design of a safe roadside throughout the state. These include steep backslopes in cut areas, steep sideslopes in embankment areas and unsafe median and ditch sections. The goal is to minimize the need and use of unsafe road sections when designing these elements.

Most projects involve some type of improvement to an existing facility. Fixed objects encountered can include utility poles, sign and lighting supports, mailboxes, fences, gates, commercial signs, unsafe rock outcroppings, bridge abutments and piers. In creating a safe roadside environment, the designer has five options to address fixed objects: (1) removal, (2) relocation, (3) modification, (4) shielding and (5) delineation. Although it may not be practical or cost-effective to treat all obstructions, studying and mitigating (where possible), each obstruction should be viewed as an opportunity to improve roadside safety.

The concept of a wide hazard-free clear zone is usually not practical in urban conditions where many roadside features come into play, such as pedestrian areas, utilities, streetlights, signs, ornamental structures, benches, on-street parking, and limited right-of-way.

Roadside safety should not be compromised in context sensitive design. To reach this goal the designer should:

- Avoid setting an artificially high or low design speed;
- Apply a consistent roadside treatment;
- Not be arbitrary in selecting a clear zone;
- Encourage the removal or relocation of fixed obstacles;
- Use safe landscaping and grading treatments; and
- Provide safe sight distances at intersections and all planted areas.

On multi-lane highways the median treatment is an important part of cross

section design. The primary purpose of a median is to separate traffic, thus reducing the occurrence of head-on collisions. However, there are other benefits associated with medians that improve overall safety, traffic flow (vehicular and pedestrian) and aesthetics. The median may be raised, depressed or flat, and includes paved or unpaved shoulders, swales, or ditches. Medians also offer the opportunity for providing landscaping. Median width selection is a function of the available right-of-way, the border area needs and providing area to allow safe maneuvers and room for traffic devices. Median width meeting one or more of these needs has a large range of values. Wide medians are not always attainable but even narrow medians can provide positive benefits.

10.1.6.5 INTERSECTIONS

Most projects will involve the treatment of intersecting roadways. Intersection design is important to a facility's efficiency, safety, speed, capacity, and user cost. Safe intersection design includes the following:

- Provision for adequate sight distance;
- Accommodation of appropriate traffic control devices;
- Provision of safe and efficient handling of turning traffic (left turns have the most conflicts);
- Avoidance of non-standard or confusing geometry, signing or other out of the ordinary treatments prior to or within the intersection; and
- Optimization of the capacity of the intersection.

The design features of an intersection are controlled by selection of the design vehicle, the larger the design vehicle the larger the intersection. Since space is limited, it may be better to accommodate the design vehicle that is anticipated to use the intersection most frequently. The available turning path templates were developed using relatively conservative dimensions. Computer programs are available to help the designer test the tracking of a design vehicle. When constraints do not permit meeting the full geometry and lane widths of the design vehicle, consideration should be given to moving roadside objects, such as signal poles, highway lighting, signs, etc. farther from the edge of pavement. When not providing the full standards, it can be expected that there will be occasional encroachment outside the designated lane area. These areas should be evaluated for type of curb, particularly if there are fixed objects. In addition, aesthetic and traffic calming improvements should not be placed in anticipation of encroachment. If the intersection can not accommodate the desired turning lane arrangements, different traffic control methods may be used including turn prohibitions, special signal phasing, advanced signing or other measures.

10.1.7 MAINTAINABILITY

All designs must be continually evaluated for maintenance needs. The frequency, ease, cost, accessibility and safety of maintaining selected design elements are essential to measuring a successful project. Much of the design effort, particularly for context sensitive projects that use innovative and unique concepts or construction materials, can be negated if they can not be properly maintained.

10.2 TRAFFIC CALMING

In communities where speeding and/or cut-through traffic has been identified as a problem, it may be appropriate to consider the feasibility of one or more traffic calming measures. Such measures are intended to encourage adherence to posted speeds and/or discourage inappropriate routing of non-residential or non-local traffic through communities.

The DelDOT *Traffic Calming Design Manual* (TCDM) and the joint FHWA-ITE *Traffic Calming-1999* publication provide general guidance regarding the appropriate use, design, signing and marking the traffic calming measures approved for use in Delaware. It also describes the steps necessary to nominate, select, develop and implement traffic calming projects. In the event it is necessary to modify or remove an existing traffic calming measure, the TCDM provides guidance in that matter as well.

10.3 TRAFFIC BARRIERS

The purpose of traffic barriers is to reduce fatalities and injuries by preventing a vehicle from leaving the traveled way and striking a fixed object or terrain feature. However, it is recognized that a protective barrier is, in itself, a roadside obstacle and may not contribute to safety. Therefore, under most situations a barrier is warranted only when the consequences of leaving the roadway are likely to be more severe than a collision with the traffic barrier.

Other factors not to be overlooked by the designer in determining the need for barriers include an analysis of a project's accident history, types of accidents, adjacent land uses, pedestrian and bicyclist use, and adverse geometrics such as sharp curvature combined with poor sight distance.

Barrier performance is based on its ability to contain and/or redirect the errant vehicle. The behavior of a vehicle during impact is very complex. Barrier designs are developed through full-scale crash tests under controlled standardized conditions. Sections 5.1 and 5.2 of the *Roadside Design Guide* provide the details on the testing programs.

Because of the on-going testing programs, the available types of barriers and installation details for barriers are dynamic. The Department's Standard Construction Details, the Standard Specifications, and Special Provisions are continually modified to reflect changes recommended by AASHTO, FHWA, the Transportation Research Board and other agencies that oversee the barrier testing programs. The testing results are published and recommendations made to transportation agencies for selecting approved barrier types based upon the anticipated service needs.

The three most commonly used barriers discussed in this section are:

- Longitudinal barriers,
- Median barriers, and
- Impact attenuators.

10.3.1 DESIGN OPTIONS

A designer's goal is to develop an acceptable design that does not warrant barriers. The designer has several options after an initial determination has been made that a traffic barrier is warranted. Design options for the treatment of the various conditions encountered within a project's limits should be evaluated in the following order:

- Examine the proposed and existing roadside features to determine the feasibility of eliminating the need for a barrier;
- Remove the obstacle or redesign it so it can be safely traversed, that is, design traversable slopes, extend pipes beyond the clear zone, install safety end sections, etc.;
- Relocate the obstacle to a point where it is less likely to be struck;
- Reduce impact severity by using an appropriate breakaway device;
- Redirect a vehicle by shielding the obstacle with a longitudinal traffic barrier and/or crash cushion; and
- Delineate the obstacle if the above alternatives are not appropriate.

10.3.2 GUIDELINES

The criteria and procedures for barrier need and design are outlined in the AASHTO *Roadside Design Guide*. Using the charts and graphs found in the guide in determining the need for barrier is relatively simple. However, this method does not consider the probability of an accident or the costs of either shielding the condition or leaving it as-is. A method has been developed that establishes warrants based on a cost-benefit analysis that considers factors such as design speed and traffic volume. The guide includes a computer program for evaluating the costs and benefits of design options for safe treatment of roadside hazards. Performing this analysis may be a useful tool for determining the preferred safety treatment.

The following are important elements to consider in determining the need for and selection of types of barriers:

- Performance capability,
- Deflection,
- Site conditions,
- Compatibility with other design features,
- Cost,
- Maintenance (routine and collision, storage of spare parts, and simplicity of repair),
- Aesthetics, and
- Past performance.

10.3.3 LONGITUDINAL BARRIERS

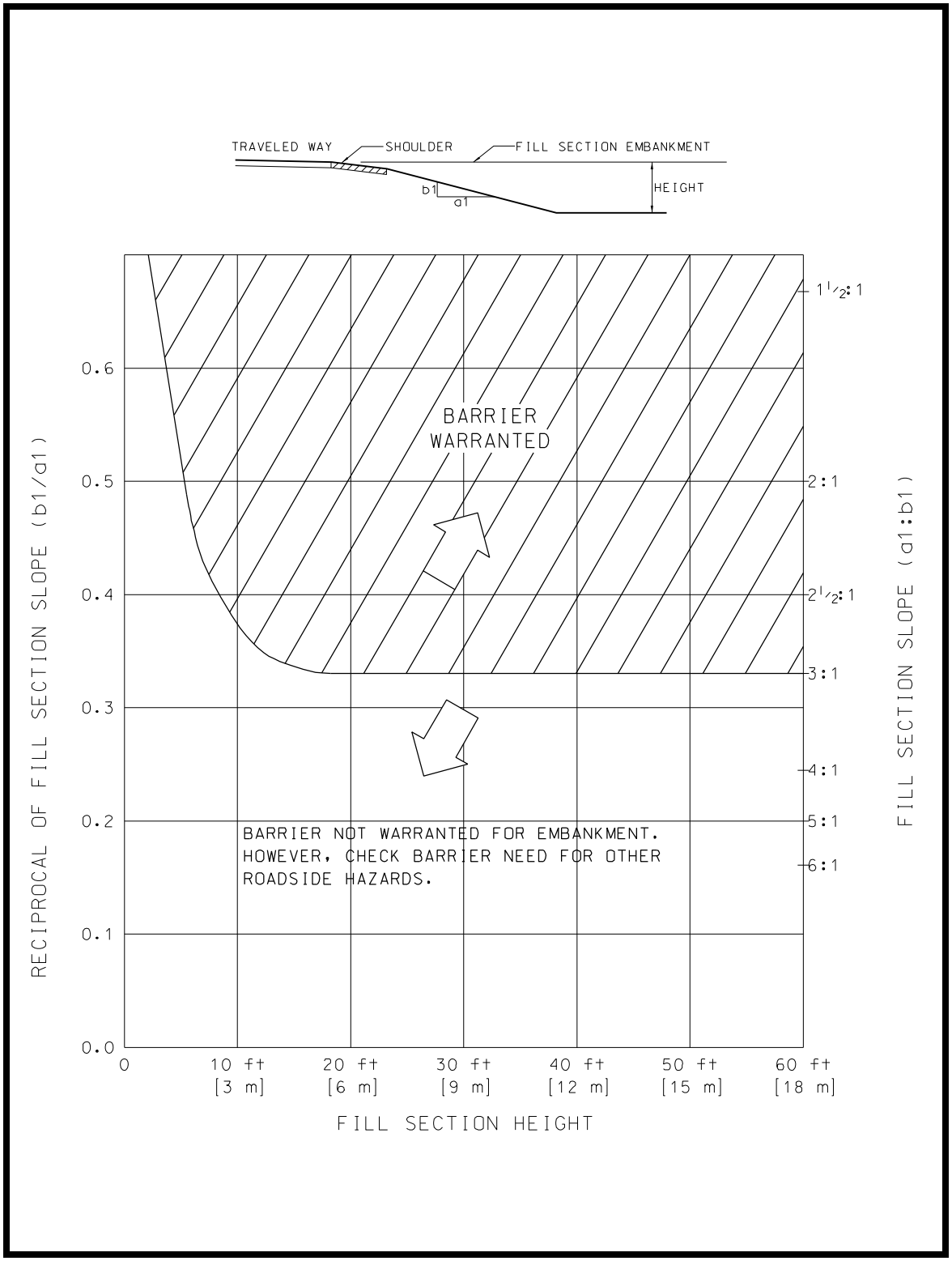
W-beam steel guardrail and concrete safety shape barriers are most commonly used for longitudinal shielding. Geometric criteria for steel beam guardrail, along with illustrative typical installations, are in the Department's Standard Construction Details. The need for barriers is directly related to the selected cross sectional elements as discussed in Chapter Four. This section is directed principally to guidelines and general considerations for designing guardrail installations.

Hazards that may warrant shielding by a roadside barrier can be placed in one of two basic categories, embankments or roadside obstacles. Protecting pedestrians and cyclists from vehicular traffic with a barrier may also be warranted.

ADT, embankment heights and slope rates are the primary factors in determining barrier needs for fill sections. Figure 10-1 shows guardrail warrants for various combinations of embankment height and slope. A barrier installation is warranted where the point of intersection falls above the curve.

Side slopes on an embankment should provide a reasonable opportunity for recovery of an out-of-control vehicle. Flat slopes characterize a traversable embankment and generous rounding of the slope breaks. In instances where construction of flat slopes is not feasible or practical, installing a barrier may be necessary. Where slopes are 3:1 or steeper, designers should first consider the alternative of the adjusting the embankment slopes, thus avoiding the use of barrier.

Figure 10-1
Guardrail Warrants for Embankments



Roadside obstacles are classified as "non-traversable hazards" and "fixed objects". If it is not possible to remove or relocate a hazard, then guardrail may be necessary. However, guardrail should be installed only if it is clear that a barrier offers the least hazard potential and is the most cost-effective in terms of safety benefits.

In the case of non-traversable hazards, a general criterion is that barrier protection should be provided for streams or bodies of water that may constitute a hazard and areas of rough rock cuts.

Warrants for barriers to shield roadside obstacles involve different considerations. A principal consideration is the required clear zone width. Clear zone selection is discussed in Chapter Four. Barriers are normally not required to shield a fixed object outside the specified clear zone. Roadside obstacles in the form of fixed objects include items such as trees, bridge piers and abutments, culvert headwalls, sign posts, light posts, and non-traversable drainage structures. In the case of drainage structures, the designer should consider extending pipes or using traversable end treatments to eliminate the hazard. Guardrail shielding normally is warranted if obstacles or fixed objects are within the limits of the lateral clear zone. Accepted designs of "breakaway types" of poles and posts are not considered fixed objects.

The need for guardrail at a bridge approach is based on the clear zone requirements for fixed hazards. For twin bridges, the length of approach rail on the median side of each bridge should be long enough to prevent an errant vehicle from impacting the bridge rail end of the other bridge. If it is within, or close to, the design clear zone, the guardrail should be long enough to protect the area between bridges at the edge of the clear zone. Consideration should be given to including a transverse berm between the endwalls of the two bridges.

Special treatment is required where side entrances interrupt the installation of guardrail. Additional flares and end treatments may be required, or it may be more economical to eliminate the need for guardrail.

An area of concern is what has been termed the "innocent bystander" problem. In most of these cases, the conventional criteria presented above cannot be used to establish barrier needs. For example, a major roadway or street may adjoin a schoolyard, but the boundaries are outside the clear zone. Since it is outside the clear zone, a barrier would not normally be required. However, if there is a reasonable probability of an errant vehicle encroaching on the schoolyard, a barrier could be considered and installed.

Protection of pedestrians and cyclists is another area of concern. As in the case of bystander warrants, there are no objective criteria for pedestrian and cyclist barrier warrants. On low-speed streets, a vertical curb may be sufficient to delineate pedestrians and cyclists from vehicular traffic. However, at speeds over 25 mph [40 km/h], a vehicle may mount the curb for relatively flat approach angles. Particularly where sidewalks or bicycle paths are adjacent to the traveled way of high-speed facilities, some additional provision may be required for the safety of pedestrians and cyclists.

10.3.4 BARRIER PLACEMENT

After the decision is made that a barrier is warranted, there are several factors that need to be considered for its placement. These are:

- Lateral offset from the edge-of-traveled way,
- Terrain effects,
- Flare rate, and
- Length of need.

10.3.4.1 LATERAL OFFSET

A roadside barrier is normally placed as far from the traveled way as possible, while still maintaining the operating characteristics of the selected type of barrier. The greater the distance the better chance for the driver to recover control of the vehicle. In addition, some barrier installations may obscure a driver's sight distance if placed too close to the traveled way.

Placing the barrier at a uniform offset distance will not only be more aesthetically pleasing but provides the driver with a feeling of security and comfort when approaching a series of protected obstruction areas. The distance from the edge-of-traveled-way, beyond which a roadside object will not be perceived as an obstacle and result in the driver reducing speed or directing the vehicle away from the barrier, is called the shy line. This theoretical distance is different based on design speed and is shown in Table 5.5 in the *Roadside Design Guide*. For long continuous runs of barrier and barrier placed beyond the shoulder, the shy line offset criterion usually does not control barrier placement.

Another consideration in the lateral placement of barrier is the expected deflection of the system selected. The distance from the barrier to a rigid obstruction should not be greater than the dynamic deflection of the system based upon data from actual impact tests under controlled conditions, i.e. vehicle weight, speed and impact angle.

In embankment areas that must be protected, it is important that the width of embankment be sufficient to adequately support the posts to ensure proper operational characteristics of the barrier, see the Standard Construction Details.

There may be considerable deflection in barriers when impacted by a vehicle. Figure 10-2 shows the dynamic deflection of W-beam and concrete safety shape. If a roadside obstacle is too close to the back face of the rail or post, there may be danger that the rail will deflect all the way to the obstacle. Under these conditions, designing the post spacing closer than normal will reduce the potential deflection of the guardrail. If the obstacle is more than 3 ft [0.9 m] behind the back of post, a post spacing of 6 ft 3 in [1905 mm] should be used.

For obstacles located from 2 to 3 feet [0.6 to 0.9 m] behind the back of post, a post spacing of 3-ft 1-1/2 in [952 mm] should be used. If the obstacle is less than 2 ft [0.6 m], a rigid concrete barrier could be used. These deflection guidelines are based on having a proper end anchorage and posts installed in stable soil.

10.3.4.2 TERRIAN EFFECTS

A roadway's cross section is important element in a vehicle's performance when approaching or impacting a barrier. Barrier systems perform best when vehicles have all wheels on the surface and its suspension is in a normal position at the point of impact. The two common features of concern are curb and the approach slope. These features may cause a vehicle to vault over a barrier or strike the barrier too high or too low.

Vehicles striking curbs can change trajectory depending upon the size of vehicle, suspension characteristics of the vehicle, its impact speed and angle, and the height and shape of the curb. Impact testing has shown that the use of any guardrail/curb combination where high-speed, high-angle impacts is not an acceptable practice. If there is not other alternative, than the curb should be limited to 4 in. [100 mm] and the guardrail stiffened.

Vehicles traversing slopes steeper than 10:1, depending upon their impact angle and speed, may go over or impact the selected barrier too low. The *Roadside Design Guide* in section 5.6.22 and Figure 5.22 details the effect of slope rate on vehicle and barrier

performance. The conclusion of the discussion is that roadside barriers perform most effectively when they are installed on slopes of 10:1 or flatter. A slope of 6:1 may be a problem and Figure 5.23 in the *Roadside Design Guide* gives a recommended barrier location when using 6:1 slopes.

Figure 10-2
Dynamic Barrier Deflection

Barrier Type	Post Spacing	Dynamic Deflection (From back of post)
Blocked-out W-Beam Steel Post	6 feet 3 inches [1905 mm]	3 feet [0.9 m]
Blocked-out W-Beam Steel Post	3 feet 1-1/2 inches [952 mm]	2 feet [0.6 m]
Concrete Safety Shape	Continuous	0 feet [0 m]

10.3.4.3 FLARE RATE

A barrier may be introduced by offsetting the beginning of the installation farther away from the traveled way than the normal offset. This allows the terminal section to be located farther away, minimizes the driver's reaction to having an obstacle close to the road, transitions the barrier to an obstacle nearer the roadway or reduces the total length of barrier needed.

There are disadvantages to flaring barriers. The greater the flare rate, the higher angle at which a barrier can be hit increasing the severity of crashes. A flared installation may increase the possibly of a vehicle being redirected back into or across the roadway.

The suggested flare rates for barrier design are shown in Table 5.7 of the *Roadside Design Guide.*

10.3.4.4 LENGTH OF NEED

The length of need *(X)* depends on the runout length *(L_R)*, the lateral extent of the area of concern (hazard) *(L_A)*, the flare rate for the tapered section *(b/a)* and the distance from the edge of traveled way to the face of barrier *(L_2)*. Guardrail is normally placed 1-foot (0.3 m), or more, beyond the shoulder. The runout length *(L_R)* is the theoretical distance needed from the edge of traveled way to the hazard measured along the edge of the pavement. The lateral extent (L_A) of the hazard is measured perpendicularly from the edge of the traveled way to the far side of the hazard or to the clear zone. The variables are shown in Figure 10-3**.** The tangent length of barrier immediately upstream from the area of concern, L_1, is a variable length selected by the designer. Runout lengths *(L_R)* for the various design speeds and traffic volumes are shown in Table 5.8 of the *Roadside Design Guide.*

The total length of barrier, without the end treatment, can be calculated with the following equation:

$$X = \frac{L_A + (b/a)(L_1) - L_2}{(b/a) + (L_A / L_R)}$$

Where,

L_A = the perpendicular distance from the edge of the traveled way to the far side of the hazard, if the hazard is a fixed object or to the outside edge of the clear zone.

L_2 = the distance from the edge of traveled way to the face of the barrier at the location where the rail is parallel to the roadway.

b/a = the flare rate.

L_1 = the length of the tangent section ahead of the hazard.

L_3 = edge of traveled way to front of hazard. L_3 equals, as a minimum, the shoulder width plus the 2 ft [0.6 m] offset for the barrier plus the allowance for dynamic deflection.

Note that for a parallel installation, i.e., no flare rate, the equation reduces to:

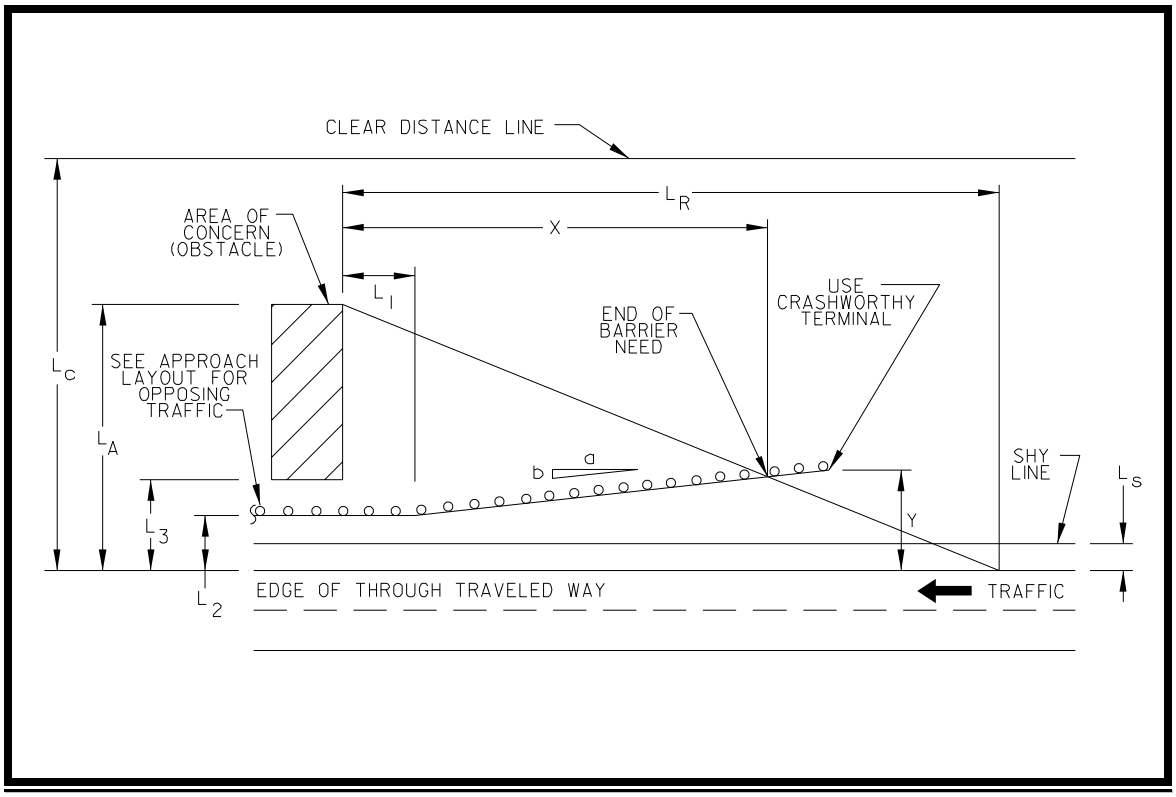
$$X = \frac{L_A - L_2}{L_A / L_R}$$

The lateral offset, Y, from the edge of the traveled way to the beginning of the length of need, can be calculated using the following equation:

$$Y = L_A - \frac{L_A}{L_R}(X)$$

The value of L_1, the length of the tangent section ahead of the hazard, is a variable length selected by the designer. If a semi-rigid railing is connected to a rigid barrier, the tangent length should be at least as long as the transition section. This reduces the possibility of pocketing at the transition and to increase the likelihood of smooth redirection if the guardrail is struck immediately adjacent to the rigid barrier. To determine the length of the transition section, see the Department's Standard Construction Details.

Figure 10-3
Approach Barrier Layout Variables



The final variable for determining the required length of guardrail is the flare rate. Suggested flare rates for various design speeds are shown in Table 5.7 of the *Roadside Design Guide.*

Slopes must be 10:1 or flatter in front of the barrier. It is desirable to place a barrier as far from the edge of the traveled way as possible. Because this may involve a combination of different slope rates, the designer should refer to the *Roadside Design Guide* for proper section and slope treatment.

The shy line offset (L_s) is the distance beyond which a roadside object will be perceived as non-hazardous and does not result in motorists reducing speed or changing vehicle positions on the roadway. This distance varies for different design speeds as indicated in Table 5.5 of the *Roadside Design Guide.* If possible, a roadside barrier should be placed beyond (outside) the shy line offset, particularly for relatively short or isolated installations. For long, continuous runs of barrier this offset distance is not as critical, especially if the barrier is first introduced beyond the shy line offset and gradually transitioned closer to the roadway. Note that flatter flare rates are suggested when the barrier must be placed inside the shy line.

The lengths of barriers can be determined by plotting the barrier layout directly on the plan sheets. By selecting an appropriate runout length and the lateral distance to be shielded, the designer can develop a guardrail installation that satisfies all placement criteria.

This method is most appropriate for determining the length of barrier needed to shield embankments or hazards on curved sections of roadways. It should be noted that the portion of the acceptable end treatment that has full effectiveness can be included in the length of need, (*X)*. The *Roadside Design Guide* provides several figures and examples of procedures for determining the length of need for typical situations.

10.3.4.5 APPROACH BARRIERS FOR OPPOSING TRAFFIC

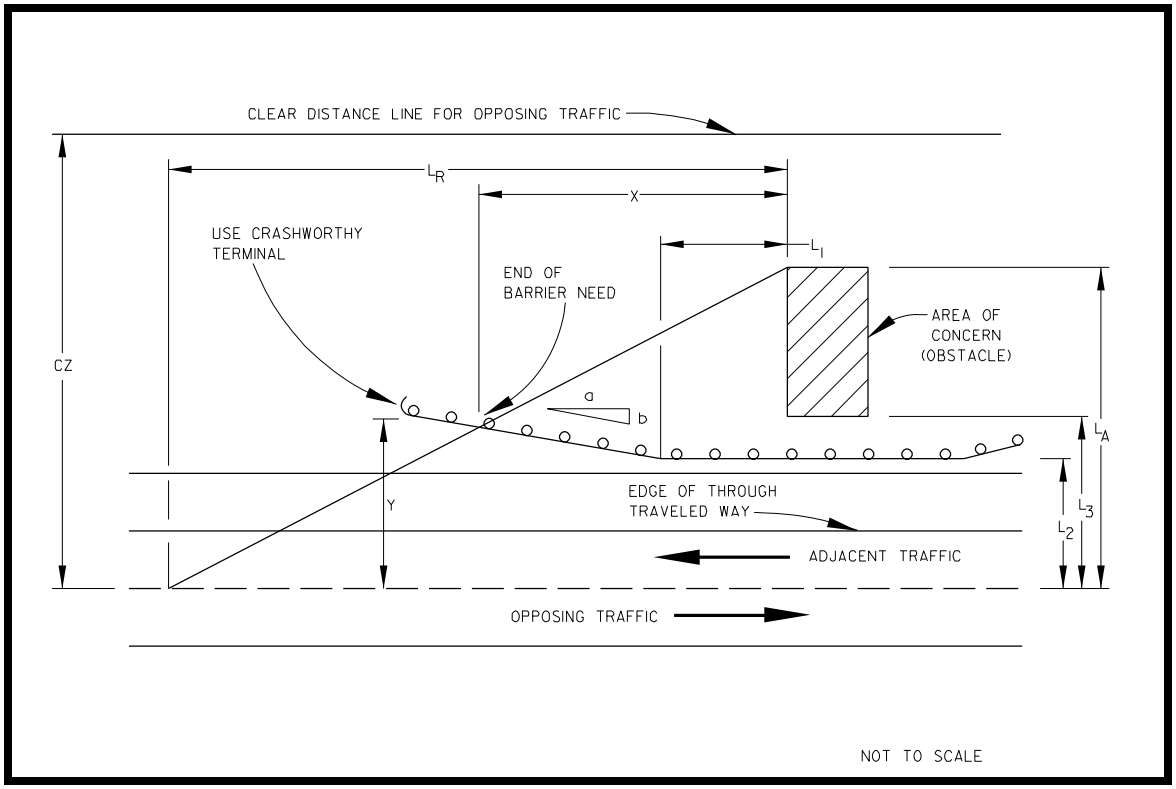
On roadways with two-way traffic, the length of need must be checked from both directions. All lateral dimensions for opposing traffic would then be measured from the centerline of the roadway. Figure 10-4 illustrates the layout variables for an approach barrier for opposing traffic. The length of need and the end of the barrier are determined in the same manner as previously described.

10.3.4.6 ROADSIDE SLOPES FOR APPROACH BARRIERS

The embankment where the flared section is to be placed should have a slope of 10:1 or flatter to prevent errant vehicles from striking the barrier either too high or too low for the barrier to be effective. If the slope is steeper than 10:1, the slope should be flattened so that the embankment criterion is not violated. See Figure 5.28 of the *Roadside Design Guide* for the suggested roadside slopes for approach barriers.

Details of typical guardrail installations at bridge ends are shown in the Department's Standard Construction Details. These sheets also show various applications of guardrail installations.

Figure 10-4
Approach Barrier Layout for Opposing Traffic



10.3.5 MEDIAN BARRIERS

The basic function of median barriers is to prevent out-of-control vehicles from crossing the median and entering opposing lanes. Effective median barriers should be installed on all high-volume, high-speed divided highways with medians where engineering studies establish a need. Figure 10-5 suggests warrants for median barriers on high-speed divided highways that have relatively flat, traversable medians. These criteria are based on a limited analysis of median crossover accidents and research studies and are suggested for use in the absence of site-specific data.

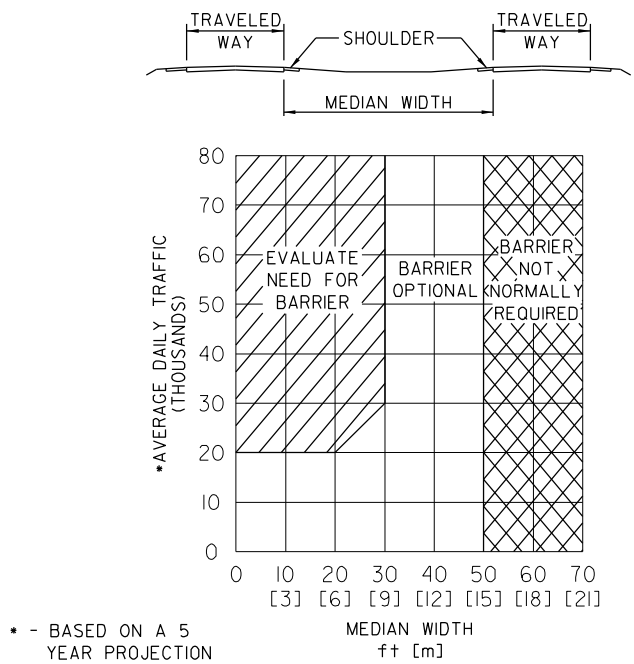
On divided highways, obstacles in the median should be treated the same as obstacles outside the roadway. If the slope is traversable and there are no other obstacles, guardrails on the exit side can be terminated opposite the hazard with a standard guardrail terminal. It need not be flared or have crashworthy end treatment unless it is in, or close to, the clear zone for traffic in the opposing lanes.

Barriers are typically considered for combinations of average daily traffic (ADT) and median widths that fall within the darkened area. In the "Barrier Optional" and "Barrier Not Normally Considered" area of Figure 10-5, a barrier is warranted only if there has been a history of cross-median accidents. When designing for the installation of a median barrier the median must be at least 10 ft [3 m] wide to provide adequate horizontal clearance between the barrier and the edges of traffic lanes.

The most common types of median barriers used in Delaware are the blocked-out W-beam guardrail and the concrete safety barrier. Concrete safety barrier is preferred in narrow medians where regular

maintenance is difficult, or where deflection of the barrier would affect opposing traffic. Temporary concrete median barriers are used for traffic control in construction areas. They are placed to separate opposing lanes of traffic on detours and to separate work areas from traffic.

Figure 10-5
Median Barrier Warrants for High-Speed Divided Highways



10.3.6 IMPACT ATTENUATORS

Impact attenuators or crash cushions are designed to prevent vehicles from impacting a fixed object by redirecting a vehicle if impacted from the side and stopping the vehicle at a rate of deceleration that is tolerable to the vehicle occupants when impacting head-on. The two most commonly used concepts used are absorption of the kinetic energy or transfer of momentum.

When designing new facilities the need for these devices should be limited to gore areas, principally on elevated structures at ramp exits.. For other projects they may be the most logical choice to shield the ends of median barriers or longitudinal barriers without crashworthy end terminals. Impact attenuators must be properly installed to provide the desired performance. The approach grading must be flat and the attenuator placed level with the approach ground.

The type of the impact attenuator depends on the expected impact speed and width of the fixed obstacle. Manufacturers provide charts that can be used to determine the appropriate device for the specific location.

The selection of a particular impact attenuator depends on several factors including cost, site compatibility, periodic maintenance needs, extent and ease of maintenance after impact, anticipated performance characteristics, and structural effect of impacts. For additional information and guidance, including the design of the size of the device for the proposed impact speed, refer to Chapter Eight in AASHTO's *Roadside Design Guide* and the manufacturer's literature. It should be emphasized that in frequent impact locations the impact attenuator should be delineated to reduce the number of hits. At these locations a cost/benefit analysis may justify the additional expenditure for the installation of a self restoring attenuator.

10.4 CURBS

Curbs serve a variety of positive functions including drainage control, roadside delineation, reduction in right-of-way needs, aesthetics, reduction in maintenance operations, protection of pedestrian traffic, and control of existing and future roadside development. Even with all of these positive attributes, a curb remains a longitudinal fixed object that can cause loss of control and vaulting of errant vehicles. Therefore, curbs must be used sparingly and only after careful evaluation of other alternatives. In most cases, a curb is to be used only on urban facilities with limited right-of-way.

10.4.1 TYPES OF CURB

Curbs are described as either barrier or mountable. A barrier curb is any curb with a total vertical rise greater than 6 inches [150 mm] no matter what face or overall shape it may have. Therefore, mountable curbs are those with a vertical rise of 6 inches [150 mm] or less. Both barrier and mountable curbs may have a variety of cross sections. The Department's Standard Construction Details should be used in making the appropriate selection for the function to be served.

10.4.2 PLACEMENT OF CURB

The placement of curb has very serious effects on the behavior of drivers and vehicles. As indicated before, the vehicle can become uncontrollable and may even become airborne when a curb is hit. On rural facilities, the use of curb should be limited to channelization and delineation at intersections, crossovers, and entrances. On rural highways mountable type curb is preferred. On urban highways, curb use, while not preferred, is very common. On urban highways, mountable curb should be used for design speeds 50 mph [80 km/h] and above. Barrier curb may be used with design speeds of 45 mph [70 km/h] or less.

The typical effect on most drivers is to shy away from a curb because it is perceived as a danger and visually restricts the travel lane width. This poses the danger of vehicles shying into adjacent lanes. Barrier curbs should be offset 2 ft [0.6 m] from travel lanes and mountable curb offset 1 ft [0.3 m] to minimize this effect. The introduction of curb in itself poses a possible serious hazard and distraction to the driver. Newly introduced curbs should be offset and flared at least 2 ft [0.6 m] beyond the normal offset. In addition, the forward edge (facing traffic) of barrier curb should be gradually depressed at a 12:1 ratio until flush with the pavement or adjacent area. This type of installation will provide a safe end treatment for the driver and not interfere with snow plowing operations.

Curbs used for island channelization should be mountable and offset from the required pavement width by 1 ft [0.3 m]. The islands are to be located in order that they will not, in any way, restrict the travel lane or shoulder area.

The use of guardrail/curb combination should be discouraged where high-speed, high-angle impacts are likely. Where guardrail is used in conjunction with curb, the following shall be adhered to:

1. For design speeds 50 mph [80 km/h] or greater, guardrail shall be offset 10 ft [3 m] minimum from the front face of curb to the front face of guardrail.
2. For design speeds of less than 50 mph [80 km/h], guardrail shall be offset 6 ft [1.8 m] minimum from the front face of curb to the front face of guardrail.
3. Curb height shall be a maximum of 4 inches [100 mm] when curb is placed at the face of guardrail for all design speeds.

10.4.2.1 CURBS AT DEVELOPMENT ENTRANCES

If it is deemed necessary that a center island must be built, then the island should have mountable curb at a development entrance. Curb used in developments should be transitioned to the appropriate type of curb at the radius point where joining state right-of-way. The Department's manuals entitled *Standards and Regulations for Access to State Highways* and *Rules and Regulations for Subdivision Streets* should be referred to in making curb use determination.

10.4.2.2 CURBS AT COMMERCIAL ENTRANCES

The Department's *Standards and Regulations for Access to State Highways* and *Rules and Regulations for Subdivision Streets* manuals should be referred to for detailed information on commercial entrance design. Commercial entrance designs are to be coordinated with the Subdivision Section. In general, due to traffic volumes, it is normally desirable to have a curbed center island of the proper length at commercial entrances to separate traffic movements. The use of center islands will ensure that parking spaces or other internal traffic configurations will not be placed too close to the intersection with the state route. Allowing internal movements close to the main roadway could cause queuing onto the main roadway and substantial reduction in the operational efficiency of both the entrance and the roadway. The center-island should be built with mountable curb. The entrance radii and curb sections parallel to the state road should conform to those on the state road. Barrier curb should be used adjacent to sidewalks. The use or introduction of barrier curb adjacent to the higher-speed roadway is not recommended unless design consideration necessitates such use. For more detailed information see the Department's manuals entitled *Standards and Regulations for Access to State*

Highways and *Rules and Regulations for Subdivision Streets.*

10.4.3 ACCESS FOR THE DISABLED

For the disabled, it is the policy of the Department to provide accessible curb cuts and adjoining sidewalks at all crosswalk locations and at existing or planned mass transit service locations. The access consists of depressed ramps through curbs suitable for baby strollers, walkers, wheelchairs, etc. Such ramps are also to be provided in refuge islands where a crosswalk crosses a channelized island. Typical details of curb ramps are shown in the Department's Standard Construction Details.

10.5 RIGHT-OF-WAY

The designer is responsible for defining the limits of right-of-way necessary to construct, operate, and maintain the highway project. The following discussion is general in nature and the designer is to refer to Project Development Manual, for a detailed description for preparing right-of-way plans. In designing the project right of way, the designer must first identify actual construction limits from the cross sections to define the toe of fills or the top of cuts. Second, the designer must identify, through input from the Utilities Section, the reasonable real property needs of public or municipal utilities required to relocate facilities as a result of the proposed project.

When right of way needs are being defined, space for current and future bus stop improvements should be considered. Depending on the type of improvement to be installed, a width of up to 14 ft [4.3 m] may be needed (measured from back of curb). The minimum right-of-way needed at each stop whether existing or proposed is 8 ft [2.4 m].

The right of way limits should be of consistent width and wide enough to accommodate the construction limits and appropriate border areas necessary for ditching, utilities, and their maintenance. In urban or other highly developed areas where open ditching is inappropriate, desirable right-of-way border areas should be minimized to reduce negative impact on adjoining private property.

To minimize right-of-way design time and avoid plan changes, the designer should consult with personnel from the design support sections early in the design process, preferably at an on-site scoping meeting before completing the preliminary plans.

10.5.1 RIGHT-OF-WAY CONFIGURATION

Generally, right-of-way lines should be designed with as few breaks and changes as possible. The following general criteria should apply:

- Where there is need for a width change close to the P.C. or P.T. of a horizontal curve, the breaks should be made coincident with the P.C. or P.T. if possible.
- Where there is need for a change of width close to a property line between two different owners, the break in widths should be made at the property line. Where utilities exist or will be located adjacent to the right of way line, the right-of-way should be tapered rather than jogged, if possible. This will ease the installation of utilities along the alignment.
- Where width changes are required both right and left of the roadway, the break points should be made at the same station, if possible.
- Breaks in widths should not occur in drainage channels, roads or drives where installation of right of way monuments would be impractical.
- Where right-of-way may only be needed from a portion of a parcel, consideration

should be given to acquiring right of way through the entire parcel for future connecting projects, so that the owner will not have to be contacted again. This determination is made in coordination with other supporting sections.

- Do not define right-of-way lines with spiral curves. Where centerlines follow spiral curves, right-of-way lines should be described using a circular or compound circular curve of similar shape.
- Proposed right-of-way lines must be described in order for surveyors to lay them out in the field. Dimensions showing survey stations and pluses with offset distance right or left of construction centerline are required in addition to coordinates, bearings and distances.

10.5.2 EASEMENTS

Under certain conditions it is preferable to obtain an easement rather than to purchase right-of-way. There are two types of easements: (1) temporary and (2) permanent easements. The type of easement should be identified on the plans.

A temporary easement should be obtained where it is not necessary to obtain permanent possession of the land or permanent right of access to the land. Temporary easements are appropriate:

- For any areas where the Department will have no maintenance responsibility after the completion of the proposed project construction;
- Where relatively flat cut or fill slopes extend beyond the right-of-way line and the lateral clear zone or for grading purposes that may benefit the property;
- To obtain proper grade on private driveways and approaches;
- For channel changes and inlet and outlet ditches at drainage structures where future maintenance is not anticipated; and
- For construction working areas.

A permanent easement should be obtained where it is not necessary that the State own the land, but where perpetual interest is necessary. Examples are where the Department needs to access the property for future maintenance, repair, or replacement of the highway facility, its drainage systems or appurtenances and as provided for in a project's right of way or railroad agreement.

10.5.3 RIGHT-OF-WAY MONUMENTS

Right-of-way monuments may be placed to provide a permanent reference for re-establishing the centerline and right-of-way line. Right of way monuments should be located and punched so the center is on the right of way line. Details of a standard right-of-way monument are shown in the Department's Standard Construction Details.

10.6 FENCING

It is Department policy that installation of fences should normally be considered under one or more of the following conditions:

- For access restrictions on interstate or other designated controlled access highways,
- Replacement fence where an existing fence was removed because of right-of-way widening, or
- For locations where there is a documented need for fencing i.e. pedestrian and bicyclist safety or right-of-way negotiations.

The location of fencing depends on who will assume the ownership and maintenance responsibility, 1 ft [0.3 m] inside the right of way for DelDOT and 1 ft [0.3 m] outside if by others. The responsibility for installing fencing varies. Fencing required for

DelDOT purposes will normally be shown on the contract plans and included as a bid item for the contractor. The type of fencing will depend on the characteristics and use of the adjacent property.

Installation of replacement fencing or new fencing as the result of negotiating easements or property takings are normally included in the right-of-way settlement agreement. This agreement provides for the affected property owner to be responsible for construction and maintenance of any new fence, with installation reimbursement by the State. Occasionally, the right-of-way agreement will specify that this fencing be included as a contract item. Locations and quantities of fencing to be constructed in this manner are determined by Real Estate and coordinated with the designer.

10.7 UTILITY ADJUSTMENTS

This section is a general discussion of a project's relationship with utilities. The specific details are covered in Chapter 11, Plan Development. In addition, the Department has published the *Utilities Design Manual*, which clearly sets forth policies and procedures regarding the relationships among the Department, the public, and private utilities in Delaware. Much of the manual is related to the accommodation of utilities within the highway right-of-way and adjustments initiated by the utilities. Other sections define responsibilities and procedures related to needed utility adjustments resulting from proposed highway improvement projects. It is this second category that is of concern to designers.

In storm drain design, it is often possible to avoid conflicts with underground utilities by making minor adjustments in the line or grade of the storm drain. The designer should consider conflicts with any utility in making the final design to minimize relocations and conflicts. Relocations of utilities frequently delay the progress of construction and so should be avoided where possible. As further described in the manual, costs for relocating and adjusting utilities may either be the affected utility's or the Department's responsibility. The designer needs to recognize that no matter which party is responsible, the cost will be directly or indirectly passed on to the user.

The procedures and responsibilities for utilities adjustments set forth in the *Utilities Design Manual* are briefly summarized below in terms of the four phases of plan development.

10.7.1 SURVEY PLANS

The designer prepares survey plans showing the approximate project limits, existing detail, and project number(s) assigned to the project. Two sets of survey plans are provided for each utility and are transmitted by the Utilities Section.

The Utilities Section returns marked-up plans showing complete information on existing and abandoned facilities in the area. The information provided includes whether the facilities are aerial, surface or underground, sizes of pipes, numbers of conduits, approximate depths, and any private or commercial easements.

10.7.2 PRELIMINARY PLANS

The designer prepares preliminary plans showing the proposed alignment, profile, drainage, signal pole location, clear zones, right-of-way taking, existing utilities from survey plan data and other details. Two sets of plans are submitted by the Utilities Section to each potentially affected utility company, who reviews these plans and returns them showing their proposed work plan.

The designer, assigned utility coordinator and, in some cases, the utility company's representative review these plans to determine if the exact horizontal and vertical

location of certain facilities are an important part of the design considerations. Where major conflicts with underground utilities appear possible, the Utilities Section arranges for determining the exact field location of the existing underground installations, typically with taking test pits. The Utilities Section submits this information to the designer. The designer and utility coordinator determine which underground or aerial conflicts cannot be avoided and discuss alternatives with the affected utility company. It is very important that the potential impact on scheduling, current project estimate and the responsibility for preparing the design plans and quantities as well as the actual construction be identified at this time.

10.7.3 SEMI-FINAL PLANS

Semi-final plans are prepared showing the final centerline, profile, drainage, right of way and other details.

Two sets of semi-final plans for each utility are provided to the Utilities Section. These plans are delivered to each affected utility. A representative of the Utilities Section coordinates a review of the project's affect on each utility's facilities.

The utilities plot their proposed underground relocation work on the plans, and the information is returned to the designer through the Utilities Section. The Utilities Section arranges with the utility for any needed relocation work and formal agreements. The scope and schedule for the work is included in the project's utility statement that is furnished to the designer for preparation of the P. S. and E. package.

10.7.4 P. S. AND E. PLANS

When the final plans are completed and the project is advertised, one set of final plans is forwarded to each utility. If it has not previously been notified to start the adjustment, the utility is directed by letter from the Utilities Section to order materials and begin making the changes or alterations to their facilities.

10.8 SIDEWALKS

10.8.1. GOALS AND OBJECTIVES

Sidewalks are an integral part of the Department's transportation infrastructure program. They facilitate and encourage safe and convenient pedestrian travel within communities and among different land uses. They provide safe and reasonable access to public transportation and other alternative modes of transportation, thereby helping alleviate vehicular traffic and reduce emissions. They also reinforce the Americans with Disabilities Act (ADA) by increasing the access opportunity for mobility impaired individuals.

The incorporation of sidewalks and pedestrian amenities also provides better land use and transportation connections, thus encouraging more trips on foot, improve access to transit, improving access to adjacent land uses and communities, conserving energy, and enhancing the Department's vision for multi-modal and inter-modal transportation systems.

In addition, by providing this transportation option, the installation of sidewalks can be an effective means in reducing automobile dependence and use. It will assist Delaware's mission toward cleaner air under the requirements of the Clean Air Act Amendments of 1990.

With the addition and installation of proper sidewalks and pedestrian amenities, safety, accessibility, ridership, and more favorable perceptions of public transit service can be increased. Communities will also be able to safely link to other land uses and transportation modes, resulting in better and more desirable neighborhoods and land development practices.

The need for sidewalks in urban or suburban areas is most noticeable at points of community and urban development that result in pedestrian concentrations near or along the state's or municipality's transportation system (roads, rail, air, and water transport facilities). Such examples include: public and private transportation depots, local businesses, industrial receiving and distributing plants, corporate centers, shopping centers, malls, schools, medical centers, religious centers, hotels and places of residence.

The design of sidewalks should reflect the community or context in which they are to be placed through the choice of materials or proximity to a traveled way. Buffer zones between curb and sidewalk are preferred to enhance pedestrian perception of safety and convenience.

It is the Department's goal to ensure that all efforts have been undertaken in determining the need and justification for installing, reconstructing, improving, requiring, or extending sidewalks for its transportation projects as well as for other initiating parties in public or private development.

10.8.2 REGULATORY REQUIREMENTS

The specific state law that governs the installation of sidewalks is Title 17, Section 132(f), of the Delaware Code. In essence, the Department is to reconstruct disturbed sidewalks or install new sidewalks when constructing in an urbanized area, if there is a demonstrated present need or a reasonably anticipated future need. The Project Development Process determines whether such need for sidewalks does or will exist for all or any part of a project. Before arriving at a decision as to the need for sidewalk construction, the Department consults with the affected county planning department, Department of Education and the local school district. Within incorporated municipalities, the Department has a town/city maintenance agreement in force that should be reviewed as to its affect on the decision to reconstruct or install sidewalks.

Since the original law was enacted, additional requirements have been initiated that affect the decision making process on when and where to construct sidewalks, the design and construction standards, and the review and field inspection procedure. The need for and encouraging the use of sidewalks are important parts of the Americans with Disabilities Act, the Clean Air Act, federal legislation funding transportation programs and the State's Long-Range Plan for Transportation. Sidewalks shall be determined as part of the Department's Project Development Process and scoped as part of the handoff prior to design initiation.

The interest of the disabled on state and federally funded projects are represented by the State's Architectural Accessibility Board. This board adopts standards and criteria to address service and accessibility for the disabled and handicapped and is the regulatory agency for ensuring compliance with all applicable standards and criteria during design and in construction.

The Department applies accessibility standards during the design and construction of transportation facilities based on project initiation, scope, and funding authorization. As a part of the initiation process, sidewalk facilities and connector points for new sidewalks, at, along, or between a community's public roadways and transit service are also considered to ensure that the elderly, disabled, and handicapped have access and use of our highway and transit facilities. The Project Design Checklist provides guidance for submission of plans to the Architectural Accessibility Board for review.

The appropriate guides to ensure conformance include: the current State of

Delaware Architectural Accessibility Standards; Part II of the Federal Register's Architectural and Transportation Barriers and Compliance Board (36 CFR Part 1191) - dated July 26, 1991; and Part IV of the Federal Register's Department of Transportation (49 CFR Parts 27, 37, 38) - dated September 6, 1991.

10.8.3 DESIGN APPROACH

Determining the need for sidewalks is a standard component of the Department's process to plan, design, construct, and reconstruct transportation projects and all multi-modal and inter-modal networks.

During the project development process, all projects are evaluated for consideration of the removal and/or relocation of roadside appurtenances, street furniture, vegetation, mailbox posts, landscaping, public or private utilities, poles, guardrail, and/or traffic signs in order to satisfy and encourage safe pedestrian movement. The installation of any obstruction directly within a sidewalk buffer strip (area between edge of curb or shoulder and edge of sidewalk facility) which prohibits or blocks safe pedestrian movement should be avoided. If prohibitive objects are placed directly within/along a sidewalk location, consideration should be given to relocating or repositioning that conflicting sidewalk section.

Certain circumstances such as geographic design, engineering, environmental, safety constraints, or the extent and scope of the project itself, may require sidewalks to be constructed along one side of a roadway, transit corridor, or land use area. For the most part, transportation projects involving a roadway or transit corridor should have sidewalks on both sides. Sidewalks may be omitted on one side of the street where that side clearly cannot be developed and where there are no existing or anticipated uses that would generate pedestrian trips on that side.

Because of legal and design criteria differences, sidewalks are not to be designed, signed, and accommodated for other than pedestrian use. Therefore, a sidewalk/pedestrian facility should not be identified, signed, or striped for some other transportation or recreational use (such as a bicycle). If shared facilities (such as both pedestrian and bike use) are desired, policies and design guidelines reflective of the joint use shall be followed.

The incorporation of sidewalks should be consistent with all other state, county, city, and/or town sidewalk policies, ordinances, or mandates, including the Department's Long-Range Transportation Plan. The Department's Division of Planning will ensure sidewalk provisions and requirements are coordinated with other transportation and land use projects in the state.

Future maintenance responsibility for sidewalks is an important consideration and should be clearly defined before installation.

10.8.4 GUIDELINES FOR ASSESSING THE NEED AND CRITERIA

Sidewalks are considered during the project development phase when evaluating a roadway or transit facility for construction, reconstruction or rehabilitation. The Department, another party or a different agency, regardless of any roadway or transit improvement may initiate sidewalks as independent projects. Therefore, sidewalks should be considered for all Department transportation networks unless it is specifically determined that:

- Sidewalk construction conflicts with public safety.
- Sidewalk cost and economic impact of construction is prohibitive in relation to the need, the benefits realized, or their probable use.
- Specific land use factors indicate there is no current or future need for sidewalk.

During the planning and project initiation phase, facilities for pedestrian movement

warranting sidewalk are considered for their impacts on project costs, right-of-way needs, additional environmental constraints, highway or bridge geometrics/standards, and public and legislative input. When planning, initiating, or recommending the location or relocation of transit service, the Department considers installing sidewalks as well as other forms of pedestrian accommodations to address safety and accessibility. More specifically, the location and environmental study report identifies the initial need for the construction, reconstruction, improvement, or extension of sidewalks.

When pedestrian facilities are provided, special needs of the young, elderly, disabled, and handicapped are to be considered. This may include extending a project's limits for reasonable distances to accommodate better access and safety.

Within the context of multi-modal and inter-modal systems (i.e. linking various land uses and destinations with accessible transportation systems), the location and provisional extent of pedestrian facilities are studied to logically connect their termini.

During the project development process, several agencies, and departments are involved ensuring reasonable consideration of pedestrian needs are met. Where appropriate, these groups help assist in identifying, justifying, securing funding and determining the extent of the sidewalk and pedestrian amenities.

Representatives from the Department of Transportation include the Bicycle and Pedestrian Coordinator, the appropriate District Engineer, the Traffic Engineering and Management Section and the Delaware Transit Corporation. Representatives from outside DelDOT include the Architectural Accessibility Board, county and local planning agencies, the appropriate Metropolitan Planning Organization, the local school district affected by the project, and the Department of Education Transportation Supervisor.

The public and other entities involved in pedestrian and transportation safety may identify and suggest sidewalk and pedestrian accommodations. This is usually realized during or after a project's public workshop/hearing. The local citizenry may also identify problems overlooked.

Projects being initiated or reviewed by the Department consider several factors in determining the appropriateness of constructing, reconstructing, extending, or requiring sidewalks. The following is a discussion of these factors.

Existing and expected land use patterns, growth areas, and generators of pedestrian movements are considered as follows:

- Land use—residential (high/cluster, medium, low), business/commercial, mixed uses, industrial, recreational, educational, agricultural, and open space.
- Growth areas—targeted, expected, or recommended areas of urban growth, usually around corridors of current or planned highway capacity improvements, sewer, or water upgrades.
- Specific generators of pedestrian movement—major employment areas (more than 100 employees), schools, entertainment special events, shopping centers/malls, residential neighborhoods, medical centers, religious centers, colleges and universities, bus stops, depot and transit locations, public and private parking garages/facilities, parks and recreational areas.
- Whether or not the Department, county, city, municipality, or other public or private organization plans on some type of capital improvement involving transportation infrastructure.

Existing and anticipated pedestrian characteristics are analyzed including:

- Special user groups—children, adolescents, elderly, disabled, handicapped, commuters, and those dependent upon or utilizing public transit.
- Trip purpose—shopping errands, commuting to school, work, or place of destination, visiting friends, recreational/entertainment, child care, vacation.
- Frequency of use—daily, weekends, seasonal, as needed.
- Other factors—weather conditions with season, time of day, holidays, school and college schedules, safety, convenience, transit routes.

Existing site characteristics affecting pedestrian use are reviewed including available parking (free, metered, hourly, monthly), roadway surface condition, shoulders, pavement markings, crosswalks, street lighting, phased signals, type of transit accommodation, accessibility to destination, access control, intersection links, safety factors, location of signs, channelization, slopes, and drainage.

An analysis of current or proposed land uses along or near a roadway corridor is made to determine if there is a failure to provide or include a design that incorporates sidewalks and pedestrian amenities. The effect of adding new sidewalks in the area is reviewed to ensure user safety. Environmental constraints such as wetlands, floodplain, steep slopes, historical properties or archaeology, hazardous contamination sites, rare or endangered species and farmlands are considered.

A final consideration is whether or not an intersection, roadway or transit corridor, subdivision or land use area can be effectively redesigned or retrofitted with sidewalks and pedestrian facilities.

Information not readily available can be obtained through the following methods:

- Observation,
- Discussion with local governments, planning groups, property owners, civic organizations, and task forces,
- Public information meetings/brochures,
- Accident reports,
- Questionnaires,
- Architectural Accessibility Board,
- Local School Districts,
- Delaware Bicycle Council,
- DNREC's Parks and Recreation, Soil and Water, Technical Services, and Air and Waste Management sections,
- State Historic Preservation Office, or
- Soil Conservation Service-U.S.D.A.

10.8.5 WARRANTS

Warrants based on pedestrian volume have not been established for sidewalks. Actual volumes counted at any one time may not reflect the demand for pedestrian use. Factors such as poor existing facilities (which discourage use), weather conditions, school schedules, holidays, proposed land use changes, growth areas, proximity of transit and depot locations/stops, and other factors affect current pedestrian use. Therefore, many of the benefits from the construction, reconstruction, or the extension of sidewalks and pedestrian facilities are not quantifiable with the actual magnitude of the safety benefit unknown. This is partially because individuals tend to walk where there are sidewalks and sidewalks tend to be installed where people are walking. In addition, pedestrian use and volumes are not regularly collected by planning and transportation agencies and cannot be easily forecasted, modeled or predicted.

The need for sidewalks should be related to the functional classification of streets. For example, collector streets are more likely to

have greater pedestrian use and volumes than residential streets. Collector streets are normally used by pedestrians to access public transit, commercial developments or other various land uses on the arterial to which they feed. Sidewalks should definitely be provided along developed frontages of arterial streets in land use zones that promote pedestrian activity.

Sidewalks should be considered whenever there is regular or periodic pedestrian travel along an existing roadside, street or transit corridor. Sidewalks should also be considered along any street, highway or transit corridor in developed areas not provided with shoulders even if existing pedestrian activity appears light.

10.8.6 DESIGN GUIDANCE FOR SAFE PEDESTRIAN CIRCULATION

Whenever a project is being initiated or planned where pedestrian movement exists or is anticipated, the initial scope and planning involved with the project should provide suitable space within the current or future right-of-way for safe pedestrian circulation. Sidewalks to be financed and maintained by others may also be included in a project.

When there are existing shoulders or walkways intended for pedestrian use, sidewalks should be evaluated at the project development stage for condition, suitable width, continuity, and limits. Where shoulders are being utilized for pedestrian movement, installing a parallel sidewalk may be considered because shoulders must be carried through intersections where turning lanes and pedestrian areas should not be combined.

Deteriorated sidewalks need to be evaluated for rehabilitation or reconstruction and additional width as necessary. Incomplete systems should be considered for connection to new sidewalks and end at logical terminations.

During the subdivision review process often dedicate right-of-way for transportation corridors and/or open space. The dedication of open space may often include areas close to a roadway edge, providing a buffer zone. County sidewalk policies provide for limited pedestrian circulation within a development. The maintenance of such pedestrian facilities becomes the responsibility of the development, local ordinance, adjacent property owner, or governing entity.

Pedestrian and sidewalk projects at intersections or along highway/transit corridors may include design, redesign, or traffic calming measures. These could include tightening of turn radii, channelized islands, medians and curbs, refuge islands, roundabouts, bulbs, neckdowns, signing, striping, transit shelters, and other features.

The extension of project limits beyond related highway or transit improvements for reasonable or short distances may be considered in order to include necessary pedestrian facilities at nearby intersections, provide safe access to public transportation facilities, or to avoid short sidewalk gaps. This decision can be reached anytime during the project development or design stage, and can be adjusted or deleted as needed. The project scope and funding may have to be revised accordingly.

10.8.7 PEDESTRIAN ACCIDENT HISTORY

An important factor in defining the need and locating and designing a sidewalk is accident history. Keep in mind that not all accidents result in a police accident report; supplement data from accident reports with field observations and discussions with stakeholders. The plan development process should include a study to define:

- Where and how did the accident occur: i.e. at an intersection or median, along the road, shoulder or existing sidewalk,

off the shoulder, or at a transit accommodation?

- Who was responsible?
- What was the pedestrian's origin and destination?
- What are the normal pedestrian movements in that area?
- Were there any existing pedestrian accommodations, lumination devices, warning signs, safety or traffic control devices, alcohol involvement or other contributing circumstances?

10.8.8 EXISTING SITE ACCOMMODATIONS

Typically included in the project development and design process is the assessment of the condition of the existing sidewalk network within the project and adjacent area. This assessment would include:

- The location of existing walkways, shoulders, worn paths, and greenway links;
- The location of incomplete walkway systems that adjoin or are within existing right-of way;
- The condition of existing facilities and how well they function or accommodate pedestrian movement;
- Any limiting geographical or architectural features that enhance or reduce feasibility of constructing pedestrian facilities;
- Any major or minor modifications in road or transit design that may enhance or reduce feasibility of constructing pedestrian facilities;
- Existing transit or depot stops with pedestrian or roadside amenities; and
- Whether surrounding or adjacent residential subdivisions, commercial or business land development, mixed land uses, or other developing land uses have provided or include a design that incorporates sidewalks and pedestrian amenities for school bus stops, greenway or walkway links to other land uses, and transit stop access for safe pedestrian movement and circulation.

10.8.9 PEDESTRIAN FACILITY LAYOUT

New and upgraded pedestrian facilities must conform to ADA standards; the information herein does not include all ADA standards. Contact DelDOT's ADA Coordinator and refer to DelDOT's *Standard Construction Details* for additional information.

Avoid warped surfaces within the pedestrian accessible route as much as possible. Warped surfaces can cause a wheelchair to become unstable by causing a wheel to lose contact with the ground.

The minimum vertical clearance for pedestrian space is 80 in [2 m]. Objects protruding into this space such as signs and utility boxes present a hazard for the visually impaired.

10.8.9.1 SIDEWALK REQUIREMENTS

For new sidewalks a minimum width of 5 ft [1.5 m], not including the width of the top of curb, is required. For new sidewalks at underpasses or overpasses, or where there is an obstruction for which the sidewalk must be narrowed, a minimum width of 4 ft [1.2 m] is permissible. If the sidewalk is narrower than 5 ft [1.5 m], then passing spaces at least 5 ft by 5 ft [1.5 m by 1.5 m] shall be provided at intervals not to exceed 200 ft [61 m]. Wider sidewalks may be preferred or required by local ordinance depending upon the volume and nature of pedestrian traffic. Minimum sidewalk thickness can vary according to materials, but be at least 4 in [100 mm] for Portland Cement Concrete. A minimum thickness of

6 in [150 mm] is required at entrance and driveway areas. The maximum cross slope is 2%, even where the sidewalk crosses a driveway entrance.

Small planting strips between the sidewalk and curb may not be practical unless the property owners, civic associations or volunteer programs can make provisions for maintenance. For increased user safety, sidewalks should be as far away from travel lanes as practical. Where possible a buffer width of at least 3 ft [0.9] m between the edge of a sidewalk and the edge of a shoulder, back of curb, or traveled way is preferred. A 3 foot [0.9 m] wide strip would improve safety, driver comfort, and provide an area for snow removal, signage and mailbox posts.

In central business districts, commercial areas, apartment complexes and generally where buildings or parking areas lie near or on the right-of-way, consideration should be given to pave the entire width from curb to building, property, or right-of-way line. The minimum desirable width of sidewalk between curb line and building face is 8 ft [2.4 m]. This permits space for utilities and other roadside appurtenances, and limited snow.

Standard material for any sidewalk or walkway is usually Portland Cement Concrete. However, sidewalk or walkway materials are not limited to Portland Cement Concrete. Upon approval and when funding is available, more aesthetic materials such as brick, asphalt, or other stable, firm, slip-resistant material surfaces may be used. This may be appropriate for traffic calming areas and in certain circumstances to address the concerns expressed by land use planners and/or communities that concrete sidewalks are aesthetically unpleasing.

When constructing, reconstructing, or extending sidewalks at or near intersections, the design should consider enhancing accommodations for pedestrians and vehicles throughout the intersection. Such elements may include refuge islands, separate pedestrian signal indications and phases (with pedestrian button in close proximity to the sidewalk) and offset room for traffic signs/poles and utilities.

In establishing the location of sidewalks, consideration will also be given to the need for or relocation of conflicting drainage facilities, side slopes, new traffic control devices and signing, intersection crossovers, striping, utility appurtenances, mailboxes with posts, and transit stops.

10.8.9.2 CURB RAMP REQUIREMENTS

Curb ramps shall be installed/upgraded for a project where the scope of work includes roadway reconstruction, resurfacing or any sidewalk construction. Examples of projects that do not require curb ramp installation are those that only consist of signing, pavement marking, lighting, and patching. For projects that fall into the qualifying categories, the following shall apply:

1. Curb ramps shall be provided where an accessible route crosses the curb line. An accessible route is a continuous unobstructed path, which includes sidewalks and other exterior pathways with an improved surface, intended for pedestrian use.
2. Curb ramps shall be provided at the termini of all existing and proposed crosswalks within the project limits if there is existing sidewalk or the project includes proposed sidewalk. Crosswalks may be marked or unmarked.
3. Curb ramps at marked crosswalks shall be placed within the crosswalk lines.
4. The curb ramp shall not be compromised by other highway features such as guardrail, utility poles, signs, drainage inlets, manholes or other obstacles. All options for working around obstacles must be considered. If an accessible route cannot be provided

around an obstacle, then the obstacle should be relocated. If the obstacle cannot be relocated then the Project Manager should work with his or her supervisor and assistant director to determine the best solution for the location. Curb ramps must be installed at all required locations when new sidewalk is constructed, existing sidewalk is reconstructed or a road is repaved. All curb ramp locations must be addressed and cannot be skipped when the road is repaved. Some obstacles may not be able to be relocated in order to make a pedestrian facility accessible, including structural elements such as bridge piers and buildings. In these limited cases where it may be technically infeasible to install a curb ramp at a specific location, the reason(s) must be documented.

5. Parking spaces shall not obstruct the accessible route associated with a curb ramp.

6. Positive drainage must be provided to keep water from ponding on curb ramps and in the street along the accessible route.

7. The accessible route shall be along a safe path for pedestrians. Curb ramps shall not be installed where it would be unsafe for any pedestrian to cross due to conditions such as uncontrolled intersections with high vehicle speeds, unsafe sight distance, or physical obstacles that cannot be crossed such as median barriers on multi-lane highways. If it is determined that a safe crossing cannot be provided, then pedestrians shall be routed to a safe, accessible crossing, and the unsafe location shall be constructed with a barrier that will direct pedestrians (including the visually impaired) to a safer crossing. Examples of barriers that could be used at unsafe locations include shrubbery, a grass strip, railings, bollards, a double-posted informational sign with two horizontal struts (one between 36 to 42 inches [0.9 to 1.1 m] and the other between 4 to 12 inches [0.1 to 0.3 m] above the ground), etc. Whatever is used would have to consider detectability by the visually impaired as well as safe roadside design considerations for the clear zone. Locations thought to be unsafe shall be reviewed by DelDOT's Traffic Section. If the crossing cannot be made safe then the Project Manager should work with his or her supervisor and assistant director to determine the best solution for the location and document the reason(s).

8. The designer needs to consider how curb ramp locations may negatively affect the operation and capacity of vehicular traffic.

Diagonal curb ramps (those placed along the curb radius, DelDOT Type 3) are not preferred when pedestrians will have to change direction in the roadway in order to complete their crossing. While it is recognized that there will be locations where they are the best option (such as at intersections with large turning radii), perpendicular curb ramps (DelDOT Types 2 and 4) are preferred. Diagonal curb ramps are often a good treatment when there are channelizing islands at right turn slip lanes. When diagonal curb ramps are used, a 5 ft [1.5 m] long bottom landing area must be provided between the curb radius and the curb line extensions, which is outside the direction of vehicle travel. If this clear distance cannot be provided, diagonal curb ramps shall not be used and another type of curb ramp must be provided. This clear distance is not necessary for right turn slip lane and channelizing island configurations or where there is a landing at the bottom of the ramp within the sidewalk.

The maximum running slope of curb ramps is 12:1. Flatter slopes should be provided where possible. Curb ramps in alterations where a 12:1 slope is not technically feasible may have a maximum running slope of 10:1 for a maximum rise of 6 inches [150 mm].

The maximum cross slope of a curb ramp is 50:1 (2%).

DelDOT's minimum curb ramp width is 60 inches [1.5m].

Where a 12:1 maximum curb ramp slope will not meet the sidewalk grade within a length of 15 feet [4570 mm] due to a steep adjacent roadway, the last ramped section transitioning back to existing pedestrian facilities may be limited to 15 feet [4570 mm], and the slope of that section allowed to exceed 12:1.

If turning or maneuvering is required within the curb ramp or landing area, a maximum slope of 50:1 (2%) in any direction must be provided. The minimum landing area dimension shall be 60 inches [1525 mm] x 60 inches [1525 mm].

Transitions from curb ramp to gutter and streets should be flush and free of level changes. There should not be a bump at the gutter flow line or where the curb meets the pavement. Changes in level up to ¼ inch [6 mm] may be vertical. Changes in level between ¼ inch [6 mm] and ½ inch [12 mm] must be sloped no greater than 2:1. Changes in level above ½ inch [12 mm] must be treated as a ramp and must be sloped at 12:1 or flatter.

Given the constraints and varying field conditions, each curb ramp must be evaluated and designed using these guidelines and the *Standard Construction Details*.

10.8.9.3 DETECTABLE WARNING REQUIREMENTS

Detectable warnings shall be installed at sidewalk curb ramps and at uncurbed sidewalks at the following locations:

- Crosswalks (marked & unmarked) and designated places where pedestrians cross public roadways (including medians and refuge islands);
- Signalized entrances;
- High volume entrances with ADT greater than about 400;
- Entrances with an operating speed of 25 m.p.h. or greater through the pedestrian area;
- Railroad crossings.

Detectable warnings may also be installed in other areas when determined necessary by engineering judgment. Factors which present a potentially hazardous situation may also be considered, including blind spots, complicated turning movements or other situations in which pedestrians with visual impairments should be signaled to stop. They should not be used at all entrances without consideration of the above criteria, since overuse can cause confusion for pedestrians with visual impairments.

A 24 inch [600 mm] long strip of detectable warnings (truncated domes) must be placed along the fully depressed width of the curb ramp at the transition to the street. Detectable warnings must also be placed at cut-through medians and cut-through islands.

10.8.10 MAINTENANCE RESPONSIBILITY

As a policy, DelDOT does not normally maintain sidewalks. Depending upon their location, applicable laws, local ordinances and the current town or city maintenance agreement, sidewalks are the maintenance and upgrade responsibilities of the property owner, homeowner's association, municipality, town, city, suburban or non-suburban area, incorporated or unincorporated area, or governing body which bears jurisdiction. Delaware Transportation Corporation usually contracts out maintenance of sidewalks and other passenger amenities immediately adjacent to transit facilities.

Projects involving pedestrian or sidewalk amenities proposed under the Transportation Enhancement program have a formal agreement with the responsible party that includes a description of maintenance standards to be upheld and assigns the responsibility for the associated costs for those amenities. Maintaining a pedestrian or sidewalk facility involves several items including snow removal, trash and debris removal, control of vegetation, reconstruction, graffiti removal, resigning or re-striping (specifically related to the sidewalk), avoiding general neglect and deterioration for whatever cause and alterations of the surface or subsurface level required to improve the appearance.

Projects under the Suburban Streets and Resurfacing Program may include repairs, replacement or general maintenance to existing deteriorated sidewalks. For all new construction and replacement, accessibility guidelines and standards are to be followed.

DelDOT repairs or replaces any existing sidewalk surface that has been damaged or altered by DelDOT forces or its contractors. Repair or replacement of sidewalks follows all accessibility guidelines and standards.

10.8.11 REMINDERS

When sidewalks are proposed or initiated without a formal agreement to the contrary, DelDOT may not assume any maintenance responsibility.

Although not to be addressed as part of the project, it may be beneficial to consider the need for future sidewalk and reserve the right-of-way.

Delaware's Department of Education does have busing rules affecting the busing rights for children. These may come into play if certain communities and schools are connected with continuous sidewalk access. The most significant affect is that public busing privileges may be revoked.

Local governments or jurisdictions may adopt land use or subdivision ordinance regulations to protect transportation facilities, corridors, and sites for their identified functions. This could include, but is not limited to, facilities providing safe and convenient pedestrian or bicycle access within and from new subdivisions, planned unit developments, transit stops, greenways, and neighborhood activity centers such as schools, parks, and shopping areas.

If not required under county, city, or local jurisdiction, DelDOT may request or require sidewalks and pedestrian facilities to be installed prior to subdivision entrance permit approval. DelDOT may also request the installation of sidewalks and pedestrian facilities along roadway or transit corridors as part of the mitigation plan under a traffic impact study.

As a part of improving the transportation network, DelDOT initiates and designs sidewalk projects to connect existing and future transit routes, transit facilities, park-and-ride lots, public/private parking areas, bus stops for schools, businesses, shopping centers, industrial parks, residential communities, or any other public or private institution. This enhances multi-modalism while decreasing vehicular traffic and automobile emissions.

10.8.12 FUNDING ALTERNATIVES

Although specially defined on the Project Initiation Form, the designer should be aware that the construction, reconstruction, or extension of sidewalks can be funded by several different methods and funding sources.

Projects initiated by DelDOT are usually 100% funded by the Department and include:

- Removal and replacement of existing sidewalk caused by the construction, reconstruction, improvement, or extension of any highway, transit,

safety, or pedestrian related improvement;

- Projects facilitating State or DelDOT transportation purposes. This may include, but is not limited to, improving or expanding transit facilities, walkway/pedestrian corridors, greenway links, or safety improvement projects.

Projects initiated by others for design and/or construction by DelDOT have various matching fund ratios. These include:

- Projects may be 100% funded under the Suburban Streets and Resurfacing Program for new sidewalk projects that have been approved and initiated by DelDOT in recognition of meeting the needs of the public, town officials, or other governing bodies.
- Projects may be funded and initiated by a school district or other agency with a 50% match by DelDOT. These would involve new or reconstructed sidewalks within a project's termini or short distances outside a project area to connect sidewalks to existing pedestrian or transit generators from or to educational facilities.
- Projects may be 100% funded and initiated by a party or agency for removal and replacement of deteriorated sidewalk.
- Projects may be 50% jointly funded by a party or agency for utility adjustments, drainage, signals, pedestrian barriers, retaining walls, crossovers, etc. required solely for sidewalk safety and enhancement. This does not include projects initiated for ADA conformance.
- Projects may be 10% funded by an initiating party or agency, 10% by DelDOT and 80% funded by the Federal Highway Administration. These projects meet the criteria for funding under the federal Intermodal Transportation Efficiency Act. Qualified applicants can initiate projects to be included in DelDOT 's Transportation Enhancement Program. These projects must first gain approval from DelDOT's Technical Advisory Board. The projects are then subject to final review and approval by the Secretary of Transportation.

10.9 BICYCLE FACILITIES

There is a wide range of facility improvements that can enhance bicycle transportation. Suitable accommodations for bicyclists shall be determined as part of the DelDOT's Project Development Process and scoped as part of the project handoff prior to design initiation. Improvements can be simple involving minimal design considerations such as changing drainage grate inlets, or they can involve a detailed design such as providing a bike path.

Facility improvements for motor vehicles through appropriate planning and design can enhance bicycle travel or should at least avoid adverse impacts on cycling. Unless access is specifically denied, some level of bicycle use can be anticipated on most roadways. All new roadways, except those where bicyclists will be legally prohibited, should be designed and constructed under the assumption that they will be used by bicyclists. Guidelines are presented here to help design and construct roadway improvements and separate facilities that accommodate the operating characteristics of bicycles. Additional information including signing layouts, striping, and design details can be found in AASHTO's *Guide for the Development of Bicycle Facilities.*

Because most highways have not been designed with bicycle travel in mind, there are often many ways in which roadways should be improved to more safely accommodate bicycle traffic. Roadway conditions should be examined and, where necessary, safe drainage inlets, safe railroad crossings and smooth pavements should be provided. Drainage inlets and utility covers are potential problems to bicyclists. When

designing a new roadway, these types of appurtenances should be kept out of the bicyclists' expected path.

Parallel bar drainage inlet grates can trap the front wheel of a bicycle causing loss of steering control. Often, the bar spacing is such that it allows narrow bicycle wheels to drop into the grates, resulting in irreparable damage to the bicycle wheel and frame and/or injury to the bicyclist who could be thrown from the bicycle. These grates should be replaced with the bicycle-safe grates in the DelDOT's *Standard Construction Details*. Parallel bar grates are not to be used where bicycles may be present.

Railroad-highway grade crossings should ideally be at a right angle to the rails. The more the crossing deviates from this ideal crossing angle, the greater is the potential for bicyclists' front wheels to be trapped in the flange way causing loss of steering control. It is also important that the roadway approach be at the same elevation as the rails.

Consideration should be given to the materials of the crossing surface and to the flange way depth and width. If the crossing angle is less than approximately 45 degrees, consideration should be given to widening the outside lane, shoulder, or bicycle lane to allow bicyclists adequate room to cross the tracks at a right angle.

Pavement surface irregularities can do more than cause an unpleasant ride. Gaps between pavement slabs or drop-offs at overlays parallel to the direction of travel can trap a bicycle wheel and cause loss of control; holes and bumps can cause bicyclists to swerve into the path of motor vehicle traffic. Thus, to the extent practicable, pavement surfaces should be free of irregularities and the edge of the pavement should be uniform in width. On older pavements it may be necessary to fill joints, adjust utility covers or, in extreme cases, overlay the pavement to make it suitable for bicycling. Longitudinal joints in pavement and between pavement and gutter pans should not be more than ½ in [12 mm] wide. Longitudinal drop-offs between pavement and gutter pans or between travel lane pavement and shoulder pavement should not exceed ¾ in [18 mm]. Ridges used to create "rumble strips" in pavements should not be more than ¾ in [18 mm] when perpendicular to bicycle travel. Properly located warning signs should precede these locations.

Roadway treatments intended to accommodate bicycle use must address the needs of both experienced and less experienced riders. One solution to this challenge is to develop the concept of a "design cyclist" and adopt a classification system for bicycle users such as the following:

<u>Group A-Advanced Bicyclists</u> - Experienced riders who can operate under most traffic conditions. They comprise the majority of the current bicycle users of collector and arterial streets.

<u>Group B-Basic Bicyclists</u> - Casual or new adult and teenage riders who are less confident of their ability to operate in traffic without special provisions for bicyclists. The basic rider is comfortable riding on neighborhood streets and shared use paths but prefer designated facilities such as bike lanes or wide shoulder lanes on busier streets.

<u>Group C-Children</u> - Pre-teen riders whose roadway use is initially monitored by parents. Eventually they are accorded independent access to the system. Provisions should be made to allow access to key destinations without encouraging them to ride in the travel lane of major arterials.

In the design of bicycle facilities two broad classes of bicyclists are used: Group A riders and Group B/C riders.

Generally, the Group A bicyclist will be best served by designing all roadways to

accommodate shared use by bicyclists and motor vehicles. This can be accomplished by:

- Providing wide outside lanes or bike lanes on collector and arterial streets built with an urban section (i.e., with curb and gutter);
- Providing useable shoulders on highways built with a rural section (i.e., no curb and gutter).

Group B/C bicyclists are best served by a network of neighborhood streets and designated bicycle facilities that can be provided by:

- A network of designated bicycle facilities (e.g., bike lanes, separate bike paths, or side-street bicycle routes) through the key travel corridors typically served by arterial and collector streets.
- Useable roadway shoulders on rural highways, 4 ft [1.2 m] wide.

10.9.1 FACILITY SELECTION

The most significant factors affecting compatibility of roadways for bicycling are motor vehicle traffic volumes, operating speed, and the width of the travel lane and shoulder. The selection of facility type should be determined by an analysis of these factors in addition to the following:

1. State and local bicycle master plans;
2. Proximity of schools, parks and other destinations where a child bicyclist may be expected;
3. Presence of a regionally significant or locally designated bicycle route;
4. Potential turning movement conflicts; and
5. Environmental, historical and right-of-way constraints.

In general, additional travel lane or shoulder width can increase the suitability of a roadway for bicycling. Designation of bicycle lanes with appropriate signs and pavement markings will help increase the predictability of both bicycle and motor vehicle movements. Additional separation of bicycle traffic from motor vehicle traffic on shared use paths may be desirable on high speed, high volume roadways, where an increase in child bicyclists can be expected or along regional pathway networks. Development of a shared use path does not preclude the need to accommodate more experienced bicyclist on the roadway.

10.9.2 FACILITY TYPES

Four basic types of facilities are used to accommodate bicyclists:

1. Shared Roadway (No Bikeway Designation) - Shared lanes are streets and highways with no special provision for bicyclists. Shared lanes typically feature 12 ft (3.6 m) lane widths or less with no shoulders, allowing cars to safely pass bicyclists only by crossing the centerline where permissible or moving into another traffic lane. In residential areas with lane widths of at least 12 ft [3.6 m] low motor vehicle traffic volumes and average motor vehicle speeds of less than 30 mph [50 km/h], shared lanes will accommodate group A riders, and will normally be adequate for group B/C bicyclists. Where the existing lane width is less than 12 ft [3.6 m], additional lane width is called for. For higher speeds and traffic volumes, shared lanes become less attractive routes, especially to group B/C riders.
2. Shared Roadway, Signed - These roadways are designated by bike route signs and either provide continuity to other bicycle facilities or designate a preferred route through high demand corridors. In designating a roadway as a shared roadway, DelDOT must assure there are advantages to using this route, the route is suitable, and it will be

maintained in a manner consistent with the bicyclist's needs. The use of signing and striping will advise the motorist to expect bicyclists and be prepared to react safely.

3. Bike Lane - Bike lanes are established with appropriate pavement marking and signing along streets in corridors where there is a significant bicycle demand. In order to accomplish this, space must be provided or created for preferential use by bicycles. Adequate space, pavement markings, traffic control, pavement conditions, surface hazards etc. must all be addressed. This will give both the bicyclist and the motorist a level of predictability of how each will maneuver. Roads marked with bike lanes indicate that the road is recommended for safe cycling. This designation should not be made unless the bike lane will be a continuous safe route for the length of the bike lane (e.g. through intersections and turn lanes). In area where bicycle and motor vehicle traffic cross paths, pavement markings and signs should indicate the location of the bicycle lane and warn of potential conflict areas.
4. Shared Use Path - Shared use paths are on exclusive right-of-way and with a minimum of cross flow by motor vehicles. Users are non-motorized and may include bicyclists, in-line skaters, roller skaters, wheelchair users, and pedestrians, including walkers, runners, people with baby strollers, people walking dogs, etc.

10.9.2.1 DESIGN APPROACH

Depending upon the information provided on the project initiation form and/or as a result of the project-scoping meeting, the designer might have to evaluate one or all of the four types of facilities as a part of the context sensitive design approach. Bicycles are a part of the available transportation system. If properly addressed, increased bicycle use can contribute to the reduction in air pollution and motor vehicle use.

10.9.3 SHARED ROADWAY

Most bicycle travel occurs on streets and roadways that have no bikeway designation (shared roadway) because the existing street system is adequate to handle safe and efficient bicycle travel. In reviewing a project for shared use, the most important item to consider is whether or not adequate space can be provided to allow safe shared use by the motorist and bicyclist. The items to be considered include adding paved shoulders, increasing lane widths, eliminating on-street parking, improving pavement surface quality, and providing safe drainage grates, utility covers, traffic control boxes, etc. in the useable area.

Improving or adding paved shoulders may be the best alternative to serve both modes. Other benefits of having shoulders are improved mainline drainage, protection of mainline pavement structure, emergency use, etc. The most obvious project effects are increases in design effort, construction costs, and right-of-way needs.

Paved shoulders should be at least 4 ft [1.2 m] wide to accommodate the bicyclist. However, any shoulder widening will be an improvement to the overall use and safety of the project. For the bicyclist, the useable shoulder width does not include the gutter pan, unless the gutter pan is 4 ft [1.2 m] or greater in width. Where guardrail is used, the shoulder width should be increased to 5 ft [1.5 m]. Greater motor vehicle speeds and volumes combined with increased use by bicyclists require that more shoulder width be provided. Design controls found in other chapters of this manual will usually be applied in these situations and will be adequate to serve bicyclists.

The use of rumble strips or raised pavement markings adjacent to shoulders warning errant drivers or discouraging the use of shoulders by motorist can be a

problem for bicyclist. Rumble strips should be marked with a warning sign. Where they are to be used there should be:

(1) A clear path of 1 ft [0.3 m] from the rumble strip to the outside edge of the traveled way;

(2) A clear area of 4 ft [1.2 m] from the rumble strip to the outside edge of paved shoulder; or

(3) A clear area of 5 ft [1.5 m] to the adjacent guardrail, curb or other obstacle.

Where shoulders can not be provided, wider lane widths may be attainable. On roadway sections without designated bikeways, an outside lane, whether curbed or not, wider than 12 ft [3.6 m] can better accommodate the motorist, bicyclist and adjacent land use.

A useable lane width of 14 ft [4.2 m] is preferred when curb lanes are to be shared. This width does not include the gutter pan and is measured from the edge stripe to lane stripe or from the gutter pan edge to the lane stripe. Where there is a continuous steep grade, drainage grate interference, pavement reflectors, or on street parking, a curb lane width of 15 ft [4.5 m] is preferred. Too much width can be hazardous by encouraging the operation of two vehicles in a lane intended as a single lane operation.

Wide curb lanes have three widely accepted advantages. They can:

- Accommodate shared bicycle/motor vehicle use without reducing the roadway capacity for motor vehicle traffic;
- Minimize the real and perceived operating conflicts between bicycles and motor vehicles; and
- Increase the roadway capacity by the number of bicyclists capable of being accommodated.

Wide outside lanes require the least amount of additional maintenance of the different facilities. The sweeping effect of passing motor vehicles and routine highway maintenance is usually enough to keep the lane free of debris and in good condition for bicycling.

Wide outside lanes are especially valuable for Group A riders who are competent in sharing the roadway with motor vehicles. The same is not true for Group B/C riders. Except on residential or low-volume streets, wide outside lanes are not generally sufficient to provide the degree of comfort and safety required by less skilled bicyclists or children and will do little to encourage them to ride.

Wide curb lanes will be most applicable in urban areas on major streets where Group A riders will likely be operating. If no alternative exists for Group B/C riders, a bike lane or shoulder should typically be used.

The designer should not overlook the potential for encouraging bicycle use on rural routes. Adding 4 ft [1.2 m] of paved shoulder and a 4 in [100 mm] edge stripe can safely provide this.

Facilities with on-street parking provide greater opportunity for conflicts between cyclist and motorist since the rider will be between moving traffic and parked vehicles. The bicyclist is subject to opening car doors, exiting vehicles, extended mirrors that narrow travel space and reduced visibility. Where this type of operation is to be permitted, a parking lane of a minimum width of 12 ft [3.6 m] is needed.

The shared roadway can also be occupied by existing or proposed surface obstacles to the bicyclist. These include drainage grates, utility covers and traffic control appurtenances. Throughout the design and construction phases these obstacles should be ether eliminated or designed to accommodate bicycle use.

10.9.4 SIGNED SHARED ROADWAY

Signed shared roadways are those that have been identified as bicycle routes and are signed accordingly. The reasons for designating a certain route as preferred for bicycle use include:

- The route provides continuity to other bicycle facilities,
- The road is a common route through a high demand corridor,
- In rural areas, the route is preferred due to low motor vehicle volumes, aesthetics, or availability of paved shoulders, or
- The route extends along local streets and collectors leading to a neighborhood destination such as a park, school or commercial district.

Signing a particular route suggests that there are advantages to using this route rather than some other alternative. This designation implies that certain criteria have been established and are being maintained. Criteria to be considered prior to selecting a route for designation include:

- In high demand corridors, the route is a direct through route;
- The route connects discontinuous segments of other types of bicycle facilities;
- Traffic control devices have been adjusted to reflect increased bicycle use;
- Street parking has been removed or restricted where width is critical to provide safe travel;
- A smooth surface has been provided;
- Maintenance will be provided to remove accumulated debris and keep traffic control devices serviceable;
- Wider curb lanes are provided as compared to parallel alternative routes; and
- Shoulder or wider lane widths meet the established minimums.

10.9.5 BIKE LANES

A bike lane is a portion of the roadway that has been designated by striping, signing and pavement markings for the preferential or exclusive use of bicyclists. Four typical bike lane layouts are shown in Figure 10-6.

Bike lanes are considered when it is desirable to delineate available road space for preferential use by bicyclists. The beginning and end of the bike lane should be clearly signed and marked. Bike lanes should not be designated on roads where the lane must bend in an unsafe location.

Bike lane markings can increase a bicyclist’s (especially B/C riders) confidence in motorists not straying into his/her path of travel. Likewise, passing motorists are less likely to swerve to the left out of their lane to avoid bicyclists on their right. Provision with appropriate signage should be made for a continuous bike lane through turn lanes and at intersections. On high-speed roadways where a left turn lane is introduced, a bike lane should be continued on the right of the high-speed lane.

Bike lanes should be one-way facilities and carry traffic in the same direction as adjacent motor vehicle traffic. Two-way bike lanes on one side of the roadway are not recommended when they promote riding against the flow of motor vehicle traffic. Wrong-way riding is a major cause of bicycle accidents and violates the Rules of the Road as stated in Delaware's Uniform Vehicle Code. Bike lanes on one-way streets should be on the right side of the street, except in areas where a bike lane on the left will decrease the number of potential conflicts (e.g., conflicts with heavy bus

traffic). Contra flow bike lanes may be considered in exceptional circumstances in urban area where appropriate traffic control devices may be used to ensure the safety of users.

For roadways with no curb and gutter, the minimum bike lane width is 4 ft [1.2 m]. Certain edge conditions indicate the need for additional bike lane width. If parking is permitted, the bike lane should be placed between the parking area and the travel lane and have a minimum width of 5 ft [1.5 m].

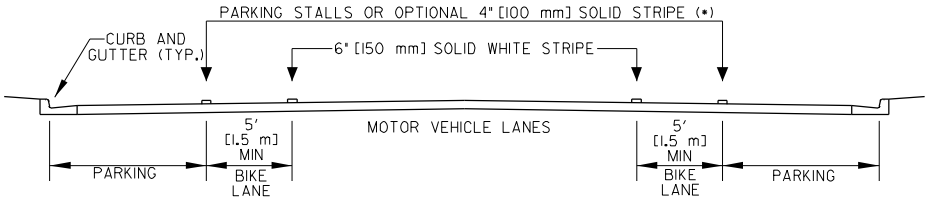
Where parking is permitted but a parking stripe or stalls are not utilized, the shared area should be a minimum of 11 ft [3.3 m] without a curb face and 12 ft [3.6 m] adjacent to a curb face. If the parking volume is substantial or turnover is high, an additional 1 to 2 ft [0.3 to 0.6 m] is desirable. Bike lanes between the curb and the parking lane can create obstacles for bicyclists from opening car doors and poor visibility at intersections and driveways, and they also prohibit bicyclists from making left turns. Therefore this placement should not be considered. Bike lanes are not advisable where angled parking exists.

Bike lanes along the outer portions of an urban curbed street, where parking is prohibited, also require special consideration. Bicyclists do not generally ride near a curb because of the possibility of debris, hitting a pedal on the curb, an uneven longitudinal joint, or a steeper cross slope. Bike lanes in this location should have a minimum width of 5 ft [1.5 m] from the curb face. If the longitudinal joint between the gutter pan and the roadway surface is uneven and falls within 5 ft [1.5 m] of the curb face, a minimum of 4 ft [1.2 m] should be provided between the joint and the motor vehicle lanes.

Bike lanes should be placed within the paved shoulder area at the outside edge in rural areas having a roadway section without curb, gutters and with infrequent parking. Bike lanes should have a minimum width of 4 ft [1.2 m]. Where the shoulder can provide additional maneuvering width, a width of 5 ft [1.5 m] or greater is preferable; additional widths are desirable where substantial truck traffic is present, or where there are excessive vehicle speeds.

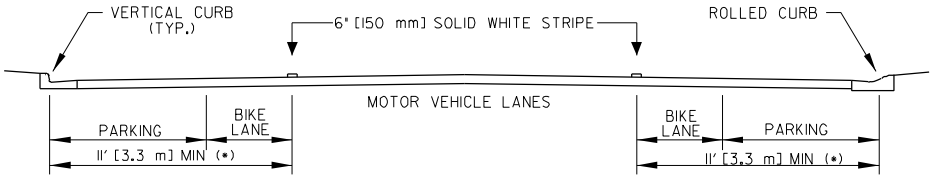
A bike lane should be delineated from the motor vehicle travel lanes with a 6 in [150 mm] wide solid white line. Bike lanes should be provided with adequate drainage to prevent water ponding, washouts, debris accumulation and other potential hazards to the bicyclist. Adequate pavement surface, bicycle-safe grate inlets, safe railroad crossing, and traffic signals responsive to bicycles should always be provided on roadways where bicycle lanes are being designated. Raised pavement markings and raised barriers can cause steering difficulties for bicyclists and should not be used to delineate bicycle lanes.

Figure 10-6
Typical Bike Lane Cross Sections



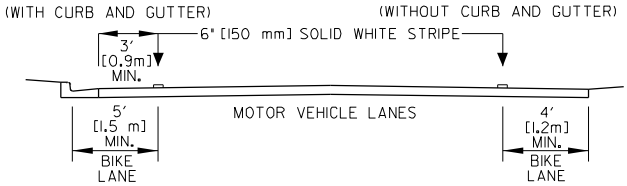
* - THE OPTIONAL SOLID WHITE STRIPE MAY BE ADVISABLE WHERE STALLS ARE UNNECESSARY (BECAUSE PARKING IS LIGHT) BUT THERE IS CONCERN THAT MOTORISTS MAY MISCONSTRUE THE BIKE LANE TO BE A TRAFFIC LANE.

(1) ON-STREET PARKING

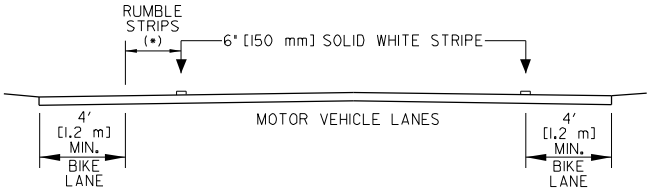


* - 13' [3.9 m] IS THE RECOMMENDED MINIMUM WHERE THERE IS SUBSTANTIAL PARKING OR WHERE TURNOVER OF PARKED CARS IS HIGH (E.G. COMMERCIAL AREAS).

(2) PARKING PERMITTED WITHOUT PARKING STRIPE OR STALL



(3) PARKING PROHIBITED



* - IF RUMBLE STRIPS EXIST THERE SHOULD BE 4' [1.2 m] MINIMUM FROM THE RUMBLE STRIPS TO THE OUTSIDE EDGE OF THE SHOULDER.

(4) TYPICAL ROADWAY IN OUTLYING AREAS PARKING PROTECTED

10.9.5.1 INTERSECTIONS WITH BIKE LANES

Bike lanes tend to complicate both bicycle and motor vehicle turning movement at intersections. Because they encourage bicyclists to keep to the right and motorists to keep to the left, both operators are somewhat discouraged from merging in advance of turns. Thus, some bicyclists will begin left turns from the right side bicycle lane and some motorists will begin right turns from the left of the bicycle lane. Both maneuvers are contrary to established rules of the road and result in conflicts.

At intersections, bicyclists proceeding straight through and motorists turning right must cross paths. Striping and signing configurations that encourage these crossings in advance of the intersection, in a merging fashion, are preferable to those that force the crossing in the immediate vicinity of the intersection.

The design of a bike lane needs to include appropriate pavement markings and signing approaching and through intersections to reduce the number of conflicts. Guidance for signing and pavement marking of bike lanes is shown in the MUTCD and AASHTO's G*uide for the Development of Bicycle Facilities.*

10.9.6 SHARED USE PATH

A shared use path is a facility that is physically separated from the roadway and intended for exclusive use of modes other than motorized vehicles. Initially perceived as bicycle paths, these facilities have grown in popularity serving bicyclists, in-line skaters, roller skaters, wheelchair users, and pedestrians, including, walkers, runners, people with baby strollers, people walking dogs, etc.

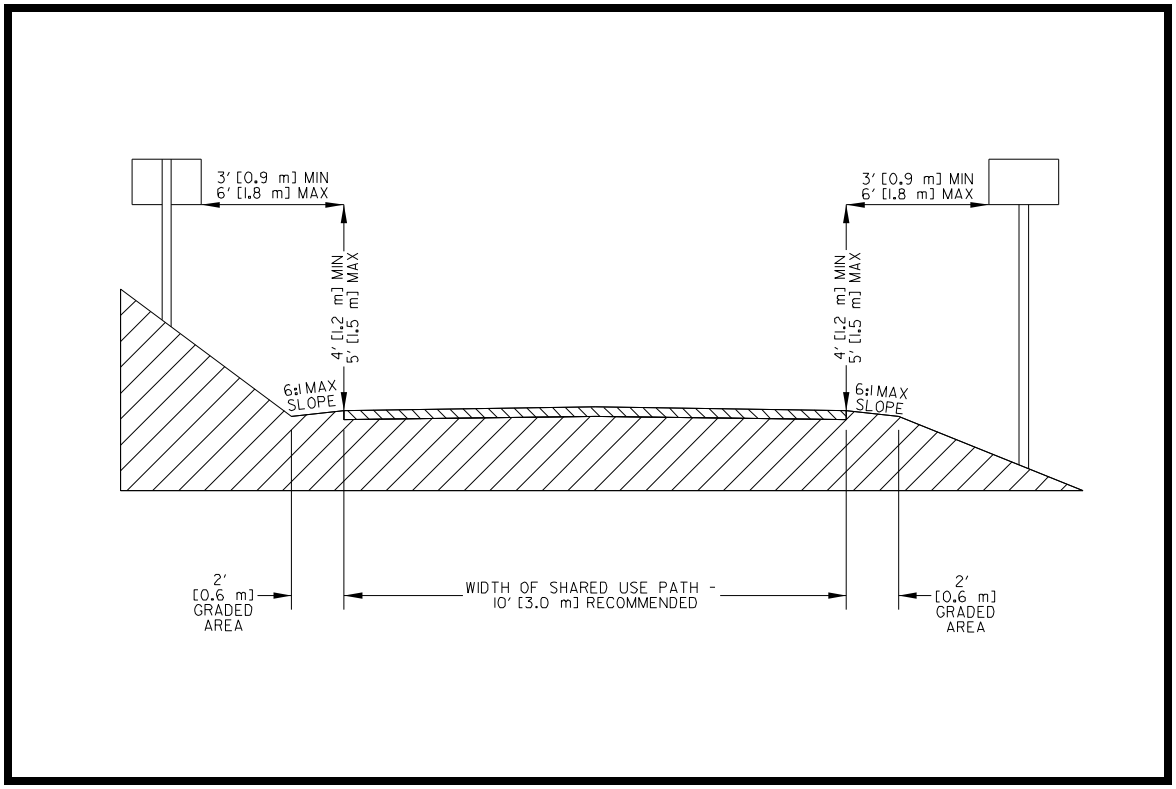
These facilities should be designed in accordance with the Americans with Disabilities Act standards for shared transportation paths. Maximum slope, cross slope and the rate of change in grade should be carefully examined during the design process. Because of their multi-use attraction they are a valuable addition to the highway system and to the range of facilities available to planners and engineers seeking to improve conditions for all categories of travelers. They can serve both a transportation and recreational function and have proven to be significant generators of bicycle use. Groups A and B/C riders (as well as other non-motorized users) can benefit from the absence of motor vehicle traffic on these paths. If bicycle and pedestrian volumes are expected and large numbers of conflicts are expected, a separate pedestrian facility may need to be considered.

The inclusion of a shared use path in a project would be the result of the planning process and be fully described in the project initiation documentation. Figure 10-7 shows the layout for a typical shared use path.

Shared use paths provide users with a shortcut through residential areas, provide enjoyable recreational opportunity, and provide access to areas not accessible by motor vehicle and areas only accessible by limited access highway facilities closed to this type of user.

Separate bike paths may be referred to as “multi-use trails” or “greenways” even though they are slightly different facilities. A trail typically runs along an independent right of way such as an abandoned railroad corridor, and a greenway is a park-type corridor of land that may or may not incorporate a trail within its boundaries.

Figure 10-7
Cross Section of Two Way Shared Use Path on Separate Right of Way



The design criteria categories that apply to shared-use paths are based on those used for highways including horizontal alignment, vertical alignment, vertical and lateral clearances, sight distance, grades and pavement structures. The similarity does not carry over into the actual criteria as the operational characteristics are significantly different.

Even when providing a shared path, street improvements to address this mode of transportation should not be overlooked. Many users will still use the local street system perceiving it to be more convenient, safer and better maintained, particularly for utility trips.

10.9.6.1 SEPARATION BETWEEN SHARED USE PATHS AND ROADWAYS

One of the most important elements in designing a successful shared path is the separation between the path and any adjacent roadways. Unless there is adequate separation there will be operational problems that make it more desirable and convenient for the user to use the roadway. The minimum separation of a bicycle path from a roadway is 5 ft [1.5 m]. When this minimum is not possible a suitable barrier at least 44 in [1.1 m] in height should be provided.

Problems associated with locating shared use paths too close to a roadway include:

- Requires one direction of bicycle traffic to ride against motor vehicle traffic;

- Upon entering or leaving a path bicyclist going against traffic tend to continue on the wrong side;
- Creates nonstandard flow conflicts at intersections;
- Signs posted for roadway users are backwards for the contra flow bicyclist;
- Many bicyclists will perceive the roadway as more convenient, better maintained or safer;
- At intersections and entrances, drivers expect the bicyclist to stop or yield but this does not usually happen;
- Stopped vehicles at cross-streets and vehicles exiting side streets and driveways block the path; and
- The distance between the two facilities may require the use of a barrier creating a hazard to the bicyclist and the motorist.

Because of the problems when shared use paths are located too close to a roadway, other types of bikeways may be more practical for accommodating bicyclists.

10.9.6.2 WIDTH AND CLEARANCE

Two-way shared paths should be a minimum of 10 ft [3.0 m] wide. In high use areas it may be desirable to increase the width to 12 ft [3.6 m].

A minimum graded area of 2 ft [0.6 m] in width should be maintained adjacent to both sides of the pavement. However 3 ft [0.9 m] or more is desirable to provide clearance from trees, poles, walls, fences, guardrails, or other lateral obstructions. A minimum 5 ft [1.5 m] separation from the edge of the path pavement to the top of slope is desirable. Depending upon the height of embankment, it may be necessary to provide a physical barrier.

The minimum width of a one-directional shared use path is 6 ft [1.8 m]. However, without adequate enforcement, it can be anticipated that the path will be used as a two-way facility and designed accordingly.

The vertical clearance to obstructions should be a minimum of 8 ft [2.4 m], however, vertical clearance may need to be greater to permit passage of maintenance vehicles and, under crossings and tunnels, a clearance of 10 ft [3 m] is desirable for adequate vertical shy distance.

10.9.6.3 DESIGN SPEED

The bicyclist controls the design speed and is dependent on several factors, including the type and condition of the bicycle, the purpose of the trip, the condition and location of the shared use path, the speed and direction of the wind and the physical condition of the bicyclist. Shared use paths should be designed for a selected bicycle speed that is at least as high as the preferred speed of the faster bicyclist. In general, a minimum design speed of 20 mph [30 km/h] should be used; however, when the grade exceeds 4 percent a design speed of 30 mph [50 km/h] is advisable.

For most bicycle path applications the superelevation rate will vary from a minimum of 2 percent (the minimum necessary to encourage adequate drainage) to a maximum of approximately 3 percent (beyond which maneuvering difficulties by slow bicyclists and adult tricyclists might be expected). The minimum superelevation rate of 2 percent will be adequate for most conditions and will simplify construction.

For a superelevation rate (e) of 2 percent, the minimum radii of curvature is 80 ft [25 m] for a design speed of 20 mph [30 km/h] and 260 ft [80 m] for a design speed of 30 mph [50 km/h]. When substandard radius curves must be used on bicycle paths due to right-of-way restrictions, topographical or other considerations, standard curve warning signs and supplemental pavement markings should be installed in accordance with the MUTCD. The negative effects of

substandard curves can also be partially offset by widening the pavement through the curves.

10.9.6.4 GRADES

Grades on shared paths should be kept to a minimum, especially on long inclines. Grades greater than 5 percent are undesirable because the ascents are difficult for many bicyclists to climb and the descents cause some bicyclists to exceed the speeds at which they are competent. Where terrain dictates, grades over 5 percent and less than 500 ft [150 m] long are acceptable when a higher design speed is used and additional width is provided. Grades steeper than 3 percent may not be practical for shared paths with crushed stone or other unpaved surfaces because of handling and increased maintenance problems.

If excessive grades must be used there are several alternatives to address this problem. These are:

- Widen the path to allow slower users to walk;
- Provide signing, alerting users of the steep slope ahead;
- Provide recommended speed signs,
- Exceed the minimum stopping sight distance;
- Exceed the minimum horizontal clearances, recovery areas, and if necessary add protective rails; or
- Design the path with a series of switchbacks.

10.9.6.5 HORIZONTAL ALIGNMENT

Unlike a motor vehicle, a bicycle must be leaned when entering a curve to prevent it from falling outward. If the bicyclist pedals through a curve and leans too far, the pedal may strike the surface. Depending upon the ability of the bicyclist, the lean angle may vary. Pedal heights may differ, but generally a 25° lean angle will cause the pedal to strike the surface. In design, a lean angle of $15\text{-}20^{o}$ should be used.

The following equations are used to determine the minimum radius of curvature.

For US Customary Units:

$$R = \frac{0.067V^2}{\tan\theta}$$

For Metric Units:

$$R = \frac{0.0079V^{2\prime}}{\tan\theta}$$

Where:

R = Minimum radius of curvature ft [m]
V = Design Speed mph [km/h]
θ = Lean angle from the vertical (degrees)

When the lean angle approaches 20^0, the minimum radius becomes a function of the superelevation of the pathway surface, the coefficient of friction between the bicycle tires and surface and the speed of the bicycle. For this design condition, the minimum radius or curvature can be derived from the equation:

For US Customary Units:

$$R = \frac{V^2}{15\left(\frac{e}{100} + f\right)}$$

For Metric Units:

$$R = \frac{V^2}{127\left(\frac{e}{100} + f\right)}$$

Where:

R = Minimum radius of curvature ft [m]
V = Design speed mph [km/h]
e = Rate of bikeway superelevation (percent)
f = Coefficient of friction

The equation variables are limited in range. Shared paths are subject to the Americans with Disabilities Act. Cross slopes are limited to a maximum of 3 percent. Friction factors are controlled by the rider's sense of discomfort. Figures 10-8 and 10-9 provide an easy reference for design application.

If the minimum radius is not attainable, warning signs and other supplemental signing as per the MUTCD should be installed

10.9.6.6 SIGHT DISTANCE

The sight distance accommodating bicycle traffic usually controls the design of a shared use path. In some cases, sight distance for pedestrians and wheelchair users should be considered as well.

Figure 10-8
Desirable Minimum Radii for Paved Shared Use Paths[1]

Design Speed (V) mph [km/h]	Minimum Radius (R) ft [m]
12 [20]	36 [12]
20 [30]	100 [27]
25 [40]	156 [47]
30 [50]	225 [74]

[1]Based on a 15^0 lean angle

Figure 10-9
Minimum Radii for Paved Shared Use Paths[1]

Design Speed (V) mph (km/h)	Friction Factor (*f*) (paved surface)	Minimum Radius (R) ft (m)
12 [20]	0.31	30[10]
20 [30]	0.28	90 [24]
25 [40]	0.25	1155 [47]]
30 [50]	0.21	260 [86]

[1]Paths Based on 2% Superelevation Rate and 20^0 Lean Angle

To provide bicyclists with an opportunity to see and react to the unexpected, a shared use path should be designed with adequate stopping sight distance. The distance required to bring a bicycle to a full controlled stop is a function of the bicyclist's perception and brake reaction time, the initial speed of the bicycle, the coefficient of friction between the tires and the pavement, and the braking ability of the bicycle.

The minimum stopping sight distance of bicycles is based on a total perception and brake reaction time of 2.5 seconds and a coefficient of friction of 0.25, to account for the poor wet weather braking characteristics of many bicycles. For two-way bicycle paths, the sight distance in descending direction, that is, where G is negative, will control the design. Sight distance is calculated using the following equation(s):

For US Customary Units:

$$S = \frac{V^2}{30(f \pm G)} + 3.67V$$

For Metric Units:

$$S = \frac{V^2}{254(f \pm G)} + \frac{V}{1.4}$$

Where:

S = Minimum stopping sight distance, m
V = Design Speed, mph [km/h]
G = Grade, ft/ft [m/m] (descending - Neg., Ascending +Pos.)

The minimum length of vertical curve necessary to provide minimum stopping sight distance at various speeds on crest vertical curves is given by the following equation. The assumptions used for the equation are the eye height of the bicyclist is 4.5 ft [1.4 m] and a object height of zero recognizing that impediments to bicycle travel exist at pavement level.

For US Customary Units:

When S>L L = 2S-900/A
When S<L L = AS^2/900

For Metric Units:

When S>L L = 2S-(280/A)
When S<L L = AS^2/280

Where:

L = Minimum Length of Vertical Curve ft (m) ≥ 3 ft [1.0 m]
S = Stopping Sight Distance ft [m]

A = Algebraic difference (%)

Height of bicyclist eye = 4.5 ft [1.4 m]

Height of object = 0 ft [0 m]

Bicyclists frequently ride abreast on bicycle paths. On narrow bicycle paths, bicyclists have a tendency to ride near the middle of the path. For these reasons, and because of the serious consequences of a head on bicycle accident, lateral clearances on horizontal curves should be calculated based on the sum of the stopping sight distances for bicyclists traveling in opposite directions around the curve. Where this is not possible or feasible, consideration should be given to widening the path through the curve, installing a yellow center stripe, installing a curve ahead warning sign in accordance with the MUTCD, or some combination of these alternatives.

Tables and curves for solutions of both sight distance equations, under various design conditions, can be found in AASHTO's *Guide for the Development of Bicycle Facilities.*

10.9.6.7 INTERSECTIONS

Intersections with roadways are important safety considerations in shared path design. There are three basic types of path-roadway intersections: mid-block, adjacent path and complex. If alternate locations are available, the one with the most favorable intersection conditions should be selected.

Mid-block crossings should be located far enough from the intersection to remain outside of the vehicular traffic mix approaching and entering an intersection. It may be preferable to provide a mid-block bicycle path crossing at a minimum distance of 100 ft [30 m] from the vehicular intersection. There are other elements that need to be considered in this type of crossing, including right of way assignment, appropriate traffic control devices, sight distance, refuge islands and pavement markings. The preferred intersecting angle for this type of crossing is 90^0.

Adjacent path intersections occur when the path is parallel to a roadway and it crosses a driveway or other intersecting roadway such as a T-intersection or a simple four-legged intersection. In designing this type of crossing it is important to keep the location close to the intersection. This allows the motorist and path user to recognize they are a part of the traffic mix and to be prepared to react accordingly. In this situation, the user is faced with multiple conflicts.

The major roadway may be the parallel route or the intersecting roadway. It is necessary to clearly definite rights of way and the appropriate traffic control devices adjusted to reflect the addition of the shared path component in the intersection flow. Considerable traffic engineering must be utilized to make this type of crossing safe.

Complex intersections are defined as all other types of intersections of paths with roadways. These intersections are unique and must be designed as site specific. Several alternative treatments are available such as moving the crossing, installing a signal, changing signalization timing, or providing a refuge island.

If these or other safe solutions are not possible, it may be preferable to have the cyclist dismount and walk the bicycle across the intersection. Designers should insure that adequate signing is in place to alert both vehicles and bicycle users of their design intent.

In most cases, the cost of grade separating the shared use path from the highway will be cost prohibitive. However, for crossings of freeways and other high speed, high volume arterials, a grade separation structure may be the only possible or practical treatment.

When intersections occur at grade, a major consideration is the establishment of right-of-way. The type of traffic control to be used (signal, stop sign, yield sign, etc.), and location, should be provided in accordance with the MUTCD.

Sign type, size and location should also be in accordance with the MUTCD. Care should be taken to ensure that shared use path signs are located so that motorists are not confused by them and that roadway signs are placed so that shared use path users are not confused by them.

It is preferable that the crossing of a shared path and a roadway be at a location away from the influence of intersections with other highways. Controlling vehicle movements at such intersections is more easily and safely accomplished through the application of standard traffic control devices and normal rules of the road. Where physical constraints prohibit such independent intersections, the crossings may be at or adjacent to the pedestrian crossing. Who yields the right of way should be assigned and sight distance provided so as to minimize the potential for conflict resulting from unconventional turning movements. At crossings of high-volume, multi-lane arterial highways where signals are not warranted, consideration should be given to providing a median refuge area for the shared path user. Where shared use paths intersect at highway intersections it may be preferable for the bicyclist to dismount and walk the bicycle through the intersection using pedestrian crosswalks. Adequate signing must be provided to alert the bicyclist of this condition.

When shared use paths terminate at existing roads, it is important to integrate the path into the existing system of roadways. Care should be taken to properly design the terminals to transition the traffic into a safe merging or diverging situation. Appropriate signing is necessary to warn and direct both bicyclists and motorists regarding these transition areas.

Bicycle path intersections and approaches should be on relatively flat grades. Stopping sight distances at intersections should be checked and adequate warning should be given to permit bicyclists to stop before reaching the intersection, especially on downgrades.

Curb ramps at intersections should be the same width as the shared use path. Curb ramps should provide a smooth transition between the shared use path and the roadway.

10.9.6.8 RESTRICTION OF MOTOR VEHICLE TRAFFIC

Shared use paths need some form of physical barrier at highway intersections to prevent unauthorized motor vehicles from using the facilities. Provisions can be made for a lockable, removable post to permit entrance by authorized vehicles. The post should be permanently reflectorized for nighttime visibility and painted a bright color for improved daytime visibility. When more than one post is used, 5 ft [1.5 m] spacing is desirable. Wider spacing can allow entry to motor vehicles, while narrower spacing might prevent entry by adult tricycles and bicycles with trailers.

An alternative method of restricting entry of motor vehicles is to split the entryway into two 5 ft [1.5 m] sections separated by low landscaping. Emergency vehicles can still enter if necessary by straddling the landscaping. The higher maintenance costs associated with landscaping should be considered.

10.9.6.9 OTHER DESIGN ISSUES

The preferred pavement surface is a good quality all weather surface. Designing the pavement structure is similar to that of a roadway. Design is based on soil investigation to determine the load carrying capacity of the existing soils. In this case, the controlling load is that of motorized maintenance and patrol vehicles. The pavement selection is influenced by frost damage potential, skid resistance, surface quality, edge support, and surface and subsurface drainage.

Motor vehicle grade separations pose a design problem for the shared use path. The first consideration is whether or not safe continuity can be provided, particularly on existing structures. A clear width equal to the approach width of the shared use path plus a lateral offset of 2 ft [0.6 m] is desirable. Vertical clearances are controlled by the motor vehicle traffic. Railing, fences or barriers on both sides of the path across the structures should be a minimum of 44 in [1.1 m] high. Expansion joints should be "bicycle friendly".

Drainage design for shared paths is similar to that of a roadway. A cross slope of 2 percent in one direction with no crown is preferred and also simplifies the construction. Side ditches, ground cover, erosion and all other drainage design elements are a part of the path design.

Designating a sidewalk as a shared use path is not recommended, even if the sidewalk is wider than normal. The introduction of a diversity of users (bicyclists, rollerbladers, etc.) and their particular operational characteristics will cause safety problems for all users. Sidewalks as a shared use path should be limited to a high speed or heavily traveled area or across long narrow structures where continuity of the path is desirable.

10.10 BUS STOPS

The Delaware Transit Corporation (DTC) establishes policy and design guidelines for bus stops and other transit related facilities. As part of the project development process highways and corridors served by transit will be identified and appropriate facilities included in the project. The following is a general discussion on bus stops. For specifics, the designer should refer to the Delaware Transit Corporation's Policy: *Bus Stop and Passenger Facilities Standards."*

10.10.1 LOCATION CRITERIA

Bus stops are generally located at or near major trip generators, destinations or at regular intervals based on the population density and other related demographic transit related criteria. Stops are located where passengers can board and alight safely and where buses can safely enter and exit the traffic flow. Sidewalks and walkways serving bus stops should separate

pedestrians from vehicular traffic. Whenever, possible, stops in opposite directions on a route should be located directly opposite each other.

Bus stops should not obstruct driveways or entranceways nor cause visual obstruction for motorists or the bus driver merging back into traffic. In areas that have high traffic volumes, significant turning movements and pedestrian crossings, the stop should be placed where it will present the least conflict.

10.10.2 BUS STOP DESIGN

The three basic configurations for bus stops are (see Figure 10-10):

- Far-side (placed immediately after an intersection),
- Near-side (placed immediately before an intersection), and
- Mid-block (placed between intersections or along the side of a stretch of roadway).

All three of these configurations have advantages and disadvantages. Of the three, far-side curb stops, those placed at the far side of the intersection, are preferred in high traffic areas because they allow for better bus operation and traffic and pedestrian flow. After determining the stop configuration, the bus stop zone design type must be selected. Figures 10-11 through 10-17 show the layout and design parameters for the five types of stop zones. These are:

- Curbside—those placed in parking or running lanes adjacent to the curb.
- Bus Bay—a separate lane outside the influence of an intersection, with appropriately designed deceleration and acceleration lanes.
- Open Bus Bay—a separate lane on the far side of an intersection with an acceleration lane. The intersection area is used as the deceleration lane.
- Queue Jumper Bus Bay—a separate lane on the far side of an intersection where the right turn lane is used by the bus to by-pass the signal queue. An appropriate acceleration lane is provided.
- Nub—In a location with on-street parking, the curb is extended toward the travel lane, removing available parking, creating an area large enough to permit the bus to stop at curb side, in the running lane.

Bus bay, open bus bay and queue jumper bus bay designs provide a protected area away from moving traffic and minimize delay to through traffic. However, they present problems for the bus to reenter the traffic flow, are expensive to install, result in the removal of considerable parking spaces and, perhaps the greatest disadvantage, are difficult and expensive to move. The nub type has similar advantages to the curbside type. However, in most situations the disadvantages due to the operational area required, cost to install and loss of parking outweigh the advantages.

The preferred type of stop is the curb-side stop which provides easy access for the bus, results in minimal delay, is simple to design, easy and inexpensive to install and easy to relocate. The disadvantages are, when placed in the running lane, that they cause traffic to back up and may lead drivers to unsafe maneuvers to avoid the stopped bus. With advanced signing and driver expectation these disadvantages are minimized. In areas with permitted on-street parking, removal of parking spaces within the stop zone is required but not to the limits required by the other types of bus stop zones.

Curbside bus stop zones are the portions of the roadway marked and signed for use by buses. Bus stop zones should be a minimum of 90 ft [27 m] for far-side stops, 100 ft [30 m] for near side stops, and a minimum of 150 ft [45 m] for mid-block stops. Bus stop zones lengths may have to be extended depending on the rate of bus

arrivals and passenger service time at the stop. Bus bays, also known as turnouts, are specially constructed areas separated from the travel lanes and off the normal roadway section. The design allows through traffic to flow freely when encountering a stopped bus. Bus bays are usually constructed on high-volume or high-speed roadways or heavily congested downtown shopping areas with large rider usage. On high volume roadways bus bays must be designed properly or they may be avoided by the bus driver due to the extreme difficulty maneuvering and lost time required in returning to the traffic flow.

Bus bay design is based on criteria that considers the established roadway speed, the entering bus speed, the required deceleration acceleration and tapers to allow for entrance and exit of the bus, plus the required stopping area allowed for boarding and alighting. The higher the traffic speeds the greater all these dimensions plus the lane width become. In urban areas far-side intersection placement of bus bays is preferred.

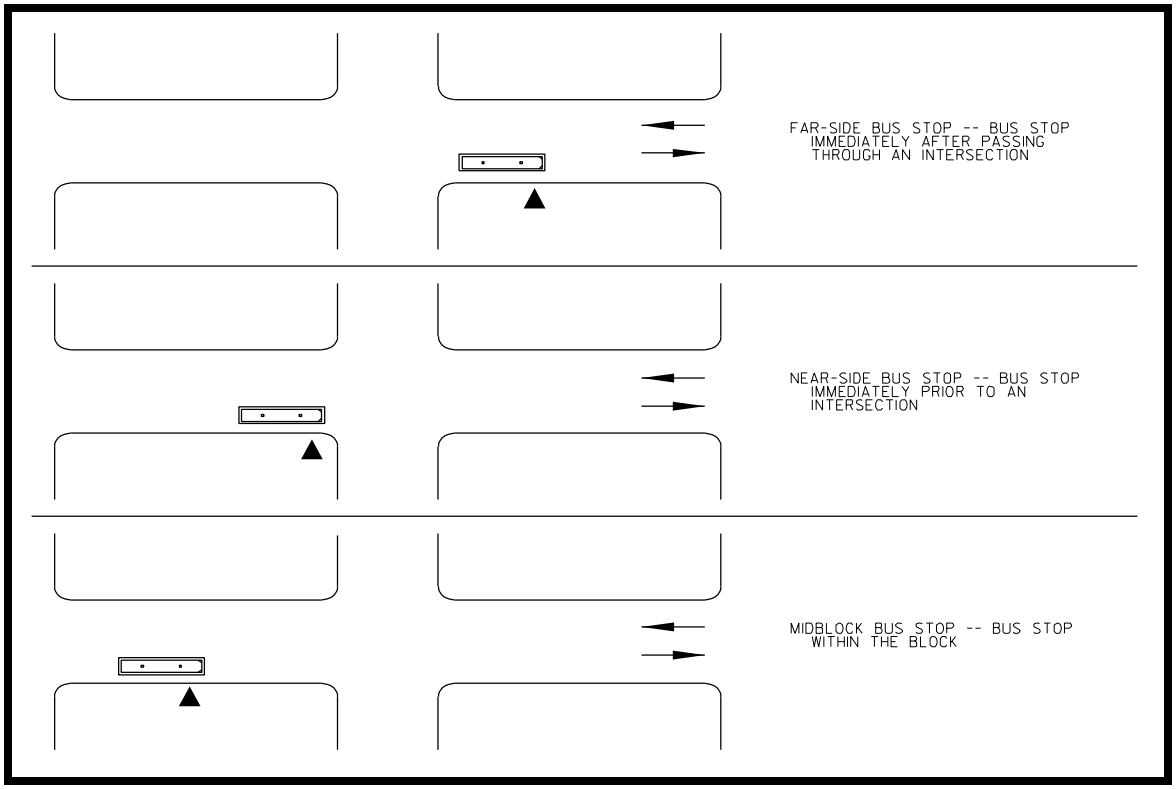
The open bus bay design is a variation of the bus bay design with the entrance end open to the upstream intersection. The bus driver has the pavement width of the upstream cross street available to decelerate and to move into the bay. The advantages of this design are that it allows efficient bus movement and permits free flow of traffic. The disadvantage is to the crossing pedestrians who have a greater distance to cross the intersection and encounter another potential conflict, particularly if the impatient vehicle driver decides to use this lane to avoid the queue. To minimize the disadvantages, a partial-open bus bay design may be considered. This alternative maintains the roadway curb line at the far side of the intersection creating a bubble that allows the buses to use the intersection approach in entering the bay and provides a partial sidewalk extension to reduce pedestrian crossing distance.

Queue jumper bus bays provide priority treatment for buses along arterial streets by allowing buses to bypass the queue at intersections. These bus stops consist of a near-side and far-side open bus bay. Buses are allowed to use the right turn lane to bypass congestion and proceed through the intersection. The right turn lane is appropriately signed to allow for this maneuver. The benefits of queue jumper stops are that they remove the bus from congested intersections and installation costs are affordable. When right turn lane volumes becomes too high an exclusive bus lane may be needed.

Nubs are sections of sidewalk that taper from the curbed parking lane to the edge of the travel lane creating an area prior to and around the intersection radius. The bus operates similar to curbside stops. Nubs are used on low speed and/or low volume streets where right turn lanes are not warranted. They are considered at locations of high pedestrian activity, crowded sidewalks, where pedestrian crossing distances need to be decreased, and where stopped buses in the travel lane can be tolerated. Nubs can be used both as bus stops and traffic calming measures.

Once a bus stop configuration is selected a design vehicle representative of those in the fleet must be determined. The most commonly used bus is the standard 40 ft [12 m] bus. Articulated 60 ft [18 m] buses may be a part of the fleet. Normally the dimensions typical for a standard 40 ft [12 m] bus are used for design. The design features that are affected by the design vehicle decision are lane and shoulder widths, lateral and vertical clearances, vehicle storage lengths, minimum turning radii, and the pavement strength.

Figure 10-10
Bus Stop Placement



Safe pedestrian access to the bus stop is important. Placing stops where there is an existing sidewalk network meeting all the criteria of the Americans with Disabilities Act to accommodate the disabled and physically challenged is the desired location. The accessible path to the stop should be well drained and, where possible, placed in street lighted areas.

The pathway to the bus stop should have the following characteristics:

- Clear width of at least 3 ft [0.9 m], preferably 5 ft [1.5 m] (the minimum width needed to allow passage of two wheelchairs)
- Running slope of pathway can be no greater than 8%
- The surface of the pathway must be firm and well-drained
- The path must comply with accessibility guidelines for curb cut at all street intersections.

10.10.2.1 BUS SHELTER SETBACK

Whether or not to install bus shelters is a decision of the Delaware Transit orporation (DTC). Primarily the number of passenger boardings per day guides this decision. However, consideration is also given to the number of transfers at a stop, the volume and frequency of transit service, the number of disabled and elderly users, the proximity to major activity areas and available space. More detail is available in DTC policy *Bus Stops and Passenger Facilities Standards.*

When DTC determines that a bus shelter is to be included in a project, then the clear zone principles found in the *Roadside Design Guide* are to be applied. Unless designed as a breakaway unit, the bus shelter is considered a fixed object. The bus shelter

location than becomes a function of design speed, traffic volumes, cross slopes and roadway curvature. The following guidance is to be used in locating a bus stop:

- Projects with a design speed of 60 mph [100 km/h] or higher should have any bus shelters located off-site.
- If the design speed is 55 mph [90 km/h], a bus shelter should be off-site or outside the clear zone without curbing.
- If the design speed is 50 mph [80 km/h], or less (such as approaching or in an urban area), a bus shelter should (a) be outside the clear zone without curbing, (b) be setback a minimum of 5 ft [1.5 m] from a barrier curb or (c) be setback a minimum of 8 ft [2.4 m] from the face of curb if the curb is depressed for the disadvantaged.
- Off-site shelters located in park-and-ride lots or transit hubs should conform to the requirements set for sites in areas with design speeds of 50 mph [80 km/h], or less.
- If a barrier curb is used, it should be extended upstream from the shelter a distance as required by Section 5.6.4 of the Roadside Design Guide. This distance is generally between 110 ft [33 m] and 210 ft [63 m] depending on the traffic volume, design speed and shelter setback.
- If the setback can not be met, a design exception with necessary documentation and justification should be processed.

10.11 PARK-AND-RIDE LOTS

DTC is responsible for establishing and maintaining park-and-ride lots. DTC is involved in the project development process to ensure site feasibility and conformance with intended use. This section is a general discussion on park-and ride facilities. The designer should refer to Delaware Transit Corporation's Policy: *Bus Stops and Passenger Facilities Standards*.

Park-and-ride lots are intended to provide a common location for individuals to transfer from low- to high-occupancy travel modes. The overall objective is to maximize the efficiency of the transportation system and to provide commuter options. In addition to serving locations to transfer automobile users to a transit facility, they are created at selected locations to encourage the formation of carpools and vanpools.

In providing this travel alternative, there are five types of park-and ride facilities; (1) remote, (2) local service, (3) peripheral, (4) exclusive use, and (5) shared-use. Remote park-and-ride lots are located relatively far from a major activity center and/or the final destination of users. Remote lots serve residents of rural and suburban areas and community centers allowing them to travel to and from central business districts or other high employment centers in a mode other than a single occupancy vehicle (SOV). To be successful they need to intercept automobiles close to the residential area or home. In order to do this, they are usually located relatively close to heavily traveled corridors.

Local service park-and-ride lots are located at the end of a local transit route. These lots are located closer to desired destinations and serve residential neighborhoods at the end of the routes, as well as those along the route. Peripheral park-and-ride lots are located on the edge of a major activity center. These lots function to expand the available parking and to intercept automobiles before entering the congested area. These types of facilities do not eliminate the commute trip. The last part of the trip into the most congested areas is made by transit. Ideally carpools and vanpools form at peripheral lots to use this facility for transfer to transit serving individual destinations.

Exclusive use park-and-ride lots are planned, designed, constructed and operated specifically to serve as park-and-ride lots.

Remote park-and-ride lots are usually exclusive facilities. However, these types of facilities are more generally associated with rail systems, high occupancy vehicle lanes, or express bus services. These lots have a high use rate, provide passenger amenities, and have frequent peak-hour service. Because they are designed exclusively to serve as a park-and-ride, provisions for adequate parking, efficient bus operation, pedestrian circulation and safety are a necessity. In order to accomplish this, these facilities have a high cost.

Shared-use lots serve several functions with only a portion designated as a park-and ride area. They may be located in a shopping center, church, school, or other activity center parking lot. Shared-use lots are usually located along an existing bus route. The advantage of this type of lot is that it is low in cost and can provide a low cost means of determining the demand for this type of service. The disadvantages include the need to negotiate a formal agreement to cover rent, maintenance, repairs, and termination of use. In addition, the lot may not be laid out or easily adapted to serve transit and its associated pedestrian-automobile conflicts, particularly during peak business seasons.

The need for park-and-ride facilities is influenced by many factors. The following characteristics influence utilization:

- A concentration of work trips to a central location, High levels of traffic congestion on destination routes,
- Limited, inconvenient and/or expensive parking at destinations,
- Frequent transit service available from the park-and-ride facility, and
- Time savings due to preferential treatment of high occupancy vehicles.

In most cases, park-and-ride facilities and the supporting transit services are oriented to a major destination, usually the central business district (CBD). However, in some cases service may be oriented to another major activity center, or service may be provided to more than one destination. General guidelines are discussed in this section. Refer to AASHTO's *Guide for the Design of High-Occupancy Vehicle Facilities* and *Guide for the Design of Park-and-Ride Facilities* and FHWA's *Park-and Ride Facilities: Guide for Planning, Design and Operation* for more detailed information.

Figure 10-11

Street-Side Bus Stop Designs

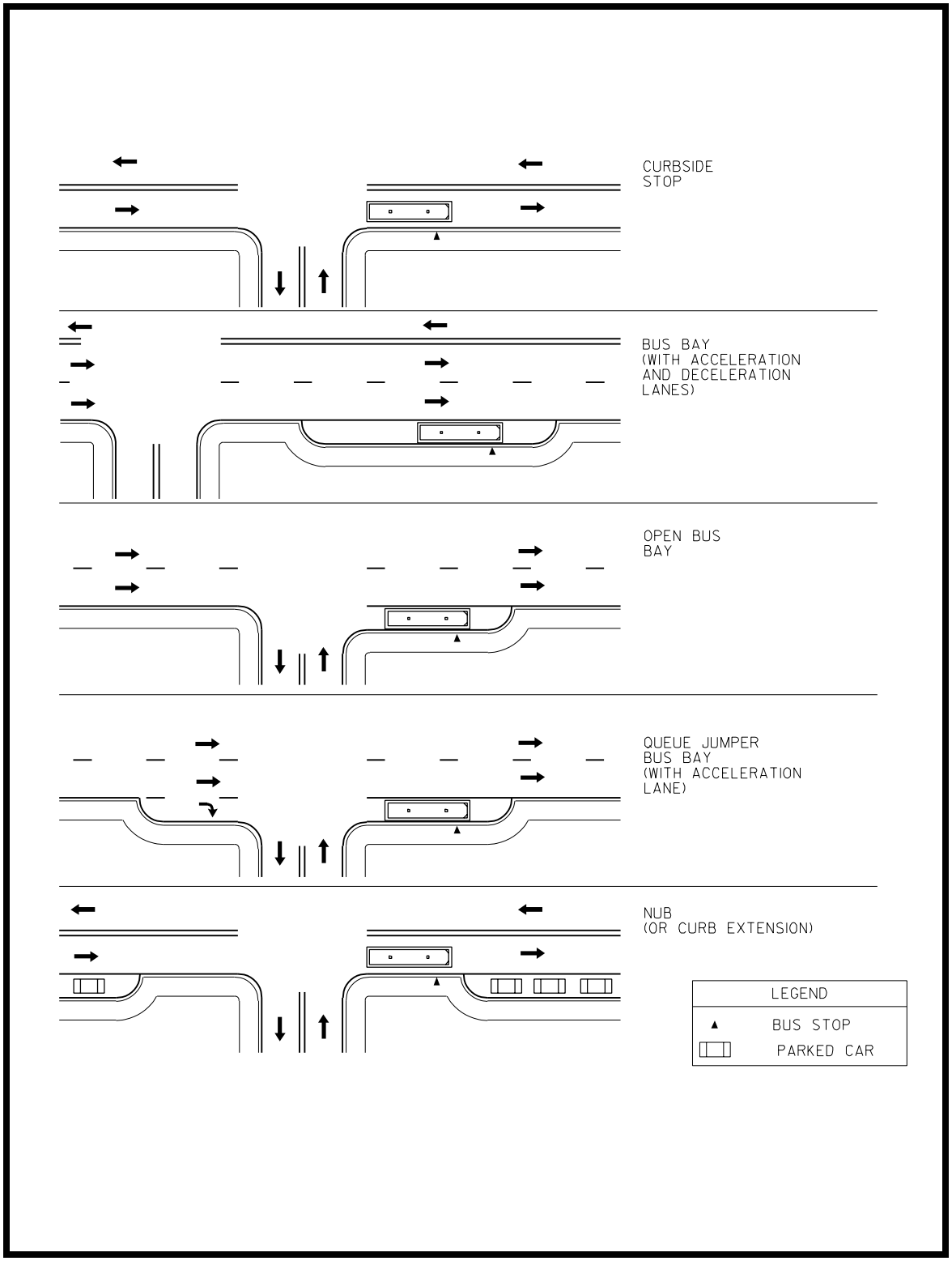
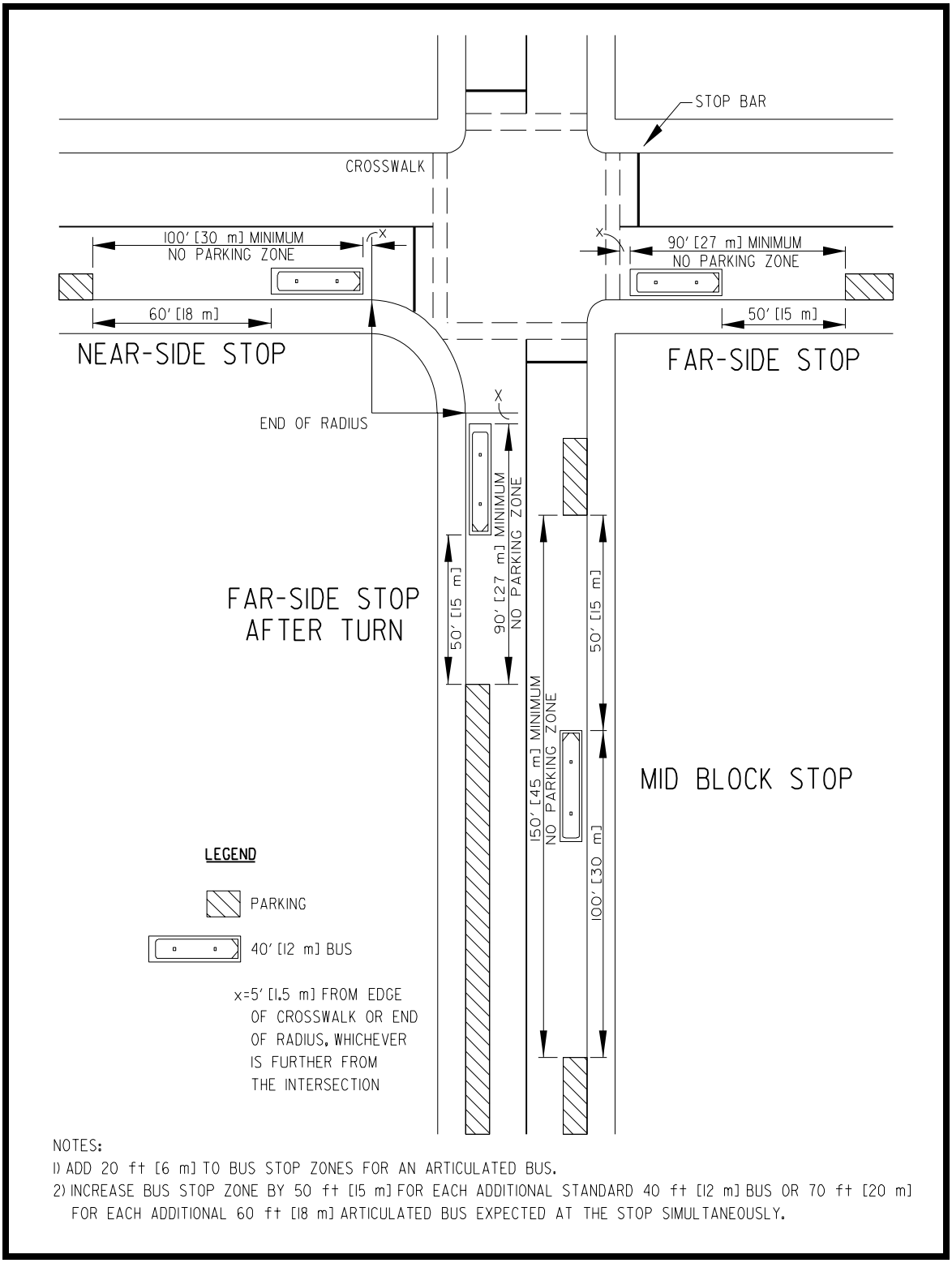


Figure 10-12
Typical Dimensions for On-Street Bus Stops



NOTES:
1) ADD 20 ft [6 m] TO BUS STOP ZONES FOR AN ARTICULATED BUS.
2) INCREASE BUS STOP ZONE BY 50 ft [15 m] FOR EACH ADDITIONAL STANDARD 40 ft [12 m] BUS OR 70 ft [20 m] FOR EACH ADDITIONAL 60 ft [18 m] ARTICULATED BUS EXPECTED AT THE STOP SIMULTANEOUSLY.

Figure 10-13
Typical Bus Bay Layout

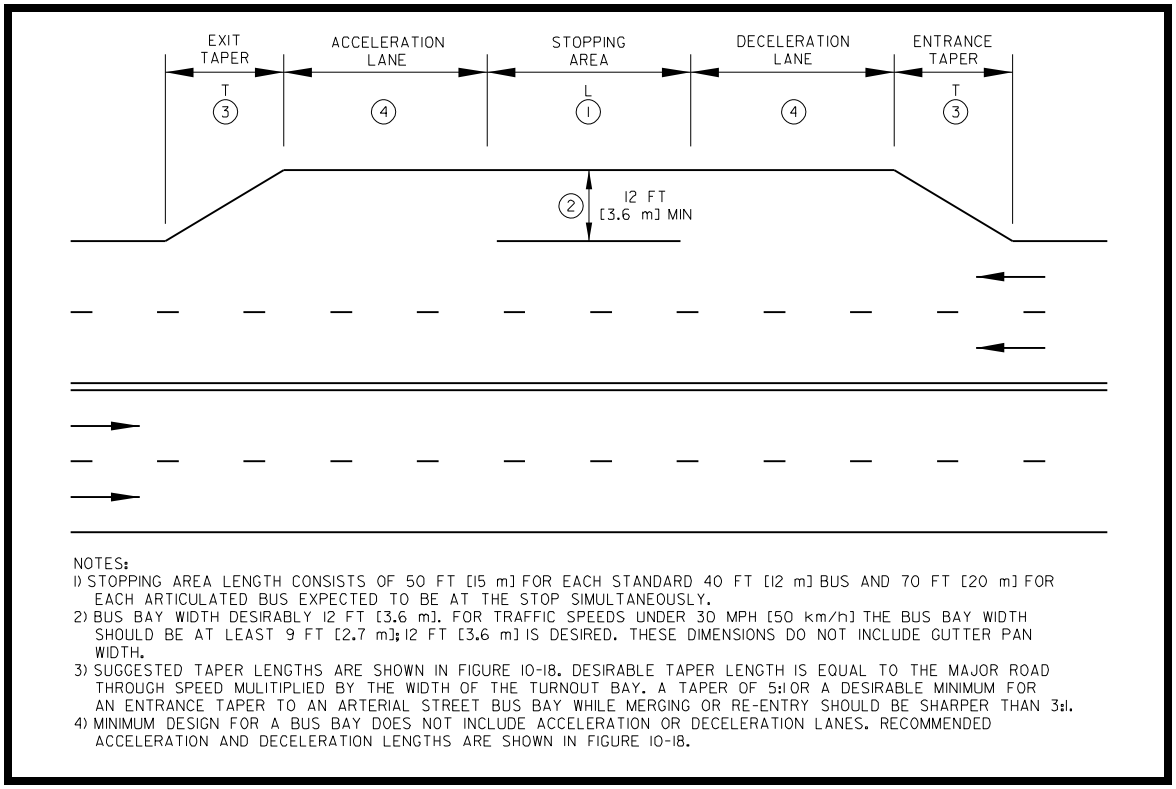


Figure 10-14
Typical Bus Bay Dimensions

Through Speed mph [km/h]	Entering Speed[a] mph [km/h]	Length of Acceleration Lane ft [m]	Length of Deceleration[b] Lane ft [m]	Length of Taper ft [m]
35 [55]	25 [40]	250 [75]	184 [55]	170 [50]
40 [60]	30 [50]	400 [120]	265 [80]	190 [60]
45 [70]	35 [55]	700 [215]	360 [110]	210 [65]
50 [80]	40 [60]	975 [300]	470 [145]	230 [70]
55 [90]	45 [70]	1400 [425]	595 [180]	250 [75]
60 [100]	50 [80]	1900 [580]	735 [225]	270 [80]

[a] Bus speed at end of taper, desirable for buses to be within 10 mph [15km/h] of travel lane vehicle speed at end of the taper.

[b] Based on 2.5 mph/sec [5 km/sec] deceleration rate.

Figure 10-15
Partial Open Bus Bay

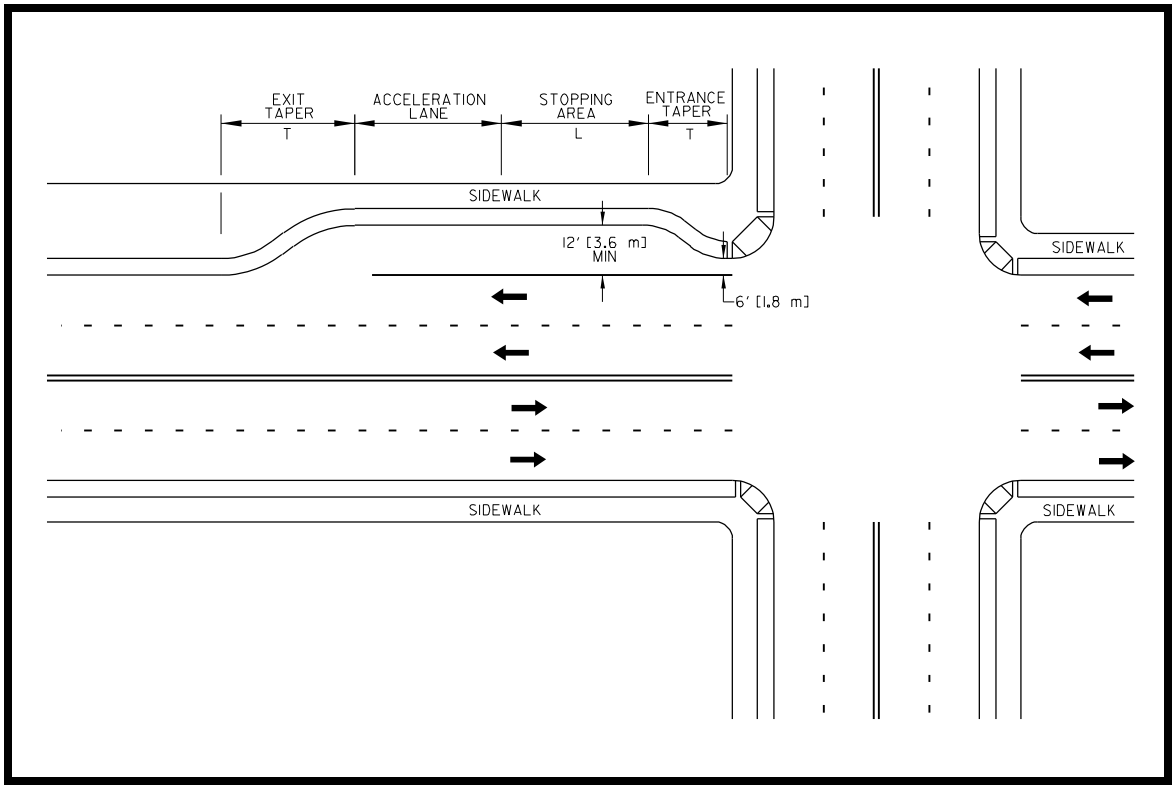
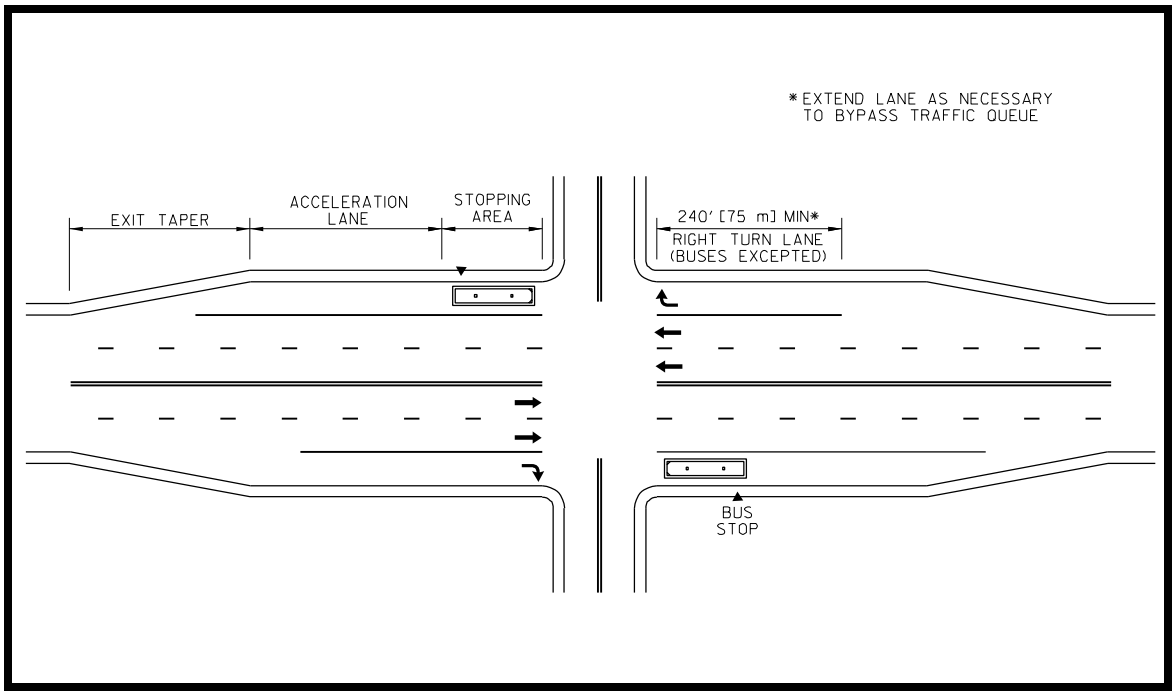
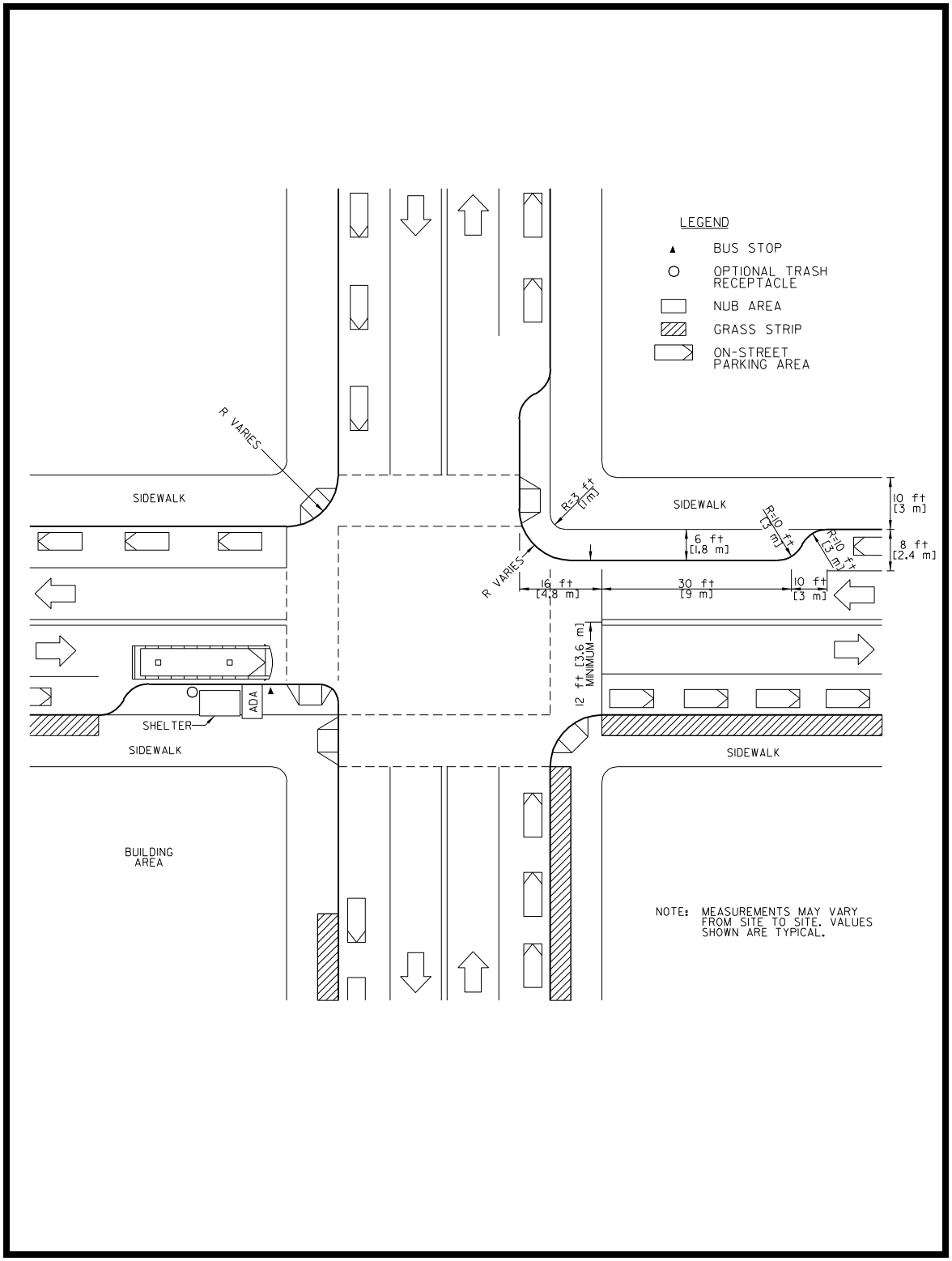


Figure 10-16
Queue Jumper Bus Bay



Notes on Figure 10-13 are applicable to Figures 10-15 and 10-16.

Figure 10-17
Nub Bus Bay



10.11.1 LOCATION

Lot location is a very critical element in the success of a park-and-ride lot. DTC, the project management team, the public, local agencies and communities are involved in this decision. The general rules for locating park-and-ride facilities are (1) locate them where there is easy access to and egress from the lot for both transit and automobiles and (2) locate them where they will be used e.g. in the desired direction of and along the route of the users. Other factors considered are the geographical location, the site conditions, the anticipated site improvement costs, and the potential user costs in out of pocket cost and personnel time.

Geographical considerations are:

- Is it within a densely used transportation corridor and close to the major roadway(s)?
- Is it located with good site visibility from the major roadway(s)?
- Is the distance to activity centers great enough to encourage its use?
- Is it conveniently accessible—on the desired travel path with easy ingress and egress?
- Does it fit into local traffic patterns?
- Does it fit acceptable commuter driving patterns?
- Does it or can it be tied into bicycle routes?
- Is it located prior to congested roadways with easy access?
- Is the site near or can it be served by a transit service?

Site considerations have varied factors such as:

- Impact on the environment and local communities i.e. is it perceived as a community asset?
- Is the site large enough to serve the projected usage?
- If successful, is there room for expansion?
- Does the adjacent area and streets have parking that is more convenient than a park-and-ride site?
- Is or can the site be made secure?

Several cost variables must be considered including:

- Land purchase value.
- Ease of acquisition, a willing seller.
- Site preparation, construction, and on-going maintenance.
- User costs—estimated vehicle operating cost to drive to the site, parking fees, and transit fares.
- User time—time from commuter's home to the site, waiting time for transfer to high-occupancy mode, and walking time to desired activity center.

One of the simplest methods of site evaluation is to observe in the field and inventory sites where people park in groups, usually at intersections or vacant lots. Addressing this need by providing better facilities at or near these areas may increase ride sharing.

Park-and-ride lots and amenities for transfer of passengers to transit routes are coordinated by DTC.

10.11.2 DESIGN

The designer must consider the use to be made of the facility. The design of a park-and-ride lot for car and vanpoolers can be quite simple, a convenient, graveled or paved parking area with safe access to an arterial route (via collector or local roads).

Design considerations are more complex when the lot must serve many users of

various modes including short and long-term commuters, buses, pedestrians, bicyclists and motorcyclists, and maybe even a kiss-and-ride option. The design of the lot should provide for each mode. The primary concern during the design phase is safe, efficient transfer and traffic flow for all potential users and transportation modes. Where possible, the differing arrival modes should be allowed separate access points. In addition, the site should have provisions for adequate numbers of useable parking spaces, facilities that are comfortable, safe, and attractive and accessibility for elderly and disabled patrons.

Parking provided in a central business district is for the convenience of both employees and customers. This creates competition for the use of park-and-rides. Park-and-rides are a user's choice, making them dependent upon perceived benefits to the user either in time and/or cost savings. This requires the designer to address the user's special characteristics and provide cost-effective designs that are safe, convenient, and reflective of other values considered by the patrons in making parking decisions.

10.11.3 ACCESS

Access to the site is an important design feature requiring internal and external traffic analysis and selection of adequate design criteria. The goals are easy maneuverability for the categories of vehicles allowed on the lot, minimizing potential traffic impacts on local roadways and/or major streets, and providing safe operations both on and off the lot.

Access to the lot should not increase congestion on the major highway that it serves nor add a major conflict point to the route. For this reason, direct access by private automobiles from the lot to a major arterial route is usually not provided. However, direct access for buses is often desirable. Usually, the most efficient access point to a park-and-ride lot will be on a collector or local street intersecting the arterial.

Locating the facility on the right side for inbound traffic on a two-way roadway will allow most users to turn right into the lot, thus eliminating the hazard of crossing an opposing traffic stream. It is likely that maximizing the accessibility for inbound trips will be more effective in attracting users than improving the flow for exiting outbound traffic. Figure 10-18 shows four typical entrance and exit configurations. It is desirable to provide separate one-way entrance and exit drives to minimize crossing conflicts. Note that the access points for the lot should be located at least 150 ft [45 m] from an intersection with the cross-street and 150 ft [45 m] between access points. When there are no more than 150 parking-stalls these distances may be reduced to 100 ft [30 m]. As parking stall capacity increases above 300 stalls, all the external and internal design parameters, circulation patterns and recognition of differing modes become more critical and must be carefully evaluated.

10.11.4 INTERNAL CIRCULATION

Major circulation routes in the lot should be located at the periphery of the parking area to minimize pedestrian-vehicle conflicts. For design, the priority sequence is feeder buses, taxis, kiss-and-ride, then park-and-ride with emphasis on the safety needs of bicyclists and pedestrians. The control vehicle in design is the bus. The ingress, egress and internal layout are most influenced by the bus's turning radius and size. Lots designed to serve cars only will probably not accommodate buses and will be avoided by the drivers. Wherever possible, buses and cars should not be mixed. Where transit service is available, circulation routes should be designed for easy bus movement, efficient terminal operation and convenient passenger transfer. A one-way roadway pattern is desirable with

two lanes provided in the bus stop area for passing the stopped bus.

When passenger-waiting areas are needed, they should be located in a central area with the various user modes surrounding the waiting area or at one end of the lot and parking for the various modes extending radially from the waiting area. Internal circulation design should give priority to fast and easy ingress and egress for transit buses, paratransit vehicles and kiss-and-ride vehicles. In shared use facilities, such as shopping centers or churches, the waiting and designated parking areas should located away from main buildings as to not interfere with normal activities.

Some of the general considerations for design of internal circulation are:

- Drivers should not be confronted with multiple choices at the same time;
- Access points should be spaced to provide for maneuvering and minimize conflicts;
- Access points should be designed to meet demand capacity;
- Flexibility to adjust to changes in operation should be available;
- The terminal/waiting area for high use lots should be located off-street but have convenient access to and from the major roadway;
- When transit services are provided and the lot also serves car/van pool formation, the car/van parking should be located in an area removed from the transit operation;
- HOV access and circulation should be separated from car access and circulation whenever possible; and
- Simple, clear signing.

10.11.5 BUSES

As previously discussed, where buses load and unload within the parking lot, traffic flow should be such that buses and automobiles do not conflict. Buses require adequate room for decelerating, proper turning radii, maneuvering into and out of an adequate loading area, and returning to the mainline traffic flow. Refer to Chapter 7 of this manual and the Green Book for design criteria such as required turning radii for bus operation.

10.11.6 KISS-AND-RIDE FACILITIES

A kiss-and-ride facility is located so that transit or commuter passengers can easily and safely access the terminal or loading zone with minimum conflicts with other vehicles; see Figure 10-19. To accomplish this, circulation in the kiss-and-ride facility should be one-way and flank the terminal or loading area. Parking should be at 45 degrees to allow for pull through and face the terminal or loading zone. To operate properly it is usually necessary to enforce kiss-and-ride restrictions.

10.11.7 PEDESTRIANS

Two pedestrian movements must be provided for park-and-ride lots that serve bus routes: a direct and safe approach from adjacent streets to the bus stop and pedestrian access from the parking area. Pedestrian circulation in parking lots is provided by aisles and crosswalks or, in larger lots, by walkways. The pedestrian path from any parking stall to the bus stop should be as direct as possible.

10.11.8 BICYCLES AND MOTORCYCLES

It is important to provide adequate bicycle storage racks at park-and-ride lots where large concentrations of bicycle traffic are expected. Similarly, a special parking area for motorcycles will improve utilization of

space. Motorcycle storage should be on Portland cement concrete to prevent stands from sinking into hot asphalt pavement. Provisions for locking both bicycles and motorcycles to prevent theft are needed. This includes bicycle racks and lockers.

10.11.9 DISABLED

At lots for transfer to buses, the design should consider provisions for safe and convenient access for the elderly and disabled. Design requirements and provisions for disabled parking shall be in conformance with the *Americans with Disabilities Act of 1990* and the State of Delaware *Architectural Accessibility Standards.*

Facilities for the disabled should also be designed in accordance with the following:

- Disabled persons should reach the bus loading zone without crossing any access roads;
- Loading areas must meet ADA space requirements;
- Disabled persons must never be forced to travel behind parked cars; and
- Suitable ramps must be provided.

10.11.10 PARKING DIMENSIONS AND LOT LAYOUT

Parking areas and roadway layout for park-and-rides can be designed in much the same manner as other parking facilities. Facilities that interact with transit, where DTC does not specify a bus size, should use a 40 foot [12 m] transit vehicle as the design vehicle. Standard dimensions for car parking stalls are shown in Figure 10-20.

For design purposes only two size stalls should be considered—standard and intermediate. If compact car parking is to be provided, it should be in a prime location or the driver will select more convenient available stalls. Combining several different types of stalls also creates an undesirable and more complicated signing layout. If there is adequate room, limiting the design to accommodate the standard stall size, 9 by 18.5 ft [2.7 by 5.6 m] is preferred. The minimum bus-parking stall should be 13 by 32 ft [4 by 15.25 m].

Figure 10-18
Typical Car Parking Dimensions

Size	Stall Width ft [m]	Stall Length ft [m]	Aisle Width ft [m]
Standard	8.5-9.5 [2.6-2.9]	18-20 [5.5-6.0]	24-26 [7.5-8.0]
Inter-mediate	8.0-9.0 [2.4-2.7]	16-18 [4.9-5.5]	22-24 [6.7-7.3]
Compact	7.5-8.5 [2.3-2.6]	15-17 [4.6-5.2]	20-22 [6.0-6.7]

Substandard stall and aisle widths are false economy. Although they permit marking more stalls in a given area, vehicles tend to encroach upon adjacent stalls such that one or more spaces are unavailable for use. The end result is no gain in actual space usage.

Vehicles and other objects should be excluded from corners or parking spots where it is necessary to provide adequate intersection sight distances. Islands at the end of rows should be considered when laying out the lot. For pedestrian safety, the parking stalls and aisles should be parallel to the direction of the desired pedestrian flow. For efficient land area use, a row of parking on each side of the aisle is preferred.

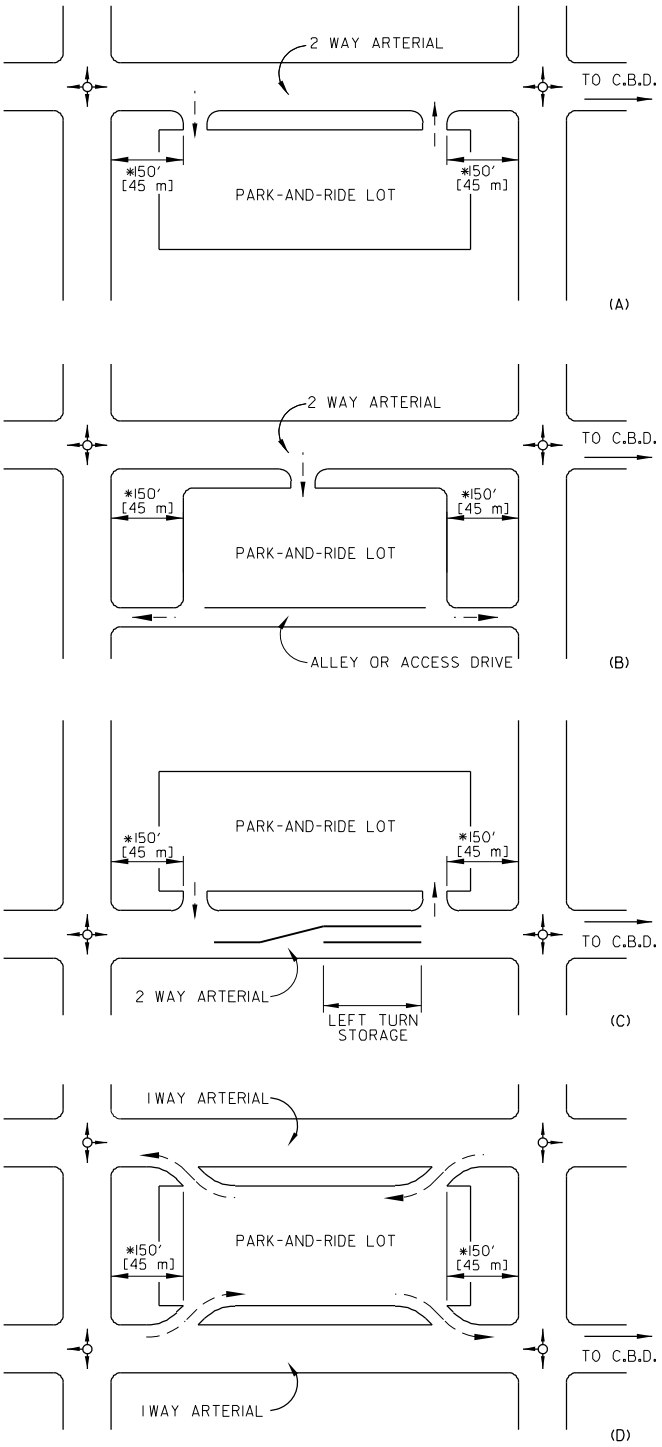
Aisle width is a function of the parking angle and stall width. One-way aisles are generally used with angle parking and two-way circulation is generally used with 90-degree parking. Aisle lengths should not exceed 400 ft [120 m] to limit pedestrian walking distance. One-way aisles should favor counterclockwise circulation with head-in parking only. Due to lower vehicle undercarriage heights, a 6 in [150 mm] curb is recommended where head-in parking is being considered. Sidewalks should be a

minimum of 5 ft [1.5 m] and loading areas should be 12 ft [3.6 m] wide. Pedestrian paths from parking spaces to loading areas should be as direct as possible. All sidewalks and curb areas are to be in conformance with ADA standards.

Figures 10-21 and 10-22 provide data for planning stall layouts for standard stall sizes of 9 by 18.5 ft [2.0 by 5.6 m]. Layouts for intermediate and compact stall sizes are available in the AASHTO guide.

The parking area should be sloped to provide positive drainage. Ponding water in a lot is undesirable for both vehicle and pedestrian movement, particularly where freezing may create icy spots. The recommended minimum grade is 1%, the desirable is 2%, and the maximum is 5%. The designer should provide adequate access and areas for snow removal and/or storage. The pavement selection needs to recognize that a variety of traffic loads, particularly when transit is expected, may be applied to the lot and the pavement type and strength designed accordingly.

Figure 10-19
Park-and-Ride Access Configuration



*MINIMUM, 350′ [107 m] DESIRABLE
C.B.D.=CENTRAL BUSINESS DISTRICT

Figure 10-20
Example of Kiss-and-Ride Parking Lot

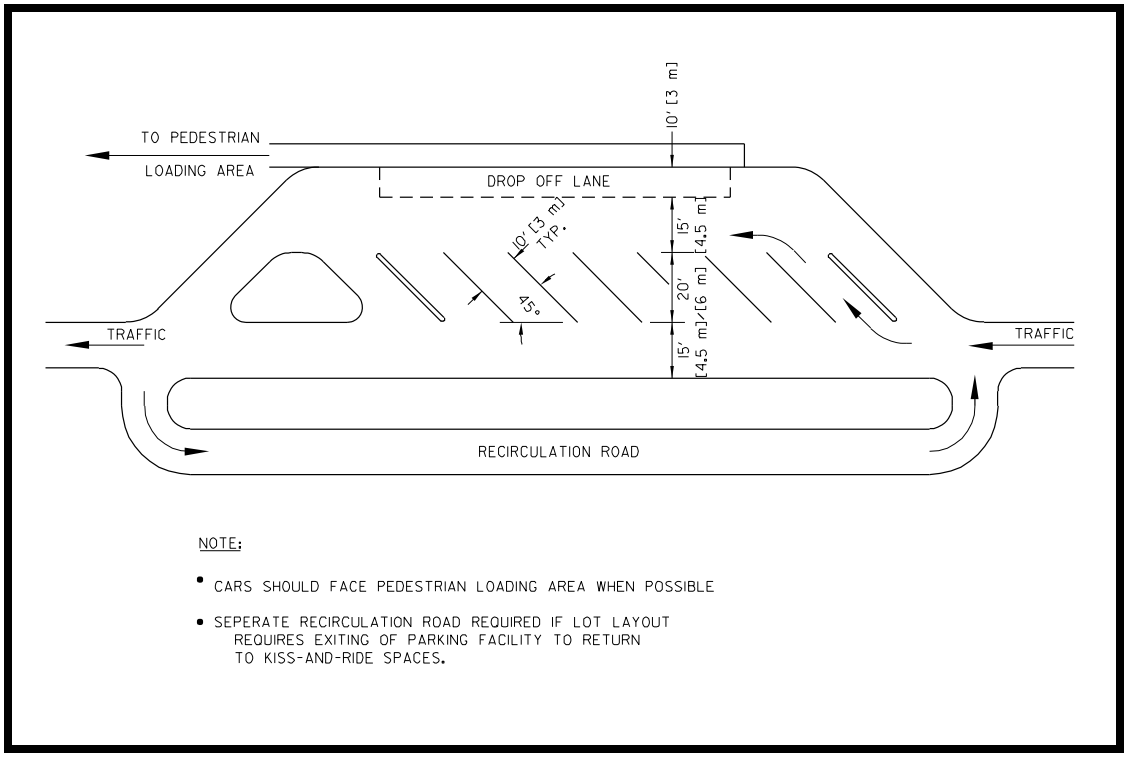


Figure 10-21
Stall Layout for Standard Vehicle (Car)

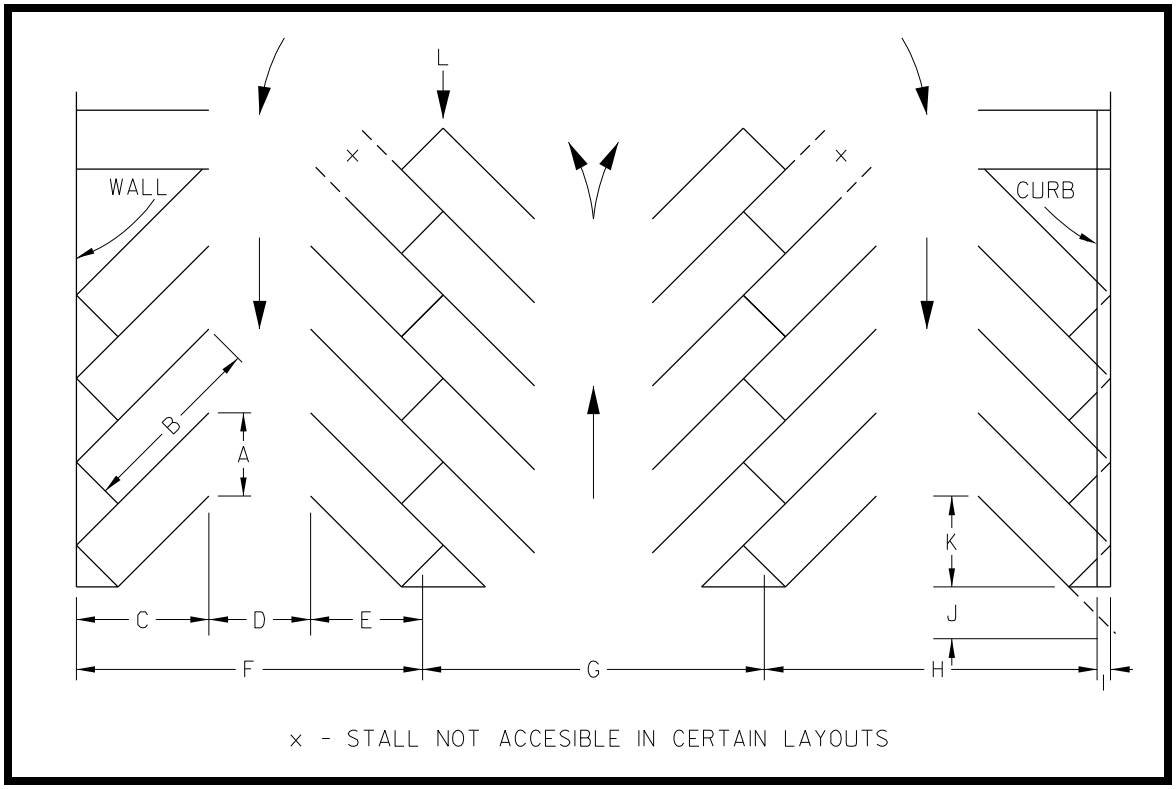


Figure 10-22
Stall Dimensions for Standard Vehicle (Car)

Dimension ft [m]	As shown in Figure 10-25	Angle			
		45^0	60^0	75^0	90^0
Stall width, parallel to aisle	A	12.7 [3.9]	10.4 [3.2]	9.3 [2.8]	9.0 [2.7]
Stall length of line	B	25.0 [7.6]	22.0 [6.7]	20.0 [6.1]	18.5 [5.6]
Stall length of line	C	17.5 [5.3]	19.0 [5.3]	19.5 [5.9]	18.5 [5.6]
Aisle width between stall line	D	12.0 [3.6]	16.0 [4.9]	23.0 [7.0]	26.0 [7.9]
Stall depth, interlock	E	15.3 [4.7]	17.5 [5.8]	18.8 [5.7]	18.5 [5.6]
Module, wall to interlock	F	44.8 [13.7]	52.5 [16.0]	61.3 [18.7]	63.0 [19.2]
Module, interlocking	G	42.6 [13]	51.0 [15.5]	61.0 [18.6]	63.0 [19.2]
Module, interlock to curb face	H	42.8 [13.0]	50.2 [15.3]	58.8 [18.0]	60.5 [18.4]
Bumper overhang (typical)	I	2.0 [0.6]	2.3 [0.7]	2.5 [0.8]	2.5 [0.8]
Offset	J	6.3 [1.9]	2.7 (0.8]	0.5 [0.2]	0.0 [0.0]
Setback	K	11.0 [3.3]	8.3 [2.5]	5.0 [1.5]	0.0 [0.0]
Cross aisle, one-way	L	14.0 [4.3]	14.0 [4.3]	14.0 [4.3]	14.0 [4.3]
Cross aisle, two-way	—	24.0 [7.3]	24.0 [7.3]	24.0 [7.3]	24.0 (7.3)

slope for adequate cross drainage than is required for gravel or a coarse-textured, intermediate-type asphalt surface. The Department has adopted the cross-slope values shown in Figure 4-1 for standard practice on tangent sections of highways.

The range of cross slope values for projects other than "new construction" permits slight variations in the slope where the scope of work is primarily resurfacing. If surface drainage is a problem, cross slopes up to 2.5% on high-type pavements may be justified. However, new construction slopes should be provided wherever practical. For two-lane highways or multi-lane undivided highways, the cross slope normally goes downward both ways from a crown point at the highway center line.

On divided highways, each one-way pavement may be crowned separately, as on two-lane highways, or each may have a one-direction cross slope across the entire width of pavement-usually downward to the outer edge. Surface drainage on roadways with three or more lanes in one direction can cause problems if the pavement slopes uniformly in one direction at the rate of slope recommended for one- and two-lane roadways. The designer has two options for cross slopes on a three-lane roadway: (1) slope the inside lane to the median and the other two lanes to the outside, or (2) slope all three lanes to the outside and increase the slope of the outside lane.

A cross section with each roadway crowned separately, such as the first option above, has an advantage in rapidly draining the pavement during rainstorms. Disadvantages are that more inlet and underground drainage lines are required, and treatment of at-grade intersections is more difficult because of several high and low points on the cross section. Sections having no curbs and a wide depressed median are particularly well suited for this design. With a crowned section, cross slopes should not exceed 2% because the rollover effect, when changing lanes, is then 4%. "Rollover" is the algebraic difference between the two slopes.

Roadways that slope in only one direction are more comfortable to drivers because vehicles tend to be pulled in the same direction when changing lanes. This design is generally desirable for divided highways with a narrow curbed median. The cross slope of the third lane (outside lane) of a three-lane roadway where the cross slope is all one direction should be increased by 0.5% to 1.0% to improve surface drainage.

In the design of urban highways and streets, it may sometimes be found that adjacent property developments dictate that the curb on one side must be higher than the curb on the other. Two options are available. The cross slope can be in one direction for the full width of the street, or the crown point can be offset from the centerline toward the high side of the street. The latter option usually is preferable with the offset crown point corresponding to an edge of travel lane, out of the wheel path and with a maximum of 4% rollover.

Typical cross slope designs are illustrated in Figure 4-2.

4.1.5 SHOULDER CROSS SECTIONS

Shoulders should be flush with the roadway surface and should abut the edge of the traffic lane. All shoulders, including median shoulders on divided highways, normally should be sloped to drain away from the traveled way. However, in the case of a raised narrow median, the median shoulders may slope in the same direction as the traffic lanes, but consideration should be given to sloping the shoulders toward the median and providing inlets and underground drainage to alleviate problems with snow and ice. Slightly sloping shoulders steeper than the traffic lanes assure rapid surface drainage, reduce the chance of ponding, and minimize subgrade penetration of moisture through the edge joint. Paved shoulders normally should slope at a rate of 4%, and unpaved shoulders should be sloped at a rate of 6%.

Special attention must be given to shoulder slopes in relation to superelevation on curves. Shoulder slopes that drain away from traffic lanes on the outside (high side) of a superelevated curve should be designed to avoid too great a cross slope break at the pavement edge. The rollover should not exceed 8%; differences greater than this tend to pull the vehicle toward the shoulder and may result in difficulty for the driver to regain control. For example, with a superelevation rate of 6% and a shoulder slope of 4%, the rollover would equal 10% which is unacceptable. The shoulder slope should be reduced to 2% but not less than 1% along the high side of the curve; this is acceptable since there is no stormwater discharge to the shoulder from the pavement and there is little opportunity for ponding or shoulder erosion damage.

Standard shoulder slopes should be used on the inside (low side) of superelevated curves unless the rate of superelevation exceeds the rate of normal shoulder slope. In this case, the shoulder slope should be the same as the superelevation slope.

4.1.5.1 GRASS SHOULDER

Vehicles often drift off the roadway and cause rutting of the shoulder edge. This is particularly noticeable at intersections and on the inside of sharp curves. Several options of grass shoulders are available. Vehicles may also park on the grass shoulder. Grass shoulders should be used only for special project needs. A grass shoulder shall not be used directly under a bridge overpass. Consider the following:

1. Use inground pavers, geogrid or a combination, which provide stability and allow grass to grow up through them. This is suitable for the area where parking is anticipated. Consider this where trash pick-up or mail delivery could rut the shoulder.
2. Provide a two foot wide paved shoulder adjacent to the grass shoulder, which may or may not be stabilized.
3. Provide a pavement edge line to determine the edge of the travel lane.
4. In a heavily salted area, alkaligrass may be used. Contact the Roadside Environmental Administrator.
5. The time of year for seeding and the length of time after seeding before the shoulder is subject to traffic are important. Phasing work should provide time for growth after seeding, and the shoulder should be protected until the grass is firmly established.

4.1.6 CURBS

Curbs are closely related to other surfacing cross section elements. They generally serve several purposes including drainage control, pavement edge delineation, delineation of pedestrian walkways, and control of entrances to roadside development. Curbs are used extensively on various types of urban highways and streets. In the interest of safety, curbs should be omitted on high-speed rural highways when the same objectives can be attained by other acceptable means. Curbs may be considered an obstruction, increase project cost and design effort. When using curbs, positive drainage of paved areas, particularly the traveled way, is necessary. This normally requires the installation of a closed drainage system with drainage inlets, positive outfalls and extensive ditching. Therefore, the need and use of curbs should be given appropriate study.

Curbs may be designed as a separate unit or integrally with the pavement structure. Separate curbs usually are a combination curb and gutter. Sometimes the curb is constructed alone without the gutter section.

The two general classes of curbs are barrier curbs and mountable curbs. Barrier curbs tend to, but do not always, prevent vehicles from crossing the curb line; mountable curbs permit such vehicle crossings without much difficulty. The types of curbs used most commonly are in DelDOT's *Standard Construction Details*. Refer to Chapter Ten for criteria for curb installations.

4.2 GRADING CROSS SECTION

The geometric elements of the grading cross section include the width and shape of the graded roadbed that consists of either suitable natural material or specified imported material. The top surface of the roadbed soil is defined as the subgrade. The pavement structure is placed on the prepared roadbed. The pavement structure includes any required selected subbase materials, base materials and the various layers of paving courses. The subgrade includes the various cut and fill slopes related to grading operations, including side ditches, to prepare a surface for constructing the pavement structure.

4.2.1 SUBGRADE CROSS SLOPES

The cross slope of the bottom of pavement box (top of subgrade) should parallel the cross slopes of the finished traffic lanes for the full width of the roadbed, including shoulders. This allows the pavement structure to drain through the porous material into side ditches or, if necessary, an underdrain system. The subgrade slope should not be broken to parallel the steeper finished shoulder slope. The parallel relationship between the subgrade and the finished traffic lanes applies to both normal crown slopes on tangent sections of highways and superelevated sections on curves.

4.2.2 SUBGRADE WIDTH

The design width of the subgrade should be shown on the roadway typical section(s). The total subgrade width should be the sum of the widths required for travel lanes, shoulders, median area, and any side slopes necessary to meet the subgrade.

For ease of computation and field staking, the design width may be rounded off to the nearest 1 ft [0.3 m]. This will result in a slope from the outside of the finished shoulder to the subgrade shoulder slightly different from the slope prescribed by the standards, but the variations will be negligible.

4.2.3 SUBGRADE WIDENING FOR GUARDRAIL

The subgrade should be widened at locations where guardrail is to be installed. The purpose is to provide the required horizontal clearance from the edge of the normal shoulder to the face of the guardrail, ensure the stability of the guardrail posts when placed in the embankment, and reduce maintenance.

Normally, 6 feet [1.8 m] of widening is required. This width includes 2 feet [0.6 m] from the normal shoulder line to the face of rail and 4 feet [1.2 m] behind the face of rail to a newly established edge of shoulder (the point of intersection of the front slope with the stabilized shoulder subgrade). Widening requirements are shown in DelDOT's *Standard Construction Details* for guardrail including special details for tapered flares for end treatments.

Refer to Chapter Ten for criteria for guardrail installations for various conditions relating to high embankments, non-traversable hazards and bridge ends.

4.2.4 SIDE SLOPES

A roadway's cross section includes side slopes as illustrated and identified in Figure 4-3. Side slopes are important in maintaining the stability of the roadbed and pavement structure as well as providing an area for the safety of errant vehicles. Side slopes are constructed in both fill (embankment) areas (those falling above the natural ground level) and cut areas (those falling below the natural ground level). As a general reference, slopes in embankment areas are commonly referred to as fill slopes or front slopes. When it is determined that no parallel ditch section is needed the front slope is graded to meet natural ground. In cut areas, side slopes are referred to as front slopes and back slopes, the back slope being necessary to bring the roadway cross section back up to meet the natural ground level. Ditch sections included as part of either fill or cut sections have a front slope, a ditch bottom with a defined shape and width, and a back slope. Criteria for rates of

these slopes (by road classes) are shown in Figure 4-4. The application of the criteria is very important in selecting a safe cross section. This application is discussed in this section; a full understanding of the concepts presented in AASHTO's *Roadside Design Guide* is critical to the proper application of the criteria.

All three slopes depend upon a lateral horizontal area measured from the edge of outside travel lane, called "clear zone". Consideration must be given to the lateral clear zone requirements when applying the criteria in selecting all side slope and ditch sections for the design cross section.

4.2.4.1 SIDE SLOPES WITHIN THE CLEAR ZONE

A roadway's "clear zone" is the total roadside border area, starting at the edge of the inside traveled way, that is considered available for safe use by errant vehicles. In addition to any shoulder area, the clear zone area may consist of a combination of a recoverable slope, a non-recoverable slope, a traversable slope, a clear run-out area and a critical slope. These slopes are defined as follows:

- A recoverable slope is flatter than 4:1 and an errant driver has a high probability of being able to recover control of the vehicle.
- Non-recoverable slopes are embankment areas with slope ratios from 3:1 to 4:1 on which the vehicle will continue to the bottom of the slope.
- A traversable slope has a slope ratio between 3:1 and 4:1. Slopes in this range, if properly graded and clear of obstructions, will not allow the driver to recover control and steer back onto the roadway but will permit the vehicle to slow down and stop safely.
- A clear runout area follows a non-recoverable slope and is graded, shaped and made free of hazards (traversable) wide enough to allow an errant vehicle to safely stop.
- Critical slopes have a slope ratio of 3:1 or steeper and will require barrier treatment to protect an errant vehicle

Please note that in the Department's presentation of slope ratios it uses horizontal to vertical while the *Roadside Design Guide* uses the ratio of vertical to horizontal, e.g. DelDOT's 4:1 versus the *Roadside Design Guide's* 1:4.

The "forgiving roadside" concept recognizes that motorists do run off the roadway and that serious accidents and injuries can be lessened if at least a traversable recovery area is provided. The concept calls for a clear, unobstructed, relatively flat roadside area providing drivers an opportunity to recover control if their vehicle accidentally leaves the pavement surface. It may not be possible or practical to provide an area with flat slopes large enough to permit the driver to regain control of the vehicle. Where these areas can not be provided every attempt should be made to have an appropriate area clear of obstructions. The desired width of a project's clear zone varies based on several factors: (1) operating speeds, (2) traffic volume, (3) the steepness of slopes, (4) changes in slopes, (5) horizontal curvature, and (6) the accident history.

Any decisions on clear zone width obviously will influence the geometrics of the cross-section design, including design of side slopes. Since funds available for roadway improvements are limited, designers must consider the benefits and costs of alternate design treatments to provide the optimum clear zone design for any specific location. The proposed improvements for some projects do not take into consideration the clear zone based on the scope of work, such as minor improvements projects like pavement rehabilitation.

Table 3.1 of the *Roadside Design Guide* was developed to determine suggested roadside recovery area or clear zone distances for selected traffic volumes and speeds. The numbers are not precise since they are based on limited empirical data extrapolated to provide information for a wide range of conditions. Keep in mind

site-specific conditions, design speeds, rural versus urban locations, project scope and practicality. The clear-zone distances from Table 3.1 for horizontal curvature may be modified by using Table 3.2. These modifications are normally considered where accident history indicates a need or a specific site investigation shows definite accident potential which could be significantly lessened by increasing the clear zone width in a cost effective manner.

For relatively flat and level roadways, the clear-zone concept is simple to apply. However, it becomes somewhat less clear when the roadway is in a fill or cut section where roadside slopes may be either positive, negative, or variable, or where a ditch exists near the traveled way. Consequently, these features must be discussed before a full understanding of the clear zone concept is possible.

A basic understanding of the clear zone concept is critical to its proper application. As previously mentioned, the numbers obtained from Table 3.1 of the *Roadside Design Guide* are based on limited empirical data and extrapolated to provide information for a wide range of conditions. Thus, the numbers represent a reasonable measure of the degree of safety suggested for a particular roadside, but they are neither absolute nor precise. In some cases, hazards outside the clear zone may require removal or shielding depending on the severity of the hazard, the projected ADT, projected construction costs, and the classification of the roadway. The selection of an appropriate clear zone distance amounts to reaching a compromise between balancing user safety, construction costs, land use and social impacts, environmental concerns and the many other constraints that influence project decisions. Appropriate application of the clear zone concept will often result in more than one possible solution.

Chapter 3 of the *Roadside Design Guide* states: "The guidelines found in this chapter may be most applicable to new construction or major reconstruction." For other types of projects the guide recognizes that it may be not within the scope, not within the available funding, too environmentally disruptive, or impractical to achieve the recommended clear zone widths. Projects at this level are evaluated on an historical basis, identifying safety problems and obvious obstructions with emphasis placed on correcting these, if clear-zone related. TRB's *Special Report 214 Designing Safer Roads* should also be referred to when designing these types of projects.

In Delaware, experience has shown that the amount of lateral clear zone that should be provided varies from location to location. For each specific project a lateral clear zone is established by considering the following factors:

- Run-off-the-road accident experience,
- Design speed,
- Operating speed,
- Traffic volume,
- Steepness of side slopes,
- Profile grade,
- Horizontal curvature,
- Amount of roadside development,
- Sight distances,
- Level of improvement,
- Policy on removal and/or preservation of trees,
- Policy on installation of above ground utilities and
- Severity and location of the hazard.

After consideration of these factors and the clear zone requirements, the designer recommends a lateral clear zone or zones for each project. Clear zone widths generally will be uniform throughout the project except where widened for curvature. Lateral clear zone width decisions are an important design issue and are fully documented as described in Chapter 3.

Figure 4-4
Side Slope Criteria

Cut Slopes						
Road Class	**Front Slopes**		**Back Slopes**			
	Desirable	**Maximum**	**Desirable**	**Maximum** [1]		
				Depth of Cut		**Slope Ratio**
				US Cus-tomary	**Metric**	
Arterial	6:1	4:1	6:1	0 to 5 ft 5 to 10 ft 10 ft+	0 to 1.5 m 1.5 to 3 m 3 m +	4:1 3:1 2:1
Collector	6:1	4:1	4:1	0 to 5 ft 5 to 10 ft 10 ft +	0 to 1.5 m 1.5 to 5 m 3 m +	4:1 3:1 2:1
Local	6:1	4:1	4:1	0 to 5 ft 5 to 10 ft 10 ft +	0 to 1.5 m 1.5 to 5 m 3 m +	4:1 3:1 2:1

Fill Slopes						
Road Class	**Within Clear Zone**		**Outside Clear Zones**			
	Desirable	**Maximum**	**Desirable**	**Maximum**		
				Depth of Fill		**Slope Ratio**
				US Cus-tomary	**Metric**	
Arterial	6:1	4:1	6:1	0 to 5ft 5 to 10 ft 10 to 15 ft 15 ft +	0 to 1.5 m 1.5 to 3 m 3 to 5 m 5 + m	6:1 4:1 3:1 2:1
Collector	6:1	4:1	6:1	0 to 5 ft 5 to 10 ft 10 to 15 ft 15 ft +	0 to 1.5 m 1.5 to 3 m 3 to 5 m 5 m +	6:1 4:1 3:1 2:1
Local	4:1	3:1	4:1	0 to 3 ft 3 ft +	0 to 1.5 m 1.5 m +	3:1 2:1

Note:

Refer to the text and Tables 3.1 and 3.2 in the *Roadside Design Guide* for proper application of side slope design and clear zone requirements.

[1] The maximum back slope ratio outside the clear zone may be increased because of right-of-way restrictions.

In urban areas where curbs are often utilized, the space available for clear zones is generally restricted. In areas where barrier curbs are used, the clear zone shall extend to a minimum of 2 ft [0.6 m] beyond the face of curb, with wider clear zones provided where possible. In locations with mountable curbs, the clear zone width provided shall be as determined in *Roadside Design Guide*'s Table 3.1 as adjusted by Table 3.2.

If the clear zone width requirements, as determined by the above procedure, are not practicable because of local conditions or are inadequate because of specific safety problems, the designer should consider adjustments to the highway geometry or the installation of barriers. Refer to Chapter Ten for policies and criteria for barriers.

Utility poles are considered an obstruction and are not permitted within the clear zone without an engineering study, including accident history, and proper documentation. A design exception may be necessary. The use of breakaway utility poles can be considered as an alternative to moving or burying the utility where there is documented accident history.

4.2.4.2 FRONT SLOPES

Slopes within the selected clear zone must have a slope rate that is relatively flat. Normally a 6:1 slope (or flatter) should be used outward and downward from the edge of the finished shoulder to the outer limits of the lateral clear zone. Steeper slopes, up to a maximum of 3:1, may be used for low traffic volume roads and for conditions where flatter slopes would cause inordinately high costs. With slopes steeper than 4:1 the horizontal distance of the steeper slope cannot be used to meet the clear zone requirements.

For relatively low embankment heights, fill slopes extending outward and downward from the outer limits of the lateral clear zone to the natural ground normally should be the same.

Under conditions of high fills and/or right-of-way restrictions, steepening the fill slopes to a maximum of 2:1 beyond the clear zone may be considered, but they should be designed as flat as practical within the constraints of local right-of-way conditions. Slopes steeper than 3:1 cannot be mowed with conventional mowers. The cost of flattening slopes versus the cost of guardrail is also a consideration as guardrail is deemed a roadside obstruction and can be a continuing maintenance problem.

Slopes that parallel the traveled way can be recoverable, non-recoverable, traversable or critical. For recoverable slopes that are smooth and traversable with slopes of 4:1 or flatter, the suggested clear zone may be taken directly from the *Roadside Design Guide*'s Table 3.1, adjusted by Table 3.2 as necessary. Motorists who encroach on recoverable slopes can generally stop their vehicles or slow them enough to safely return to the roadway. Fixed object hazards, such as culvert headwalls, should not extend above the embankment either within the clear zone or beyond if the embankment is traversable to the bottom.

Slopes between 3:1 and 4:1 are considered non-recoverable slopes for which most motorists would be unable to safely stop or return to the roadway. It is very important that these embankments be traversable since a high percentage of encroaching vehicles will reach the toe of these slopes. The clear zone cannot logically end on the slope. Fixed object hazards should not be constructed along such slopes, and a clear runout area should be provided at the base of the slope. The runout area is a relatively flat clear area wide enough to allow the vehicle to stop. Figure 4-3 shows an example of such a clear zone. The clear zone width is the sum of the widths of the shoulder, any recoverable slopes, and the clear runout area, but excludes the non-recoverable slope.

Front slopes steeper than 3:1 are considered critical slopes and a driver will most likely lose control. If a slope steeper than 3:1 begins closer to the traveled way than the suggested clear zone for that specific roadway and the slope

fic safety are observed advantages of wide, depressed medians.

Designing a relatively narrow depressed median creates problems. The result is that the longitudinal drainage ditch in the center of the median is too shallow, or the transverse slopes from the roadways to the ditch are too steep.

Median side slopes of 6:1 or flatter, for a distance of at least 30 ft [9 m] from the edge of the traffic lanes, are preferred. Other median slopes (for median crossovers, ditch blocks, etc.), that might be in the path of an out-of-control vehicle, should be 6:1 as a minimum and preferably 10:1 or flatter as a safety feature.

A width of 40 ft [12.0 m] or more for depressed medians permits adequate drainage design with flat slopes. A median width of at least 50 ft [15.0 m] can safely store a school bus. Wider medians are desirable where right-of-way permits allowing for the placement of a median bridge pier or overhead sign structure without the need for barrier protection. Wider medians should also be considered where there is a potential for adding travel lanes in the median to meet future traffic demand. Also see the *Green Book* pages 460 and 461 for further discussion on this subject.

Where flat longitudinal slopes on the roadway are encountered, the cross slopes of the median may be varied to increase the longitudinal slope of the median ditch. For example, the cross slope may be kept very flat (10:1 or flatter) at the upper end of the drainage area and steeper (6:1) at the lower end.

4.3.4 MEDIAN BARRIERS

For divided highways with large traffic volumes and high operating speeds, a wide, depressed median is the best choice. Under some conditions this is not practicable, and a flush or raised median must be provided. But in this case, some type of physical barrier must be placed in the median to prevent out-of-control vehicles from crossing into opposing traffic lanes.

Several types of physical median barriers can be designed. Refer to Chapter 10 and the *Roadside Design Guide* for criteria for median barriers.

4.3.5 MEDIAN OPENINGS

Refer to Chapter 7 for the design of median openings and channelization for left turns.

Certain principles should be followed in the design of a channelized intersection, but the extent to which they are applied will depend on the characteristics of the total design plan. These principles are:

- Motorists should not be confronted with more than one decision at a time.
  - Unnatural paths that require turns greater than 90° or sudden and sharp reverse curves should be avoided.
- Areas of vehicle conflict should be reduced as much as possible. Channelization should be used to keep vehicles within well-defined paths that minimize the area of conflict.
- The points of crossing or conflict should be studied carefully to determine if such conditions would be better separated or consolidated to simplify design with appropriate control devices added to ensure safe operation.
- Refuge areas for turning vehicles should be provided clear of through traffic.
- Prohibited turns should be blocked wherever possible.
- Location of essential control devices should be established as a part of the design of a channelized intersection.
- Channelization may be desirable to separate the various traffic movements where multiple-phase signals are used.

7.3.3 ISLANDS

Design of islands is the principal concern in channelization. An island is a defined area between traffic lanes for control of vehicle movements. Islands may be delineated or outlined by a variety of treatments, depending on their size, location and function. Types of delineators include: (1) raised islands outlined by curbs, (2) islands delineated by pavement markings, and (3) non-paved areas formed by the pavement edges–possibly supplemented by delineators on posts or other guide posts.

Islands provide three major functions:

- **Channelizing islands** control and direct traffic movement, usually turning;
- **Divisional islands** divide opposing or same-direction traffic streams, usually through movements; and
- **Refuge islands** provide refuge for pedestrians and bicyclists.

Most islands combine two or all of these functions. Triangular channelizing islands, used for roadways, subdivision street entrances or commercial driveway entrances, serve all three functions where there are pedestrian facilities.

Islands should be sufficiently large to command attention, and to accommodate pedestrian refuge and pedestrian signal poles where they are needed. Curbed islands normally should be no smaller than 50 ft^2 [5 m^2] for urban streets and about 75 ft^2 [7 m^2] for rural intersections; however, 100 ft^2 [9 m^2] minimum is preferred for both. Triangular islands should not be less than 12 ft [3.6 m], preferably 15 feet [4.5 m], on a side before rounding the corners; those with curb ramps, pedestrian refuge and pedestrian signal poles should have sides at least 15 ft [4.5 m] and preferably 20 feet [6.0 m] on a tangent side resulting in a minimum island area of 175 ft^2 [16 m^2]. Median islands narrower than 8 ft [2.4 m] from back of curb to back of curb cannot be mowed effectively; therefore they should be paved. Elongated or divisional islands should not be less than 4 ft [1.2 m] wide, 6 ft [1.8 m] if pedestrians are anticipated and 20 to 25 ft [6.0 to 8.0 m] long.

DelDOT has adopted general rules for the placement of islands. The first preference is to design the intersection with radii that accommodate the selected design vehicle path without creating large open paved areas requiring the need for islands to direct traffic. The overriding criteria are operational efficiency and safety. In many locations the best way to accomplish this is with islands. Curbed islands usually provide the better alternative to delineate the location, minimize maintenance and control drainage. Using the proper type of curb and offsets are important design considerations. Full shoulder width offset and mountable P.C.C. Curb, Type 2, are the preferred design elements. Typically, the island is offset from the traveled way the full width of the

shoulder or turn lane. This offset may be reduced where necessary as determined by a traffic study to only five feet to accommodate bicycles under the following conditions:

- Urban, suburban and developing areas where, due to queue lengths and congestion, traffic needs to be discouraged from using the shoulder to pass on the right.
- Commercial driveway entrances or streets leading up to an urban, suburban or developing intersection to prevent illegal shoulder traffic before the deceleration lane. Here, the island also offers protection to the vehicle entering the highway and prevents a car crossing the highway entering the business or side street from being involved in an angle crash.
- Crosswalks where pedestrian refuge and shortening the length of the crossing is needed, particularly with signalization.

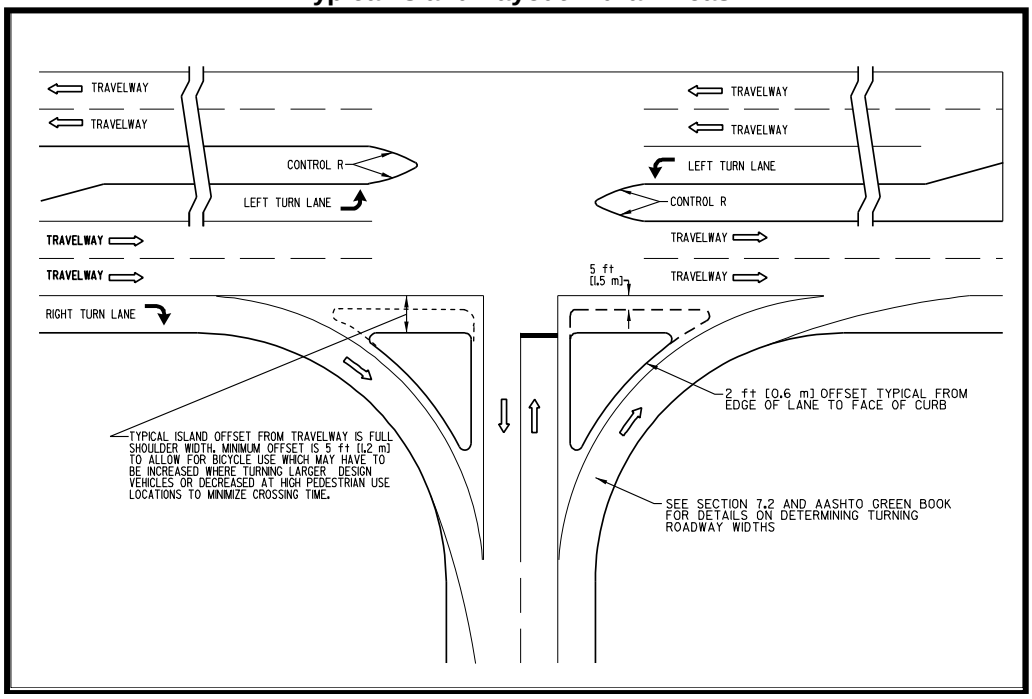
In areas with high pedestrian traffic where there is a need to minimize the distance between refuge areas, the offset for bicycles may be reduced to 4 ft [1.2 m] from an approaching curbed section; in this case, it is preferable to offset the nose of larger islands 4 to 6 ft [1.2 to 2 m] from approaching curbed lanes to allow an errant vehicle to recover before striking the curb. To accommodate the variety of vehicle, pedestrian and bicycle movements in urban areas, various offsets and configurations may be needed within the same intersection.

Figures 7-8 and 7-9 show a typical preferred island layout for rural and urban conditions.

No matter what the island configuration:

- Positive drainage must be provided for the safety of vehicles and pedestrians.
- The corners of the island shall be flush with the pavement, tapering back to full height at a slope of 4:1 for triangular channelizing islands and 10 feet [3.05 m] for median islands (where the sleeve for the "Keep Right" sign would be placed) for snow plowing operations and errant vehicles.
- The corners of islands not offset the full width of the shoulder adjacent to the roadway shall be delineated with tubular delineators.

Figure 7-8
Typical Island Layout–Rural Areas



Chapter Nine

PAVEMENT SELECTION

This chapter discusses the general criteria, procedures and responsibilities for structural design of highway pavements. In addition there is information given on the various types of pavements, pavement rehabilitation techniques, and other factors that enter into pavement design and final pavement selection.

For the purposes of uniform and consistent design practices, the Department has adopted the criteria and procedures as set forth in the *AASHTO Guide for Design of Pavement Structures 1993*. This chapter briefly reviews the concepts and criteria used. Reference should be made to the AASHTO Guide for more detailed information on design procedures, if needed.

The design procedures include the determination of total thickness of the pavement structure as well as the thickness of the individual components using input parameters specified in the DARWIN 3.01 computer program. Provision is made for the design of equivalent alternate pavement sections, with the selection primarily a function of availability of materials, comparative costs, constructibility, and availability to traffic.

The discussion and explanatory material presented here are intended to give the designer a general understanding of pavement design concepts, alternative paving treatments, and a basic understanding of the information contained in a soil survey and pavement design report.

9.1 DESIGN RESPONSIBILITY

The design of pavement structures has some jointly shared responsibilities between the Materials and Research (M&R) Section and the responsible Project Development section. However, the primary responsibility for structural design and final recommended pavement sections is that of the M&R Section.

9.1.1 SOIL SURVEY/PAVEMENT EVALUATION REQUEST

There are several elements in the design process that need to be accomplished before a pavement section recommendation can be requested from the M&R Section. The project handoff package will describe the project scope. If the intent is to construct new pavement or replace the existing pavement then a soil survey, pavement design and pavement type recommendation will have to be requested.

The following should be available when requesting borings for a soil survey from DelDOT's Geotechnical Engineer and/or corings of the existing pavement from DelDOT's Pavement Design Engineer:

- preliminary surveys,

- survey plans with the location of the investigation, including the road name and state maintenance road number, and any predetermined locations marked,
- existing right-of-way verified,
- Right-of-entry to trespass, if needed,
- Purpose of the investigation in order to determine what pertinent information is required from the cores or the borings.
- If a boring is required, the depth of the boring. If the designer is unsure of depth to sample, contact the Geotechnical Engineer for guidance.

Coring and boring requests are typically processed within 30 days. If a coring and boring request is received simultaneously, they will be processed concurrently. If the boring request is received after the coring request has been completed, it will be treated as a different request and will be processed within 30 days of receipt.

For a pavement design, the following information is required with the request:

- Design year traffic data (AADT)
- Design year truck percentages
- Weight group pattern of trucks
- Directional split
- Existing pavement structure (if applicable)
- Subsurface investigation report (if applicable)
- Description of any existing pavement deterioration

Copies of any existing corings, borings, or subsurface condition reports should be provided to the Pavement Design Engineer when sending the pavement design request. The Pavement Design Engineer may elect to have additional corings, borings, or other investigations performed to ensure the existing conditions are known and considered when performing the pavement design. All information, including the design, will be forwarded to the designer.

For projects designed by a consultant, the design consultants must:

- Develop the subsurface investigation plan to have drilling and inspection services provided under their direction.
- Notify the DelDOT Project Manager of the scope of the subsurface investigation plan and provide a boring plan sheet(s) to the Department. The DelDOT Project Manager will forward the boring plan to the Materials & Research (M&R) Geotechnical Engineer
- Specify to M&R what soil testing is requested.
- Inform the Geotechnical Engineer at least 24 hours in advance when drilling and soil sampling is to begin and provide the name and phone number of the consultant who is to receive the M&R soil test information.
- Arrange to have the soil samples and copies of the field logs brought to the M&R lab within 24 hours after obtaining samples.

The Geotechnical Engineer shall:

- Perform, or arrange for, all soil testing required for the project as requested by the design consultant.
- Furnish completed boring logs and laboratory test summaries to the design consultant with a copy to the DelDOT Project Manager.
- Provide the DelDOT Project Manager regular weekly updates on the progress of the drilling/testing programs.
- Inform the DelDOT Project Manager when problems arise and when the testing programs are completed.

Many projects are initiated with the intent of extending the service life of the existing pavement section. Rather than rebuilding the entire pavement, improvements are made in the riding quality, skid resistance and limited structural improvements through various rehabilitation methods. For projects of this type a pavement

evaluation is requested. A pavement evaluation includes an extensive pavement survey noting pavement conditions, drainage and major distress problems. The initial evaluation will determine which rehabilitation method(s) should be considered. Based upon the alternatives being considered, it may be necessary to conduct detailed measuring and testing, including coring and sampling, and accurately determining structural clearances to any overhead structures. The final report received by the designer will contain most if not all this collected field data and will include the recommended rehabilitation method. Much of the field data collected can be a very valuable tool in designing the project.

9.1.2 SOIL AND PAVEMENT DESIGN REPORT

The M&R Section performs the soil sampling and pavement coring. This field data is tested in the laboratory and the results documented in the form of a Soil Survey. This report includes the soil profile data summary showing the sample location, depth, soil profile description, soil classification and any remarks. The soil samples are tested and a summary of this analysis is included in the report.

Based upon the subgrade soil characteristics, the report may provide recommendations for muck excavation, limits of special fill, need for underdrains, grade adjustments, embankment construction, the use of geotextiles or other special construction considerations.

An important part of the report is the pavement design portion. Using the AASHTO Guide, the soil survey data, and past experience, the recommended pavement type and thickness by components is included in the report. Pavement sections for shoulders and turn lanes are also included.

The report may provide an alternate design including at least one rigid pavement and one or more flexible alternatives. Project Development and M&R will meet and mutually agree on which pavement to use on the project. If necessary, the M&R Section may perform an economic and life cycle cost analysis or other studies in making the final determination and recommendation for pavement type and section. For many projects, only one pavement design recommendation is made.

The Soil and Pavement Design Report should be evaluated thoroughly and considered as the design progresses. In the design process significant changes in the proposed profile or alignment can dramatically affect the pavement design and need to be discussed with M&R, perhaps even before such decisions are finalized. In addition the proposed construction sequencing, methods of construction and maintenance of traffic plan can dramatically affect whether the recommended pavement materials can be placed in a timely manner and meet the in-place performance quality necessary for the intended service life.

9.1.3 PAVEMENT SELECTION

As mentioned in Section 9.1.2, there may be several choices of structurally equivalent pavements. The choice of pavement, particularly on reconstruction and new construction is a major decision and needs to be approved prior to proceeding with the design.

The factors considered in making the final decision on pavement type are quite varied from empirical to subjective and may include several of all of the following:

- Project scope—as initiated,
- Cost to construct or rehabilitate the pavement,
- Available project funding,
- Construction sequencing as it relates to controlling through and local traffic,
- Construction sequencing as it relates to serving commercial areas,

property of soil. The resilient modulus is used directly for designing flexible pavements but must be converted to a modulus of subgrade reaction (k-value) for the design of rigid pavements or composite pavements. The resilient modulus is also a soil property used in analyzing multilayered material systems for predicting roughness, cracking, faulting, rutting, and other potential distresses.

The value of a roadbed's resilient modulus is dependent on how well the roadbed soil is placed in conformance with the specified compaction parameters. For most projects, the material is to be placed in accordance with the *Standard Specifications* with no special treatment. However, the Soil and Pavement Design Report may indicate that there is anticipated difficulty with the existing roadbed soil meeting the design M_R value. For soils that are excessively expansive the report may recommend these soils be covered by select material sufficiently deep enough to reduce or eliminate the expansive affect of the natural material. Other solutions may include the adding of an admixture to reduce the water content or the use of a geotextile.

One of the more difficult soils encountered on projects are those having a large organic content. These materials are extremely compressible, unstable and frequently non-uniform in properties and depth. These soils are the most complicated and expensive to deal with in order to provide an adequate roadbed. Small, shallow or localized deposits are most often excavated and replaced with suitable material. Deeper and more expansive areas involve more detailed geotechnical design, more complicated construction techniques and costs. Treatments other than complete removal are more time dependent allowing for the slow consolidation and removal of excess moisture. Methods available include surcharge embankments for preconsolidation of the underlying material usually involving sand drains which allow the water to rise to the surface and be removed. The M&R Section is responsible for identifying and designing the most economical method treating this type of problem area.

Underdrains (a system of perforated pipes to collect and transmit the water to an outfall site) are recommended for use on all roadway projects to adequately address drainage and removing water from the roadbed. If site conditions indicate that underdrains may not be required, contact the M&R Section to initiate further investigation.

The soil and pavement condition survey will normally identify roadbed drainage problem areas or soils highly susceptible to expansion or loss of strength with increase in water content. When either of these conditions exist, the M&R Section may recommend additional work and/or materials to address the existing conditions.

Another type of material encountered in constructing roadbeds is classified as cohesionless (sandy) soil and is much more difficult for the contractor to place and compact; it is readily displaced under the load of the equipment. To stabilize this type of soil it may be necessary to blend granular material or add a suitable admixture. Wet clay soils may also be encountered. Because of high moisture content this type of soil is unstable and cannot be compacted. Long periods of dry weather and exposure to the air are required to reduce the water content. To reduce the time necessary to reuse these materials, the recommendation may be to add a suitable admixture that hastens drying or cover the area with a more suitable select material. Removing the material and replacing it with suitable material allowing construction to continue is an option. The material may be used in areas that don't require compaction or moved to an available site for air-drying and reuse at a later time.

9.3.5 PAVING MATERIALS

Depending upon materials that comprise a pavement, the pavement structure is identified as either a flexible or rigid pavement. Combining these two types of paving materials in a pavement structure as a subbase or surface course results in a composite pavement.

pavement structure. For lesser facilities, it may be perfectly acceptable and even economical to use a reduced thickness.

9.3.9 LIFE-CYCLE COSTS

Life-cycle costs are costs and benefits that accrue during a pavement's complete life cycle. These costs include the initial construction costs, maintenance costs, rehabilitation costs, resurfacing costs, maintenance of traffic costs, salvage or residual value and user costs.

As a part of the pavement selection decision, particularly when more than one pavement design or rehabilitation procedure is proposed, an economic comparison may be needed. Two methods are detailed in the ASSHTO Guide to determine life cycle cost comparisons, net present worth and equivalent uniform annual cost. Whichever method is used, it is essential that the analysis periods be of equal length.

9.3.10 SHOULDER DESIGN

The inclusion of a shoulder adjacent to the main pavement structure improves pavement performance. The AASHTO guide does not provide a design method for determining the shoulder section. The M&R Section recommends a shoulder section that is compatible with the proposed mainline pavement section, has good constructibility and has performed well in the past. Shoulders are usually designed to carry 10 percent of the projected Average Daily Traffic (ADT).

9.4 DESIGN FOR NEW CONSTRUCTION OR RECONSTRUCTION

Constructing a pavement section is one of the most costly items on new construction and reconstruction projects. In making a selection on the type of pavement to construct it is usually necessary to analyze alternate pavement types and combinations of various support materials. In performing the analyses, the AASHTO procedure requires the pavement designer to provide input in several categories:

- Design variables,
- Performance criteria,
- Material properties for structural design,
- Structural characteristics, and
- Reinforcement variables.

9.4.1 DESIGN VARIABLES

A set of design criteria is established for each project including the pavement's expected service life (performance period) and projected design life (replacement). The traffic projection for the cumulative expected 18-kip [80 kN] equivalent single axle loads (ESAL) during the analysis is determined. The level of reliability is selected and any detrimental environmental factors identified. There is a heirachy of application of these variables with the most important roadways assigned the most stringent and detrimental values to a pavement's performance.

9.4.2 PERFORMANCE CRITERIA

Performance of a pavement is measured by its serviceability to the expected users. The concept is to design a pavement, which at the end of the proposed performance period will still have a predefined minimum level of serviceability (PSI). The terminal level of serviceability is selected based on the lowest index the user will tolerate, or as defined in a pavement management strategy before rehabilitation, resurfacing or reconstruction becomes necessary. DelDOT typically uses a terminal PSI of 2.5 or 3.0, which varies based on functional classification and use.

9.4.3 MATERIAL PROPERTIES FOR STRUCTURAL DESIGN

For the design of flexible pavements, roadbed materials are characterized based on their effective elasticity or resilient modulus, M_R. Their

Figure 9-2
Layer Coefficients

Material Type	Layer Coefficient
Type C Surface Mix	0.40
Type B Binder Course	0.40
Bituminous Concrete Base Course	0.32
Soil Cement	0.20
Graded Aggregate Base Course	0.14
Select Borrow	0.08

9.5.6 MINIMUM LIFT THICKNESS

Although the equations allow for a great number of thickness variations, there are the practicalities of constructing and maintaining a facility, which must be considered. Depending upon the material being placed, there are minimum and maximum limits in the placement depth that are practical for the available equipment to compact and are economical.

Minimum lift thickness for hot-mix is recommended to be three times the nominal aggregate size in the mix. Figure 9-3 shows the practical maximum and minimum lift thickness (compacted) that are to be applied to the materials normally used in constructing a flexible pavement section.

9.5.7 TEMPORARY PAVEMENTS

It is not practical to attempt to follow the formalized AASHTO procedures for design of temporary pavements such as needed for detours during construction. Variations in speed and ease of placement as well as the anticipated required service life of the detour significantly affect the economic justification for the structural design.

When temporary pavements are needed designers should closely coordinate with the M&R Section in the development of a practical pavement design based on knowledge of local conditions and engineering judgment.

Figure 9-3
Lift Thickness

Type of Material	Minimum Lift Thickness	Maximum Lift Thickness
Type C Surface Mix (9.5 mm)	1-¼ inches [30 mm]	2 inches [50 mm]
Type B Binder/Base	2-¼ inches [60 mm]	3 inches [75 mm]
Bituminous Concrete Base Course	3 inches [75 mm]	6 inches [150 mm]
Graded Aggregate Base Course	4 inches [100 mm]	8 inches [200 mm]
Soil Cement	4 inches [100 mm]	6 inches [150 mm]
Select Borrow	4 inches [100 mm]	8 inches [200 mm]
Open Graded	1 inch [25 mm]	1 inch [25 mm]

9.6 DESIGN FOR RIGID PAVEMENTS

Rigid pavements consist of a Portland cement concrete slab on a subbase course. The design procedure consists of developing an effective modulus of subgrade reaction based on subbase treatment and thickness, determine the slab thickness, allowing for any stage construction, adjusting for adverse environmental conditions, determining type of joints, joint sealant, and the required reinforcement.

ly. The only reinforcement in non-reinforced pavements is tie bars across the longitudinal joints to keep the slab from separating and dowels at the transverse joints to transfer loads across the joints. If dowels are not used, the design depends only upon aggregate interlock to transfer these loads. With loss of subgrade support, cracking of the slab can occur at almost any location. However, more common problems occur at joints. Once the joints are no longer free to move, spalling, buckling, and random slab cracking can result. If joint sealant is lost, aggregate interlock is lost, or cracks become too wide, pumping of the subgrade within the travel lane and the shoulder can occur. This leads to erosion of support material and faulting and cracking

Reinforced concrete pavements perform much like non-reinforced pavements. They usually have longer joint spacing and internal reinforcement to resist the larger tensile stresses. There are fewer joints to relieve stresses; when their free movement is restricted, rapid faulting of the pavement can occur. In addition, normal shrinkage, thermal curl, and load applications cause cracks in the slab that over time grow in width, allowing moisture and road salt to infiltrate. Corrosion of the reinforcing mesh occurs. As the loads are repeated, pumping begins leading to faulting and spalling of the pavement.

Continuously reinforced concrete pavements deteriorate under heavy truck loading and are also adversely affect by moisture under the slab. This type of pavement is more complicated to construct and deterioration will occur due to inadequate consolidation, poor vertical steel placement, and inadequate steel overlap.

Because of the need to maintain the pavement systems now in place, several rehabilitation strategies have been developed to address the various problems that may affect a pavement's performance. The objective is to extend the pavement's service life. The alternative methods of rehabilitating a pavement range from a simple overlay to complete removal and replacement. When developing a rehabilitation strategy a combination of the following alternatives are considered:

1. Resurfacing to provide structural capacity and/or serviceability either using concrete or asphalt,
2. Replacing or restoring malfunctioning joints,
3. Pavement subsealing prior to resurfacing or as a part of concrete restoration,
4. Grinding rigid pavements to restore smoothness.
5. Removing and replacing deteriorated materials,
6. Reworking or strengthening bases and subases,
7. Recycling existing material,
8. Improving the subdrainage or adding underdrains,
9. Joint and crack sealing,
10. Full depth pavement repair,
11. Partial depth pavement repair, and
12. Cracking and seating.

M&R and other team members develop the most effective strategy through a detailed pavement evaluation. This evaluation is normally prepared as a part of the Department's long term pavement rehabilitation program and will be a part of the project initiation data. The three steps for determining the preferred strategy are to (1) determine the cause of the pavement distress, (2) develop a list of possible solutions that cure and hopefully prevent reoccurrence, and (3) recommend the preferred solution. The preferred solution includes an analysis of funding, traffic control problems, minimum desirable service life, utility conflicts, clearances to overhead structures, available materials and equipment, contructibility, future maintenance and reliability based on past performance.

9.7.2 TYPES OF DISTRESS

Although furnished at the project initiation stage, each project usually will have a project scoping meeting and preliminary field review prior to beginning detailed design. The field review will include checking the pavement for condition and any damage that may have occurred since the last pavement survey to confirm the proposed strategy is still valid. The following discussion is a brief description of the various conditions the designer may observe depending upon the type of pavement being evaluated.

9.7.2.1 ASPHALT PAVEMENTS

Asphalt pavements usually have the following major distress conditions: (1) alligator or fatigue cracking, (2) longitudinal and transverse cracking, (3) depressions, and (4) rutting.

Alligator or fatigue cracking is caused by repeated traffic loadings. They are a series of interconnecting cracks caused by fatigue failure of the bituminous concrete surface. The crack starts at the bottom of the asphalt surface or the stabilized base due to high tensile stresses and propagates to the surface as a longitudinal crack. After repeated load applications, a network of these cracks form that look like chicken wire or the skin of an alligator. This type of cracking does not occur in asphalt overlays over concrete—only in high load areas and is considered a major structural distress. Pattern cracking not in high load areas is called block cracking.

Longitudinal cracks are parallel to the pavement's centerline or paving laydown direction. They may be caused by: (1) a poorly constructed paving lane joint; (2) shrinkage of the bituminous concrete surface due to low temperatures or hardening of the asphalt; or (3) a reflective crack caused by cracks beneath the surface course, including cracks in Portland cement concrete. Transverse cracks are perpendicular to the pavement's centerline; they are caused by (2) or (3) above and are not usually load-related.

Depressions are localized pavement surface areas that are slightly lower than the surrounding pavement. Depressions are most noticeable during and after a rain. If deep and large enough, depressions may cause hydroplaning or an unpleasant ride. Depressions may be initially built into the pavement by the paving operation or as a result of settling of the surface support structure.

Rutting is a surface depression in the wheel paths. Usually, there is uplift along the sides of rutted areas. Rutting is the result of permanent consolidation or lateral movement of any of the pavement layers or subgrade due to traffic loads. Rutting may also occur because of plastic movement of the asphalt due to high temperatures, poor design mix or inadequate compaction during construction.

In addition to the major distresses, the pavement survey may indicate surface corrugation areas, joint reflection cracking, lane and shoulder drop off, lane and shoulder separation, patch deterioration, polished aggregate, potholes, raveling, and weathering.

If necessary, the M&R Section may determine that field observations are not adequate or need to be supplemented to identify underlying problems. In that case nondestructive testing (NDT) will be conducted. NDT is used to:

- Evaluate the in-situ (in-place) structural capacity of the pavement,
- Evaluate the capacity of joint and load transfer, and
- Detect the presence of voids under the pavement.

9.7.2.2 CONCRETE PAVEMENTS

Jointed concrete pavement may have the following distresses: (1) pumping, (2) longitudinal cracking, (3) spalling of transverse or longitudinal joints, or (4) Alkali-Silica-Reactivity (ASR).

Pumping is the ejection of material through joints or cracks, caused by the deflection of the slab under moving traffic. In some poorly drained pavements particularly in superelevated sections, water may bleed through the joints and cracks after a rain or continuously if large amounts of water are present.

Longitudinal cracks are caused by a combination of heavy load repetition, locking of load transfer devices, thermal and moisture stresses, and curing shrinkage stresses. Cracks that are spalling and/or faulting are considered a major structural problem.

Spalling of cracks and joints is the cracking, breaking or chipping of the slab edges within 2 ft (0.6 m) of the joint or crack. Spalling usually does not extend the full depth of the slab, but intersects the joint or crack at an angle. Spalling is a result of one or a combination of the following: (1) excessive stresses at poorly sealed or cleaned joints and cracks which allow incompressible material to accumulate preventing the pavement from expanding, (2) disintegration of the concrete, (3) weak concrete overstressed by repeated loading, and (4) a poorly designed or placed load transfer device.

Continuously reinforced concrete pavements usually show punchout and patch distress. Punchout is the loss of aggregate interlock between closely spaced cracks. The cracks fault, spall and under load applications the steel reinforcement ruptures causing concrete pieces to punch down. This type of distress is considered a major structural problem. Due to the difficulty in patching continuously reinforced pavements the failure of previously constructed patches can be anticipated.

ASR is another type of distress commonly found in Delaware. The pavements exhibiting the most severe ASR were generally constructed in the 1980's. ASR is the reaction between the alkalis (sodium and potassium) in Portland cement and certain siliceous aggregates. The product of this reaction is a thermodynamically metastable. The gel in the presence of water absorbs it, and causes expansion and cracking of the concrete. Once an ASR pavement has been identified, one possible solution is to apply lithium treatments. Lithium treatment will not repair the concrete, but will slow the further progression of ASR. Normally the pavement will have to be removed and replaced. Overlaying ASR concrete with a standard hot-mix overlay may trap moisture in the slab and accelerate the ASR causing premature failure of the overlay. However, hot-mix overlays with water proofing properties may minimize further deterioration for several years. M&R will recommend the most suitable solution when encountering ASR pavements.

In addition, other pavement distresses that may be observed in a field review are blow ups, corner cracks, depressions, durability "D" cracking, lane and shoulder drop-offs, lane and shoulder separation, patch deterioration, popouts, and staining of the pavement due to subgrade drainage problems.

Concrete pavements can also display roughness caused by irregularities in the pavement surface that adversely affect the ride quality, safety, and vehicle maintenance costs. Roughness is measurable based on the multi-frequency of waves, wavelengths and amplitudes. Roughness can be built into the pavement when constructed or develop over time due to traffic, climate, and other factors. Equipment is used to measure the roughness and a profile developed showing the vertical movement between the trailer axle and body. The results are reported in in/mile or m/km for the International Roughness Index (IRI). The roughness survey identifies areas where severe roughness exists and needs correction. Data provided can be used in developing a PSI which estimates the user's subjective assessment of the pavement condition. Surveys taken before and after a project can be used to document the benefits of the work to the traveling public.

potential of overruns, the specific distress or distresses to be addressed should be clearly established both during the design and construction phases. Depending upon the severity, distresses that may lend themselves to full-depth repair are blow-ups, corner breaks, durability "D" cracking, ASR, excessive spalling, and loss of load transfer. Intermediate working cracks may also have to be repaired by full-depth replacement or a working joint. Not addressing areas that need full-depth repair prior to an overlay could result in continued deterioration and premature failure of the overlay.

The major considerations to ensure satisfactory performance using this rehabilitation approach are:

1. Joint design,
2. Selection of the repair areas and their boundaries,
3. Preparation of the repair area,
4. Placement and finishing of the repair material,
5. Joint sealing material and its installation, and
6. Curing time and traffic control.

Usually the full-depth repair material is the same as the adjacent pavement. However, funding, traffic control or other reasons may require that concrete pavement be repaired with bituminous material. Using materials of different physical properties and characteristics can cause several problems. Differentials in expansion and contraction lead to pushing, shoving and humping requiring frequent milling to maintain rideability. The bituminous patch is more compressible and may allow excessive opening of remaining joints in the concrete pavement resulting in spalling, pumping, and faulting. In overlaying the patched pavement, there may be an increase in reflective cracking from the underlying joints The patch design may not have an equivalent structural strength. The difference in initial cost and possible future maintenance problems may not make using dissimilar materials the best solution.

9.7.4.2 PARTIAL-DEPTH REPAIR

Some distresses within the upper third of the slab in concrete pavements lend themselves to partial-depth repairs. Partial-depth repairs may or may not be more cost effective than full-depth depending upon the size, location, number, materials used, lane closure time and production limitations.

The distresses applicable to partial-depth repair are:

1. Spalls due the use of certain types of joint inserts,
2. Spalls caused by joint movement locking due to failed sealant and subsequent intrusion of incompressible material,
3. Spalls caused by misplaced dowels or other load transfer devices, and
4. Localized areas of scaling.

The M&R Section should be contacted for details on partial-depth patches as they require the use of better designs and construction techniques to be successful.

9.7.4.3 SLAB STABILIZATION AND SLAB JACKING

Although not that frequently used, voids under pavements can be filled, restoring the support strength of the pavement structure. The loss of support is caused by erosion of the subbase and/or subgrade by pumping or, in severe cases, movement of freely flowing water. Slab stabilization does not increase a pavement's structural strength, correct depressions or faulting and other distresses.

A slab that has settled may be raised through a process called slab jacking, also known as mudjacking. This procedure which injects water-soil-cement slurry (i.e. grout or mud) through holes drilled into the pavement slab under high pressure lifting the pavement back into its origi-

type of problem. Unless the rehabilitation program involves complete pavement reconstruction, improvements are limited to design and construction of longitudinal drains. Any modifications to or additions of transverse drains or drainage blankets would be limited to projects involving complete pavement replacement. However, the addition of longitudinal perforated underdrains to collect and outlet excess moisture in the pavement structure is a feasible and cost effective option.

9.7.4.10 SHOULDER IMPROVEMENTS

Shoulders are evaluated much like the mainline pavement and are include in the distress survey, drainage survey, traffic survey, structural evaluation, and subgrade and materials evaluation. Shoulders provide not only safer traffic flow but also give lateral support for the mainline pavement structure.

Shoulders can be rigid or flexible and distress similar to mainline pavements. Many of the same rehabilitation techniques are used. Reviewing the existing cross section can be very valuable. If the initial shoulder pavement design was not compatible with the mainline pavement structure, it may be a major contributor to its failure.

Two of the most common problems which occur are lane/shoulder joint separation which allows water into the subbase, and blockage of water draining out of the mainline subbase which is usually of more granular and higher quality. The material used in shoulder construction also may be of a different thickness. All of these affect the interaction between the two pavement structures.

Other distresses found in shoulders are pumping, fatigue cracking, lane/shoulder drop-off, frost heaving and differential shoulder support.

9.7.5 RECYCLING

Recycling is the term used to describe the process, which uses existing pavement materials to construct new pavements. The primary purpose is to conserve natural resources. In some cases, there may also be a net cost savings.

9.7.5.1 RECYCLING RIGID PAVEMENTS

Eventually, all pavements reach the end of their useful and/or structural life and must be reconstructed. This occurs when there is little or no structural life left due to extensive cracking, extensive slab settlement or heave, extensive joint deterioration requiring excessive full-depth repair, extensive concrete deterioration due to poor durability, and failure to meet geometric design standards.

Two methods of recycling are: to break the pavement slabs into smaller sections and leave them in place (rubblizing) as a base for resurfacing; or to break and remove the pavement slabs to an area for crushing and reuse. The crushed pavement material may be reused as aggregate in untreated dense graded aggregate bases for Portland cement concrete surfacing, asphalt concrete surfacing, fill, filter material, and as a drainage layer for edge drains.

Each method of recycling has its own cost and feasibility studies that need to be conducted before making a selection. Both methods involve the use of vibratory, hydraulic, pneumatic, or diesel chisels or hammers to demolish the existing pavement. Common factors to consider are: is the resulting recycled material actually reusable; are the underlying soils adequate to support an upgraded pavement; and are there shallow utilities (older gas, water or sewer lines such as vitrified clay or cast iron) that are sensitive to the equipment pounding and vibration and may rupture during the demolishing process.

The pavement may be demolished, removed, crushed and used for other construction purposes. This involves hauling the material to a crushing plant and having an electromagnet remove any reinforcement. The crushed material and salvaged steel would be available for reuse. The coarse-aggregate-sized particles resulting from the crushing process have a good shape and an-

gularity, high absorption and may have a lower specific gravity than virgin aggregates. For reuse of this material the concerns are the possible contaminants in the rubble including reinforcing steel, dowel bars and baskets, admixtures, chemical substances such as deicing salts, sea salt, oil, joint sealant material and soil.

9.7.5.1.1 RUBBLIZATION

Rubblization is initially the most expensive pavement recycling method, however, a selective use of it can be very effective in cost and construction time.

When rubblizing plain or reinforced concrete pavement, the rubble is leveled and left in place as the base for a new pavement. Maintaining traffic is a consideration as there must be enough area for the equipment to operate as well as a protective barrier. Even on a four lane facility this will require a median shoulder strong enough to temporarily carry full traffic loads or possibly a detour.

The rubblized surface essentially remains at the same elevation as the existing pavement. Therefore, matching existing curbs, drainage structures, intersecting roadways, and driveways is more complicated and time consuming. In addition, it may become difficult to meet height standards for guardrail and other appurtenances, as well as maintain vertical clearances for structures.

Rubblization is for total reconstruction. Rubblization reduces the structural value of a PCC pavement to a stone base. It requires a thick overlay, typically 11 inches [280 mm] of asphalt or 10 to 12 inches [255 to 305 mm] of concrete. It is a major reconstruction technique and therefore it should be used only when the pavement has reached the end if its service life, as indicated by severe deterioration, ASR, or severe freeze and thaw damages, etc.

Rubblization can be used when other concrete pavement restoration methods will not work. Thoroughly evaluate the existing condition of PCC pavements. Concrete pavement restoration (CPR) techniques, such as diamond grinding, patching or sealing, should be ruled out before the use of rubblization is specified. Pavement Management and M&R should be consulted in the selection of reconstruction/restoration technique.

Rubblization cannot be used over a subgrade demonstrating widespread instability or of poor condition. Many concrete distresses result from poor subgrade support conditions. Rubblizing a pavement destroys the concrete slab's bridging action, causing problems to become more pronounced. The poor support condition could be due to poor soils, poor drainage or high moisture content. If the problem is widespread, rubblization cannot be used. Thorough subgrade investigation is essential for the successful application. Contact M&R for subgrade investigation.

Treat rubblized PCC pavement as a subbase. Although rubblization provides the benefits of an in-place recycle opportunity and an interlocked stone base, it has challenges as well. As for all subbase materials, gradation and density are two important factors, but the control of these two factors is more difficult since it is in-place recycling.

The pavement breaker may be powerful, but as energy dissipates through the depth of the slab, it produces smaller pieces at the top and larger pieces at the bottom. A soft subgrade or the reinforcement in the slab only compounds the difference in sizes. The recommended requirement is for the top pieces to be 3" maximum and 12" maximum at the bottom. Acceptance of a larger size will increase the probability of future reflective cracking. A good density is achieved through the interlocking and good compaction.

With good control of gradation and density, it is reasonable to expect a good fatigue resistance performance of an asphalt overlay, which is a major controlling factor for a flexible pavement service life.

Rubblization is a balancing act. The size of rubblized concrete can be controlled. A larger size will provide a stronger support (thus a better structural value), but it increases the likelihood of reflective cracking (thus a reduced service life). The designer needs to balance these two in the pavement design. A long term performance evaluation to validate this design value may be necessary.

Install a drainage system. An adequate subgrade drainage system is essential for rubblization and future performance of the pavement. Rubblization cannot be successfully done over a wet subgrade. The drainage system should be in place at least two weeks prior to the rubblization. In a special case, if the drainage system cannot be installed prior to actual rubblization, then a time limit should be specified to have the drainage system installed immediately following rubblization.

Use a test section, not a second pass of the breaker. A second pass over a rubblized area does not enhance the quality of rubblization, and it could cause more damage. Test sections should be done to calibrate all rubblization variables (machine related-velocity, frequency, pressure or force, shoe size and conditions related-concrete condition or state of distress, thickness and subgrade conditions). The objective is to achieve the required sizes of rubblization both at the top and bottom of the PCC pavement for a good service life.

Selection of Rubblization Equipments and Production Rates - Among the different types of equipment for breaking the pavement, two frequently used types are resonant pavement breakers (a low impact, low-amplitude, high frequency vibration to the slab) and multiple head breakers (12 to 16 drop hammers mounted laterally in pairs with half of the hammers in a forward row and the remainder diagonally offset in a rear row). The multi-head breaker rubblizes a full lane width in a single pass with a production rate of about one lane mile per shift per day, while a resonant breaker may take up to 20 passes for a full lane width with a production rate of about ½ miles per shift per day.

The multi-head breaker may cause damages to the subgrade. the resonant breaker may produce concrete size too small at the top to meet the design requirement. Unless the equipment selection is specified on the plan, the specification will allow the use of either breaker.

A concrete overlay can be used. Rubblization was developed to eliminate reflective cracks in the asphalt overlay. Engineers still have an option of using concrete overlay, considering cost, service and construction time. Consult M&R for pavement design options.

Soft spots need to be repaired. Original PCC pavement could bridge the soft spot and this soft spot will show up after rubblization. Any depression one inch or greater in depth from the immediate surrounding area should be examined to see if it is due to poor underlying subgrade before the application of filler aggregate as required by the specification. A bearing capacity failure during rubblization could cause depression on one area and heave in other areas.

The repair of soft spots is necessary not only for a long term performance of the pavement but also for a good working platform for paving operations.

Pavements with delamination type cracks should not be rubblized. Horizontal cracks hinder the rubblization process by absorbing energy and decreasing the effective depth of rubblization.

Survey and set a new profile. Although rubblization does not significantly change the existing grade, simply specifying a few inches over the existing grade may not be adequate. The existing PCC pavement may not have adequate cross slope, or it is distressed due to poor subgrade which might have resulted in an irregular profile. Comparing survey results with the original geometry could provide clues on distress or subgrade conditions.

The design service life should be 20 years or more; this should be noted on the plans. Rubblization with an overlay is major reconstruction. It should not be used as a short-term fix.

9.7.5.2 SURFACE RECYCLING OF BITUMINOUS PAVEMENTS

Surface recycling is the process that either reworks or removes and replaces a limited portion, usually about one inch [25 mm], of the asphalt surface. The distresses addressable through this process include: raveling, flushing or bleeding, low surface friction, weathering, poor drainage profile, shallow rutting, minor corrugations and block cracking. It may also be used to correct problems in the profile grade line.

The major advantage for selecting this rehabilitation method is the minimal amount of work involved. There are two processes. The first is hot surface recycling involving heating, scarifying, remixing, and repaving recycled material. Other surface recycling methods include hot pavement removal using a heater-planer or cold milling using a rotary drum equipped with closely spaced carbide teeth.

The material to be reused has a rejuvenating agent and soft asphalt added to restore the pavement consistency, viscosity or penetration. DelDOT does not typically use this method for recycling surface courses.

9.7.5.3 IN-PLACE RECYCLING OF BITUMINOUS PAVEMENTS

In-place recycling is a process in which the pavement surface is ripped up or pulverized to a depth greater than 1 inch [25 mm]. The material is cold worked and reused as an aggregate base. The recycled material may be further strengthened by the addition of admixtures such as asphalt, lime, cement or fly ash. The recycled material will perform similar to new stabilized material upgrading the structural capacity, correcting surface distresses and mixture problems in the asphalt pavement, and correcting base course problems such as gradation, moisture and density. Correction of profile grade problems can be made with this material.

9.7.5.4 HOT-MIX RECYCLING OF BITUMINOUS PAVEMENTS

Hot-mix recycling is the process in which all or some of the pavement structure is removed, reduced to the required size, and mixed hot with added asphalt cement at a central plant.

This process is used to correct surface roughness, cracking, rutting, surface friction, raveling, inadequate structure, and inadequate traffic capacity. It should be remembered that the underlying cause for structural inadequacy would not be corrected using this process.

9.7.6 RESURFACING

The most common type of rehabilitation for existing pavements is resurfacing. Resurfacing can correct many common distresses and add additional strength to the pavement structure.

Problems encountered with overlay projects include inadequate thickness to correct surface problems when the problem is actually structural, inadequate repair of the deteriorated areas, unanticipated increasing traffic loadings, and not addressing reflective cracking.

9.7.6.1 TYPES OF OVERLAYS AND THEIR FUNCTIONS

Overlays can be of asphalt or Portland cement concrete. There are several variations of these overlay techniques, which are designed for specific applications.

The most commonly used overlay is dense-graded, hot-mixed asphalt concrete, which may be used on existing asphalt or Portland cement concrete pavements.

Portland cement concrete overlays are designed specifically for the type of existing

10.3.4.1 LATERAL OFFSET

A roadside barrier is normally placed as far from the traveled way as possible, while still maintaining the operating characteristics of the selected type of barrier. The greater the distance the better chance for the driver to recover control of the vehicle. In addition, some barrier installations may obscure a driver's sight distance if placed too close to the traveled way.

Placing the barrier at a uniform offset distance will not only be more aesthetically pleasing but provides the driver with a feeling of security and comfort when approaching a series of protected obstruction areas. The distance from the edge-of-traveled-way, beyond which a roadside object will not be perceived as an obstacle and result in the driver reducing speed or directing the vehicle away from the barrier, is called the shy line. This theoretical distance is different based on design speed and is shown in Table 5.5 in the *Roadside Design Guide*. For long continuous runs of barrier and barrier placed beyond the shoulder, the shy line offset criterion usually does not control barrier placement.

Another consideration in the lateral placement of barrier is the expected deflection of the system selected. The distance from the barrier to a rigid obstruction should not be greater than the dynamic deflection of the system based upon data from actual impact tests under controlled conditions, i.e. vehicle weight, speed and impact angle.

In embankment areas that must be protected, it is important that the width of embankment be sufficient to adequately support the posts to ensure proper operational characteristics of the barrier, see the Standard Construction Details.

There may be considerable deflection in barriers when impacted by a vehicle. Figure 10-2 shows the dynamic deflection of W-beam and concrete safety shape. If a roadside obstacle is too close to the back face of the rail or post, there may be danger that the rail will deflect all the way to the obstacle. Under these conditions, designing the post spacing closer than normal will reduce the potential deflection of the guardrail. If the obstacle is more than 3 ft [0.9 m] behind the back of post, a post spacing of 6 ft 3 in [1905 mm] should be used.

For obstacles located from 2 to 3 feet [0.6 to 0.9 m] behind the back of post, a post spacing of 3-ft 1-1/2 in [952 mm] should be used. If the obstacle is less than 2 ft [0.6 m], a rigid concrete barrier could be used. These deflection guidelines are based on having a proper end anchorage and posts installed in stable soil.

10.3.4.2 TERRIAN EFFECTS

A roadway's cross section is important element in a vehicle's performance when approaching or impacting a barrier. Barrier systems perform best when vehicles have all wheels on the surface and its suspension is in a normal position at the point of impact. The two common features of concern are curb and the approach slope. These features may cause a vehicle to vault over a barrier or strike the barrier too high or too low.

Vehicles striking curbs can change trajectory depending upon the size of vehicle, suspension characteristics of the vehicle, its impact speed and angle, and the height and shape of the curb. Impact testing has shown that the use of any guardrail/curb combination where high-speed, high-angle impacts is not an acceptable practice. If there is not other alternative, than the curb should be limited to 4 in. [100 mm] and the guardrail stiffened.

Vehicles traversing slopes steeper than 10:1, depending upon their impact angle and speed, may go over or impact the selected barrier too low. The *Roadside Design Guide* in section 5.6.22 and Figure 5.22 details the effect of slope rate on vehicle and barrier

have greater pedestrian use and volumes than residential streets. Collector streets are normally used by pedestrians to access public transit, commercial developments or other various land uses on the arterial to which they feed. Sidewalks should definitely be provided along developed frontages of arterial streets in land use zones that promote pedestrian activity.

Sidewalks should be considered whenever there is regular or periodic pedestrian travel along an existing roadside, street or transit corridor. Sidewalks should also be considered along any street, highway or transit corridor in developed areas not provided with shoulders even if existing pedestrian activity appears light.

10.8.6 DESIGN GUIDANCE FOR SAFE PEDESTRIAN CIRCULATION

Whenever a project is being initiated or planned where pedestrian movement exists or is anticipated, the initial scope and planning involved with the project should provide suitable space within the current or future right-of-way for safe pedestrian circulation. Sidewalks to be financed and maintained by others may also be included in a project.

When there are existing shoulders or walkways intended for pedestrian use, sidewalks should be evaluated at the project development stage for condition, suitable width, continuity, and limits. Where shoulders are being utilized for pedestrian movement, installing a parallel sidewalk may be considered because shoulders must be carried through intersections where turning lanes and pedestrian areas should not be combined.

Deteriorated sidewalks need to be evaluated for rehabilitation or reconstruction and additional width as necessary. Incomplete systems should be considered for connection to new sidewalks and end at logical terminations.

During the subdivision review process often dedicate right-of-way for transportation corridors and/or open space. The dedication of open space may often include areas close to a roadway edge, providing a buffer zone. County sidewalk policies provide for limited pedestrian circulation within a development. The maintenance of such pedestrian facilities becomes the responsibility of the development, local ordinance, adjacent property owner, or governing entity.

Pedestrian and sidewalk projects at intersections or along highway/transit corridors may include design, redesign, or traffic calming measures. These could include tightening of turn radii, channelized islands, medians and curbs, refuge islands, roundabouts, bulbs, neckdowns, signing, striping, transit shelters, and other features.

The extension of project limits beyond related highway or transit improvements for reasonable or short distances may be considered in order to include necessary pedestrian facilities at nearby intersections, provide safe access to public transportation facilities, or to avoid short sidewalk gaps. This decision can be reached anytime during the project development or design stage, and can be adjusted or deleted as needed. The project scope and funding may have to be revised accordingly.

10.8.7 PEDESTRIAN ACCIDENT HISTORY

An important factor in defining the need and locating and designing a sidewalk is accident history. Keep in mind that not all accidents result in a police accident report; supplement data from accident reports with field observations and discussions with stakeholders. The plan development process should include a study to define:

- Where and how did the accident occur: i.e. at an intersection or median, along the road, shoulder or existing sidewalk,

off the shoulder, or at a transit accommodation?

- Who was responsible?
- What was the pedestrian's origin and destination?
- What are the normal pedestrian movements in that area?
- Were there any existing pedestrian accommodations, lumination devices, warning signs, safety or traffic control devices, alcohol involvement or other contributing circumstances?

10.8.8 EXISTING SITE ACCOMMODATIONS

Typically included in the project development and design process is the assessment of the condition of the existing sidewalk network within the project and adjacent area. This assessment would include:

- The location of existing walkways, shoulders, worn paths, and greenway links;
- The location of incomplete walkway systems that adjoin or are within existing right-of way;
- The condition of existing facilities and how well they function or accommodate pedestrian movement;
- Any limiting geographical or architectural features that enhance or reduce feasibility of constructing pedestrian facilities;
- Any major or minor modifications in road or transit design that may enhance or reduce feasibility of constructing pedestrian facilities;
- Existing transit or depot stops with pedestrian or roadside amenities; and
- Whether surrounding or adjacent residential subdivisions, commercial or business land development, mixed land uses, or other developing land uses have provided or include a design that incorporates sidewalks and pedestrian amenities for school bus stops, greenway or walkway links to other land uses, and transit stop access for safe pedestrian movement and circulation.

10.8.9 PEDESTRIAN FACILITY LAYOUT

New and upgraded pedestrian facilities must conform to ADA standards; the information herein does not include all ADA standards. Contact DelDOT's ADA Coordinator and refer to DelDOT's *Standard Construction Details* for additional information.

Avoid warped surfaces within the pedestrian accessible route as much as possible. Warped surfaces can cause a wheelchair to become unstable by causing a wheel to lose contact with the ground.

The minimum vertical clearance for pedestrian space is 80 in [2 m]. Objects protruding into this space such as signs and utility boxes present a hazard for the visually impaired.

10.8.9.1 SIDEWALK REQUIREMENTS

For new sidewalks a minimum width of 5 ft [1.5 m], not including the width of the top of curb, is required. For new sidewalks at underpasses or overpasses, or where there is an obstruction for which the sidewalk must be narrowed, a minimum width of 4 ft [1.2 m] is permissible. If the sidewalk is narrower than 5 ft [1.5 m], then passing spaces at least 5 ft by 5 ft [1.5 m by 1.5 m] shall be provided at intervals not to exceed 200 ft [61 m]. Wider sidewalks may be preferred or required by local ordinance depending upon the volume and nature of pedestrian traffic. Minimum sidewalk thickness can vary according to materials, but be at least 4 in [100 mm] for Portland Cement Concrete. A minimum thickness of

6 in [150 mm] is required at entrance and driveway areas. The maximum cross slope is 2%, even where the sidewalk crosses a driveway entrance.

Small planting strips between the sidewalk and curb may not be practical unless the property owners, civic associations or volunteer programs can make provisions for maintenance. For increased user safety, sidewalks should be as far away from travel lanes as practical. Where possible a buffer width of at least 3 ft [0.9] m between the edge of a sidewalk and the edge of a shoulder, back of curb, or traveled way is preferred. A 3 foot [0.9 m] wide strip would improve safety, driver comfort, and provide an area for snow removal, signage and mailbox posts.

In central business districts, commercial areas, apartment complexes and generally where buildings or parking areas lie near or on the right-of-way, consideration should be given to pave the entire width from curb to building, property, or right-of-way line. The minimum desirable width of sidewalk between curb line and building face is 8 ft [2.4 m]. This permits space for utilities and other roadside appurtenances, and limited snow.

Standard material for any sidewalk or walkway is usually Portland Cement Concrete. However, sidewalk or walkway materials are not limited to Portland Cement Concrete. Upon approval and when funding is available, more aesthetic materials such as brick, asphalt, or other stable, firm, slip-resistant material surfaces may be used. This may be appropriate for traffic calming areas and in certain circumstances to address the concerns expressed by land use planners and/or communities that concrete sidewalks are aesthetically unpleasing.

When constructing, reconstructing, or extending sidewalks at or near intersections, the design should consider enhancing accommodations for pedestrians and vehicles throughout the intersection. Such elements may include refuge islands, separate pedestrian signal indications and phases (with pedestrian button in close proximity to the sidewalk) and offset room for traffic signs/poles and utilities.

In establishing the location of sidewalks, consideration will also be given to the need for or relocation of conflicting drainage facilities, side slopes, new traffic control devices and signing, intersection crossovers, striping, utility appurtenances, mailboxes with posts, and transit stops.

10.8.9.2 CURB RAMP REQUIREMENTS

Curb ramps shall be installed/upgraded for a project where the scope of work includes roadway reconstruction, resurfacing or any sidewalk construction. Examples of projects that do not require curb ramp installation are those that only consist of signing, pavement marking, lighting, and patching. For projects that fall into the qualifying categories, the following shall apply:

1. Curb ramps shall be provided where an accessible route crosses the curb line. An accessible route is a continuous unobstructed path, which includes sidewalks and other exterior pathways with an improved surface, intended for pedestrian use.
2. Curb ramps shall be provided at the termini of all existing and proposed crosswalks within the project limits if there is existing sidewalk or the project includes proposed sidewalk. Crosswalks may be marked or unmarked.
3. Curb ramps at marked crosswalks shall be placed within the crosswalk lines.
4. The curb ramp shall not be compromised by other highway features such as guardrail, utility poles, signs, drainage inlets, manholes or other obstacles. All options for working around obstacles must be considered. If an accessible route cannot be provided

around an obstacle, then the obstacle should be relocated. If the obstacle cannot be relocated then the Project Manager should work with his or her supervisor and assistant director to determine the best solution for the location. Curb ramps must be installed at all required locations when new sidewalk is constructed, existing sidewalk is reconstructed or a road is repaved. All curb ramp locations must be addressed and cannot be skipped when the road is repaved. Some obstacles may not be able to be relocated in order to make a pedestrian facility accessible, including structural elements such as bridge piers and buildings. In these limited cases where it may be technically infeasible to install a curb ramp at a specific location, the reason(s) must be documented.

5. Parking spaces shall not obstruct the accessible route associated with a curb ramp.

6. Positive drainage must be provided to keep water from ponding on curb ramps and in the street along the accessible route.

7. The accessible route shall be along a safe path for pedestrians. Curb ramps shall not be installed where it would be unsafe for any pedestrian to cross due to conditions such as uncontrolled intersections with high vehicle speeds, unsafe sight distance, or physical obstacles that cannot be crossed such as median barriers on multi-lane highways. If it is determined that a safe crossing cannot be provided, then pedestrians shall be routed to a safe, accessible crossing, and the unsafe location shall be constructed with a barrier that will direct pedestrians (including the visually impaired) to a safer crossing. Examples of barriers that could be used at unsafe locations include shrubbery, a grass strip, railings, bollards, a double-posted informational sign with two horizontal struts (one between 36 to 42 inches [0.9 to 1.1 m] and the other between 4 to 12 inches [0.1 to 0.3 m] above the ground), etc. Whatever is used would have to consider detectability by the visually impaired as well as safe roadside design considerations for the clear zone. Locations thought to be unsafe shall be reviewed by DelDOT's Traffic Section. If the crossing cannot be made safe then the Project Manager should work with his or her supervisor and assistant director to determine the best solution for the location and document the reason(s).

8. The designer needs to consider how curb ramp locations may negatively affect the operation and capacity of vehicular traffic.

Diagonal curb ramps (those placed along the curb radius, DelDOT Type 3) are not preferred when pedestrians will have to change direction in the roadway in order to complete their crossing. While it is recognized that there will be locations where they are the best option (such as at intersections with large turning radii), perpendicular curb ramps (DelDOT Types 2 and 4) are preferred. Diagonal curb ramps are often a good treatment when there are channelizing islands at right turn slip lanes. When diagonal curb ramps are used, a 5 ft [1.5 m] long bottom landing area must be provided between the curb radius and the curb line extensions, which is outside the direction of vehicle travel. If this clear distance cannot be provided, diagonal curb ramps shall not be used and another type of curb ramp must be provided. This clear distance is not necessary for right turn slip lane and channelizing island configurations or where there is a landing at the bottom of the ramp within the sidewalk.

The maximum running slope of curb ramps is 12:1. Flatter slopes should be provided where possible. Curb ramps in alterations where a 12:1 slope is not technically feasible may have a maximum running slope of 10:1 for a maximum rise of 6 inches [150 mm].

The maximum cross slope of a curb ramp is 50:1 (2%).

DelDOT's minimum curb ramp width is 60 inches [1.5m].

Where a 12:1 maximum curb ramp slope will not meet the sidewalk grade within a length of 15 feet [4570 mm] due to a steep adjacent roadway, the last ramped section transitioning back to existing pedestrian facilities may be limited to 15 feet [4570 mm], and the slope of that section allowed to exceed 12:1.

If turning or maneuvering is required within the curb ramp or landing area, a maximum slope of 50:1 (2%) in any direction must be provided. The minimum landing area dimension shall be 60 inches [1525 mm] x 60 inches [1525 mm].

Transitions from curb ramp to gutter and streets should be flush and free of level changes. There should not be a bump at the gutter flow line or where the curb meets the pavement. Changes in level up to ¼ inch [6 mm] may be vertical. Changes in level between ¼ inch [6 mm] and ½ inch [12 mm] must be sloped no greater than 2:1. Changes in level above ½ inch [12 mm] must be treated as a ramp and must be sloped at 12:1 or flatter.

Given the constraints and varying field conditions, each curb ramp must be evaluated and designed using these guidelines and the *Standard Construction Details*.

10.8.9.3 DETECTABLE WARNING REQUIREMENTS

Detectable warnings shall be installed at sidewalk curb ramps and at uncurbed sidewalks at the following locations:

- Crosswalks (marked & unmarked) and designated places where pedestrians cross public roadways (including medians and refuge islands);
- Signalized entrances;
- High volume entrances with ADT greater than about 400;
- Entrances with an operating speed of 25 m.p.h. or greater through the pedestrian area;
- Railroad crossings.

Detectable warnings may also be installed in other areas when determined necessary by engineering judgment. Factors which present a potentially hazardous situation may also be considered, including blind spots, complicated turning movements or other situations in which pedestrians with visual impairments should be signaled to stop. They should not be used at all entrances without consideration of the above criteria, since overuse can cause confusion for pedestrians with visual impairments.

A 24 inch [600 mm] long strip of detectable warnings (truncated domes) must be placed along the fully depressed width of the curb ramp at the transition to the street. Detectable warnings must also be placed at cut-through medians and cut-through islands.

10.8.10 MAINTENANCE RESPONSIBILITY

As a policy, DelDOT does not normally maintain sidewalks. Depending upon their location, applicable laws, local ordinances and the current town or city maintenance agreement, sidewalks are the maintenance and upgrade responsibilities of the property owner, homeowner's association, municipality, town, city, suburban or non-suburban area, incorporated or unincorporated area, or governing body which bears jurisdiction. Delaware Transportation Corporation usually contracts out maintenance of sidewalks and other passenger amenities immediately adjacent to transit facilities.

Projects involving pedestrian or sidewalk amenities proposed under the Transportation Enhancement program have a formal agreement with the responsible party that includes a description of maintenance standards to be upheld and assigns the responsibility for the associated costs for those amenities. Maintaining a pedestrian or sidewalk facility involves several items including snow removal, trash and debris removal, control of vegetation, reconstruction, graffiti removal, resigning or re-striping (specifically related to the sidewalk), avoiding general neglect and deterioration for whatever cause and alterations of the surface or subsurface level required to improve the appearance.

Projects under the Suburban Streets and Resurfacing Program may include repairs, replacement or general maintenance to existing deteriorated sidewalks. For all new construction and replacement, accessibility guidelines and standards are to be followed.

DelDOT repairs or replaces any existing sidewalk surface that has been damaged or altered by DelDOT forces or its contractors. Repair or replacement of sidewalks follows all accessibility guidelines and standards.

10.8.11 REMINDERS

When sidewalks are proposed or initiated without a formal agreement to the contrary, DelDOT may not assume any maintenance responsibility.

Although not to be addressed as part of the project, it may be beneficial to consider the need for future sidewalk and reserve the right-of-way.

Delaware's Department of Education does have busing rules affecting the busing rights for children. These may come into play if certain communities and schools are connected with continuous sidewalk access. The most significant affect is that public busing privileges may be revoked.

Local governments or jurisdictions may adopt land use or subdivision ordinance regulations to protect transportation facilities, corridors, and sites for their identified functions. This could include, but is not limited to, facilities providing safe and convenient pedestrian or bicycle access within and from new subdivisions, planned unit developments, transit stops, greenways, and neighborhood activity centers such as schools, parks, and shopping areas.

If not required under county, city, or local jurisdiction, DelDOT may request or require sidewalks and pedestrian facilities to be installed prior to subdivision entrance permit approval. DelDOT may also request the installation of sidewalks and pedestrian facilities along roadway or transit corridors as part of the mitigation plan under a traffic impact study.

As a part of improving the transportation network, DelDOT initiates and designs sidewalk projects to connect existing and future transit routes, transit facilities, park-and-ride lots, public/private parking areas, bus stops for schools, businesses, shopping centers, industrial parks, residential communities, or any other public or private institution. This enhances multi-modalism while decreasing vehicular traffic and automobile emissions.

10.8.12 FUNDING ALTERNATIVES

Although specially defined on the Project Initiation Form, the designer should be aware that the construction, reconstruction, or extension of sidewalks can be funded by several different methods and funding sources.

Projects initiated by DelDOT are usually 100% funded by the Department and include:

- Removal and replacement of existing sidewalk caused by the construction, reconstruction, improvement, or extension of any highway, transit,

safety, or pedestrian related improvement;

- Projects facilitating State or DelDOT transportation purposes. This may include, but is not limited to, improving or expanding transit facilities, walkway/pedestrian corridors, greenway links, or safety improvement projects.

Projects initiated by others for design and/or construction by DelDOT have various matching fund ratios. These include:

- Projects may be 100% funded under the Suburban Streets and Resurfacing Program for new sidewalk projects that have been approved and initiated by DelDOT in recognition of meeting the needs of the public, town officials, or other governing bodies.
- Projects may be funded and initiated by a school district or other agency with a 50% match by DelDOT. These would involve new or reconstructed sidewalks within a project's termini or short distances outside a project area to connect sidewalks to existing pedestrian or transit generators from or to educational facilities.
- Projects may be 100% funded and initiated by a party or agency for removal and replacement of deteriorated sidewalk.
- Projects may be 50% jointly funded by a party or agency for utility adjustments, drainage, signals, pedestrian barriers, retaining walls, crossovers, etc. required solely for sidewalk safety and enhancement. This does not include projects initiated for ADA conformance.
- Projects may be 10% funded by an initiating party or agency, 10% by DelDOT and 80% funded by the Federal Highway Administration. These projects meet the criteria for funding under the federal Intermodal Transportation Efficiency Act. Qualified applicants can initiate projects to be included in DelDOT 's Transportation Enhancement Program. These projects must first gain approval from DelDOT's Technical Advisory Board. The projects are then subject to final review and approval by the Secretary of Transportation.

10.9 BICYCLE FACILITIES

There is a wide range of facility improvements that can enhance bicycle transportation. Suitable accommodations for bicyclists shall be determined as part of the DelDOT's Project Development Process and scoped as part of the project handoff prior to design initiation. Improvements can be simple involving minimal design considerations such as changing drainage grate inlets, or they can involve a detailed design such as providing a bike path.

Facility improvements for motor vehicles through appropriate planning and design can enhance bicycle travel or should at least avoid adverse impacts on cycling. Unless access is specifically denied, some level of bicycle use can be anticipated on most roadways. All new roadways, except those where bicyclists will be legally prohibited, should be designed and constructed under the assumption that they will be used by bicyclists. Guidelines are presented here to help design and construct roadway improvements and separate facilities that accommodate the operating characteristics of bicycles. Additional information including signing layouts, striping, and design details can be found in AASHTO's *Guide for the Development of Bicycle Facilities.*

Because most highways have not been designed with bicycle travel in mind, there are often many ways in which roadways should be improved to more safely accommodate bicycle traffic. Roadway conditions should be examined and, where necessary, safe drainage inlets, safe railroad crossings and smooth pavements should be provided. Drainage inlets and utility covers are potential problems to bicyclists. When

designing a new roadway, these types of appurtenances should be kept out of the bicyclists' expected path.

Parallel bar drainage inlet grates can trap the front wheel of a bicycle causing loss of steering control. Often, the bar spacing is such that it allows narrow bicycle wheels to drop into the grates, resulting in irreparable damage to the bicycle wheel and frame and/or injury to the bicyclist who could be thrown from the bicycle. These grates should be replaced with the bicycle-safe grates in the DelDOT's *Standard Construction Details*. Parallel bar grates are not to be used where bicycles may be present.

Railroad-highway grade crossings should ideally be at a right angle to the rails. The more the crossing deviates from this ideal crossing angle, the greater is the potential for bicyclists' front wheels to be trapped in the flange way causing loss of steering control. It is also important that the roadway approach be at the same elevation as the rails.

Consideration should be given to the materials of the crossing surface and to the flange way depth and width. If the crossing angle is less than approximately 45 degrees, consideration should be given to widening the outside lane, shoulder, or bicycle lane to allow bicyclists adequate room to cross the tracks at a right angle.

Pavement surface irregularities can do more than cause an unpleasant ride. Gaps between pavement slabs or drop-offs at overlays parallel to the direction of travel can trap a bicycle wheel and cause loss of control; holes and bumps can cause bicyclists to swerve into the path of motor vehicle traffic. Thus, to the extent practicable, pavement surfaces should be free of irregularities and the edge of the pavement should be uniform in width. On older pavements it may be necessary to fill joints, adjust utility covers or, in extreme cases, overlay the pavement to make it suitable for bicycling. Longitudinal joints in pavement and between pavement and gutter pans should not be more than ½ in [12 mm] wide. Longitudinal drop-offs between pavement and gutter pans or between travel lane pavement and shoulder pavement should not exceed ¾ in [18 mm]. Ridges used to create "rumble strips" in pavements should not be more than ¾ in [18 mm] when perpendicular to bicycle travel. Properly located warning signs should precede these locations.

Roadway treatments intended to accommodate bicycle use must address the needs of both experienced and less experienced riders. One solution to this challenge is to develop the concept of a "design cyclist" and adopt a classification system for bicycle users such as the following:

Group A-Advanced Bicyclists - Experienced riders who can operate under most traffic conditions. They comprise the majority of the current bicycle users of collector and arterial streets.

Group B-Basic Bicyclists - Casual or new adult and teenage riders who are less confident of their ability to operate in traffic without special provisions for bicyclists. The basic rider is comfortable riding on neighborhood streets and shared use paths but prefer designated facilities such as bike lanes or wide shoulder lanes on busier streets.

Group C-Children - Pre-teen riders whose roadway use is initially monitored by parents. Eventually they are accorded independent access to the system. Provisions should be made to allow access to key destinations without encouraging them to ride in the travel lane of major arterials.

In the design of bicycle facilities two broad classes of bicyclists are used: Group A riders and Group B/C riders.

Generally, the Group A bicyclist will be best served by designing all roadways to

accommodate shared use by bicyclists and motor vehicles. This can be accomplished by:

- Providing wide outside lanes or bike lanes on collector and arterial streets built with an urban section (i.e., with curb and gutter);
- Providing useable shoulders on highways built with a rural section (i.e., no curb and gutter).

Group B/C bicyclists are best served by a network of neighborhood streets and designated bicycle facilities that can be provided by:

- A network of designated bicycle facilities (e.g., bike lanes, separate bike paths, or side-street bicycle routes) through the key travel corridors typically served by arterial and collector streets.
- Useable roadway shoulders on rural highways, 4 ft [1.2 m] wide.

10.9.1 FACILITY SELECTION

The most significant factors affecting compatibility of roadways for bicycling are motor vehicle traffic volumes, operating speed, and the width of the travel lane and shoulder. The selection of facility type should be determined by an analysis of these factors in addition to the following:

1. State and local bicycle master plans;
2. Proximity of schools, parks and other destinations where a child bicyclist may be expected;
3. Presence of a regionally significant or locally designated bicycle route;
4. Potential turning movement conflicts; and
5. Environmental, historical and right-of-way constraints.

In general, additional travel lane or shoulder width can increase the suitability of a roadway for bicycling. Designation of bicycle lanes with appropriate signs and pavement markings will help increase the predictability of both bicycle and motor vehicle movements. Additional separation of bicycle traffic from motor vehicle traffic on shared use paths may be desirable on high speed, high volume roadways, where an increase in child bicyclists can be expected or along regional pathway networks. Development of a shared use path does not preclude the need to accommodate more experienced bicyclist on the roadway.

10.9.2 FACILITY TYPES

Four basic types of facilities are used to accommodate bicyclists:

1. Shared Roadway (No Bikeway Designation) - Shared lanes are streets and highways with no special provision for bicyclists. Shared lanes typically feature 12 ft (3.6 m) lane widths or less with no shoulders, allowing cars to safely pass bicyclists only by crossing the centerline where permissible or moving into another traffic lane. In residential areas with lane widths of at least 12 ft [3.6 m] low motor vehicle traffic volumes and average motor vehicle speeds of less than 30 mph [50 km/h], shared lanes will accommodate group A riders, and will normally be adequate for group B/C bicyclists. Where the existing lane width is less than 12 ft [3.6 m], additional lane width is called for. For higher speeds and traffic volumes, shared lanes become less attractive routes, especially to group B/C riders.
2. Shared Roadway, Signed - These roadways are designated by bike route signs and either provide continuity to other bicycle facilities or designate a preferred route through high demand corridors. In designating a roadway as a shared roadway, DelDOT must assure there are advantages to using this route, the route is suitable, and it will be

Delaware Department of Transportation

Road Design Manual

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Chapter One

ROAD DESIGN MANUAL INTRODUCTION

The mission of the Delaware Department of Transportation (DelDOT) is to provide a safe, efficient and environmentally sensitive transportation network that offers a variety of convenient, cost-effective choices for the movement of people and goods.

In meeting this mission, one goal of DelDOT is to plan, design and implement projects that serve existing and future transportation needs, including alternative modes that are safe, have a long service life with low maintenance costs. In addition, projects have to satisfy the purpose and needs as perceived by the driver, other users, and the community. To accomplish this, the designer must adopt design criteria that are in harmony with the community and preserve environmental, scenic, aesthetic, historic and natural resource values of the area. Current design standards allow and suggest that there is flexibility to select design guidelines that will help to deliver a "context sensitive" design.

Projects for making improvements to the transportation network are the result of an annual Capital Transportation Program (CTP). The CTP establishes priorities for new projects, continues funding for ongoing projects, and establishes new initiatives to meet changing transportation needs. The CTP is developed based on the State's Long-Range Transportation Plan and the Department's Project Development and Implementation Process (Pipeline Process). The Long-Range Plan provides for managing and developing a transportation system that supports county and local government comprehensive plans. The Department's Pipeline Process further defines, assesses, categorizes and prioritizes projects for inclusion in the CTP.

Projects from the Pipeline Process that are deemed eligible for the CTP are submitted to the Project Development Committee (PDC) which consists of Division Directors and their support staff. The PDC prioritizes projects for further refinement of their scope of work, costs, complexity of design, staffing needs and scheduling requirements. After completion of these tasks, projects are resubmitted to the PDC for approval and inclusion in the recommended Transportation Improvement Program. The approved projects make up the annual Capital Transportation Program (CTP).

Upon approval of the CTP, the Project Development Committee formally initiates and assigns projects to the section responsible for either beginning the design process or further project development. Many projects approved through the CTP process require some level of design. Each project design begins with understanding the purpose and need for the project. The design process recognizes that each project is unique and has its own community values, social, economic, and environmental concerns and constraints. The design process identifies and addresses these elements through a collaborative approach in which the public is a part of the design team. A successful design must balance many factors that will result in a safe facility with features that meet the expectations of the user and community.

Factors to be recognized and addressed in the design process include:

- Selecting design guidelines that will provide for the safety of the user.
- Identifying the need for access to the facility, as well as mobility along the facility.
- Preservation or enhancement of historic sites and districts.
- Protection or enhancement of existing environmental assets.
- Identifying the economic needs of and impacts to the affected community or area.
- Developing an understanding of the social context of the community and area within which the facility exists.

Ensuring that a project design will have a balance of these factors is the result of a continuous and meaningful public involvement process. Throughout the public involvement process, the designer must make sure the purpose for the project as established in the project initiation is fulfilled while understanding and addressing the needs of the community. By doing this the introduction of new or additional issues during the final design phase that may result in delays and/or redesign will be minimized.

The basic design tools available to the designer are this manual and the 2004 American Association of State Highway Transportation Officials, (AASHTO) "*A Policy on Geometric Design of Highways and Streets*" commonly referred to as the "Green Book". In addition designers need to refer to other related publications and guidelines prepared by the Federal Highway Administration (FHWA), the Transportation Board (TRB), and other recognized experts in the transportation field. The principal publications are:

- AASHTO's *Roadside Design Guide*,
- AASHTO'S *Context Sensitive Design for Integrating Highway and Street Projects with the Community and the Environment*,
- Transportation Research Board's (TRB) *Highway Capacity Manual*,
- TRB's *Special Report 214 Designing Safer Roads*,
- FHWA's *Manual on Uniform Traffic Control Devices*,
- FHWA's *Flexibility in Highway Design*, and
- DelDOT's *Traffic Calming Manual.*

In addition, there are numerous other documents, particularly related to highway drainage and intersection design that must be referenced in designing a project.

Using these documents, the Road Design Manual was developed with emphasis on standards and practices that have proven to be successful in this state. The flexibility to design a project that will meet the expectations of the user, the community and Department exists in the Green Book and in the standards found in this manual. Published design standards have a measure of flexibility, usually stating a maximum and minimum value. Many of these values are empirically based using mathematical modeling techniques with assumed roadway surface conditions, driver reaction times and adverse weather conditions.

Designers need to recognize that there is a difference between the strict application of design standards found in the tables and charts versus providing consistency in design. The design should ensure there is consistency in application of the standards that allows the driver to react in a consistent and predictable manner when encountering similar roadway conditions. However, in responding to the many issues that arise on each project, there is a need for flexibility in

selecting design features while maintaining operational consistency and user safety.

Design features that fall outside normal design criteria and accepted practice are to be determined using sound engineering judgment and should be thoroughly documented. Liability is a potential concern when design standards are not met. The key to minimizing liability is documentation of the rationale for major design decisions in terms of safety, capacity, route compatibility, project intent, time to construct, construction costs, and environmental, historic, and aesthetic considerations. Documentation is particularly valuable on projects having extensive public involvement where participants frequently change. Information on previously studied issues can be rapidly disseminated reducing duplication of effort.

The steps for selecting each project's design criteria have historically remained fundamentally the same. At the earliest stages of a project the designer needs to determine and gather information concerning the following:

1. The functional classification of the roadway—who is it to serve and for what purpose?

2. Specific design controls such as design speed, predominant type of vehicle anticipated to use the facility, future traffic volumes, proposed level of service, access needs and other modes of transportation to be served.

3. The elements of geometric design such as stopping and passing sight distance, horizontal alignment, vertical alignment and other elements that affect the geometric design.

4. The proposed physical cross sectional elements, such as: lane widths, shoulder widths, pavement types, lateral clear zones, use of curbs, median width, median treatment and type of drainage systems.

Selecting the appropriate design speed has one of the most significant impacts on a project's design parameters. The selected design speed considers the topography, adjacent land use, the type of facility, available funding, projected traffic volume, acceptable level of service, traffic composition, anticipated operating speed, anticipated posted speed, the expected 85th percentile speed and other constraints that exist on most projects. Most of the tables and charts that give guidance in selecting critical design elements are based on design speed. In the context sensitive design environment, it is recognized that there are limitations, constraints, community values, and other factors that require the designer to look beyond the full standards found in these tables and charts for a workable solution. The selected criteria should properly reflect driver safety, desires, expectations, comfort and convenience.

The design process also allows for recognition of areas of increased residential and commercial growth and density. These areas have a design environment that is not only sensitive to the safety of the reasonable and prudent driver but also the bicyclist, pedestrian, rollerblader and elderly as well as the general well being of the community. If a road's existing functional classification does not recognize these users then the designer needs to address them in the design process.

In areas with a highly diverse user population, one option for creating a safe roadway may be selecting design features that encourage drivers to operate at lower speeds, change driver behavior, and improve the environment for non-motorized uses. This concept is referred to as "traffic calming." The Department's *Traffic Calming Design Manual* should be consulted before introducing traffic calming features since the designer should consider the many possible consequences of their use. When properly

applied, with community acceptance, traffic calming is an enhancement to a community's overall living environment.

Selecting the right combination of design guidelines that will control the designer's work tasks is very complex, requiring flexibility and the use of good judgment. The final design must meet the needs and expectations of the Department and community, as well as providing for the user's safety.

Chapter Two

Design Controls

Plans for roadway improvement projects are based on established geometric design standards for the various elements that constitute a roadway. These elements include roadway width, side slopes, curvature, and gradients. Decisions on appropriate geometric standards are influenced by the characteristics of the specific highway section to be improved. Each project area has its own unique characteristics. Many of these are identified through the project development process, included in the Project Initiation and further defined in the project scope of work. These items need to be made a part of determining a project's design controls and are evaluated along with such elements as:

- The social context of the communities and area within the affected project area. What are the perceived goals for creating a more livable community?
- Will the design provide the expected level of service and safety of the users?
- What is the existing and future traffic volume?
- What is the anticipated operating speed?
- What are the terrain features?
- What are the needs for access and mobility along the facility?
- Will the selected design controls protect or enhance the natural environment?
- Are there wetlands, historic sites and districts or other environmentally sensitive features that need to be preserved?
- What are future goals of the transportation network?
- What are the economic needs of the affected communities or area?

These characteristics serve as a basis for selecting design standards. Chapter Three-Design Standards presents the specific geometric design standards adopted by the Delaware Department of Transportation (DelDOT).

2.1 OBJECTIVES OF DESIGN CONTROLS

Four basic objectives should be kept in mind:

- Level of Service. The design should provide a level of service appropriate for the characteristics of the traffic that will be using the facility.
- Safety. The completed facility should present motorists with a safe environment. Special consideration should be directed to avoiding potentially hazardous situations.
- Economics. The cost-effectiveness of proposed improvements must be considered. Improvement standards higher than appropriate for a particular facility may result in increased

facility may result in increased expenditures that could be spent more effectively by improving additional road sections.

- Context. Design features should be selected that are in balance with the social context of the community and surrounding area. This is accomplished by gathering and including information from the public throughout the design process. A context sensitive design advances the objectives of safety, mobility, enhancement of the natural environment, and preservation of community values. Projects that improve the livability of the community or quality of the natural environment are considered context sensitive.

2.2 LEVELS OF SERVICE

In general terms, the level of service of a highway facility may be influenced by many factors, including surface condition and ride-ability. From the standpoint of design controls, the level of service is principally related to the ease and convenience with which the highway facility can serve the expected volumes of traffic.

The Transportation Research Board's (TRB) *Highway Capacity Manual* presents a thorough discussion of the level of service concept. Six levels of service are established from level A (the highest) through level F (the lowest).

The general characteristics of the various levels of service are:

- Level of Service A – free-flowing traffic; users virtually unaffected by other traffic, able to select desired speeds and maneuver unrestricted.
- Level of Service B – reasonably free traffic flow; users able to select desired speeds, but with a slight decline in freedom to maneuver.
- Level of Service C – stable flow, but operation of individual users is significantly affected by traffic; ability to select speeds is reduced and maneuvering requires substantial vigilance by the users.
- Level of Service D – high density approaching unstable flow. Speeds and freedom to maneuver are severely restricted. Small increases in traffic flow will generally cause operational problems.
- Level of Service E – operating conditions at or near capacity with unstable flow. All speeds at a low and relatively uniform value. Freedom to maneuver is extremely difficult.
- Level of Service F – forced or breakdown flow. Traffic exceeds capacity causing queues with stop-and-go waves, and operations are extremely unstable.

The traffic flow rates that can be served at each level are termed "service flow rates." Once a level of service has been identified as applicable for design, the accompanying service volume logically becomes the design service flow rate, implying that if the traffic volume using the facility exceeds that amount, operating conditions will fall below the level of service for which the facility was de-signed. A guide for selecting design levels of service is shown in Figure 2-1.

More detailed guidelines for selecting appropriate levels of service are given in AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the "Green Book") and TRB's *Highway Capacity Manual* (HCM).

Figure 2-1
Guidelines for Selection of Design Levels of Service

Highway Type	Type of Area and Appropriate Design Level of Service		
	Rural Level	Rural Rolling	Urban and Suburban
Freeway	B	B	C
Arterial	B	B	C
Collector	C	C	D
Local	D	D	D

2.3 SPEED-RELATED CONTROLS

Many design decisions are controlled by the expected speed of vehicles on the facility, particularly decisions related to required sight distance and maximum permissible curvature. Geometric requirements normally will be less stringent with lower speeds in difficult terrain and in urban areas. The design objective is to provide a facility serving the users' needs in a safe and economical manner. The speed on a facility is related to many factors such as the physical characteristics of the roadway, the amount of roadside access and activity, the weather, the volume of traffic, and legally established speed limitations. Roadways should be designed to allow most drivers to operate at their desired speed under normal weather conditions.

There are three terms used to describe a vehicle's speed when using the roadway: (1) operating speed, (2) running speed, and (3) design speed. Each of these either directly or indirectly plays a role in the design process. Operating speed is used to measure and study the overall efficiency of the design (attained level of service), individual roadway sections and selected roadway features. Running speed is a mathematical tool used to develop road user costs and level of service. Design speed is used in selecting design criteria, guidelines and other elements that will control the design. The process of selecting a design speed is found in Chapter Three-Design Standards.

2.3.1 OPERATING SPEED

AASHTO defines operating speed as "the speed at which drivers are observed operating their vehicles during free flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric feature."

2.3.2 RUNNING SPEED

Running speed is defined as the speed at which an individual vehicle travels over a roadway section. Running speed is mathematically determined by dividing the length of roadway being studied by the running time required for the vehicle to travel through the section. The average running speed of all vehicles is used to evaluate service levels and road user costs. The average running speed varies during the day depending upon the traffic volume. Peak and off-peak values are used in operation studies and in design; average running speeds for an entire day are used in user cost and other economic analysis.

2.3.3 DESIGN SPEED

For each proposed roadway, a design speed is selected to determine various geometric design features such as curvature, superelevation rate, sight distance, and critical length of grades. The design speed should not be less than the anticipated normal operating speed and at least 5 mph [10 km/h] above the posted speed.

Some design features, such as curvature, superelevation, and sight distance, are directly related to design speed. Other features, such as lane widths, the inclusion of shoulders, shoulder widths, and clearances to obstacles, are not directly related to design speed but have a significant affect on driver speed. Drivers react to the physical limitations and traffic rather than

the importance of the facility. When a change in design speed is made, many elements of the highway design will change accordingly.

The selected design speed should be logical with respect to the characteristics of the terrain, adjacent land use, and functional classification. A highway in level terrain may justify a higher design speed than one in rolling terrain. A highway in lightly developed or undeveloped (open) areas may justify a higher design speed than in a developed area. The selected design speed should be consistent with the speeds that drivers are likely to expect on a given highway facility.

A highway carrying a high volume of traffic may justify a higher design speed than a lower volume facility or a facility that is of a lower functional category with similar topography. But a low design speed should not be assumed for a low volume road where the topography is such that drivers are likely to travel at high speeds. This could lead to selecting design criteria resulting in unsafe horizontal and vertical geometry for the user.

Except for local streets where speed controls features are included intentionally, every effort should be made to use a design speed as high as practicable to obtain a desired degree of safety, mobility, and efficiency. In meeting this objective the constraints of environmental quality, economics, aesthetics, and the social context of the impacted areas must be identified and addressed. Above minimum design values should be used where feasible, but the designer needs to recognize that project constraints may lead to selecting a practical value. Selecting higher or lower values that do not reflect the driver's travel desires, habits, and expectations is not consistent with producing a balanced project meeting all the associated constraints imposed on most projects. The design speed selected should be inclusive of the typically desired speed of the highest-percentile of drivers.

It is necessary to recognize conditions where vehicle speeds typically may exceed the design speed. For example, terrain conditions may limit the overall design speed of a roadway section to 50 mph [80 km/h], but several long tangents within the section may encourage much higher speeds. This situation should be recognized, and the curves at each end of the tangent should be somewhat flatter than minimum standards for 50 mph [80 km/h] to permit a vehicle's safe transition back to the design speed.

Similarly, if vertical curves on tangent sections are designed based on the 50 mph [80 km/h] stopping sight distance criteria, a danger exists when actual operating speeds may exceed 50 mph [80 km/h]. Designers should anticipate locations where operating speeds may exceed design speeds for certain situations, and should avoid applying minimum levels of geometric standards in these locations.

2.4 TRAFFIC-RELATED CONTROLS

The characteristics of the traffic expected on a particular facility are significant factors in establishing many of the design controls for a project. The primary traffic characteristics affecting design are volumes, the directional distribution, the composition, and the future projections for each of these elements.

2.4.1 TRAFFIC VOLUMES

Traffic volumes are expressed in several different ways:

- Average Annual Daily Traffic (AADT). The total yearly volume divided by the number of days in the year.
- Average Daily Traffic (ADT). The total volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
- Peak-Hour Traffic (PHT). The traffic volume during an interval shorter than a

day, usually one hour, that reflects the frequently repeated rush-hour periods.

- Design Hourly Volume (DHV). The peak hourly volume expected in the 30th highest hour during the chosen design year. This helps give a better picture of unusual seasonal fluctuations or other conditions that may cause capacity problems.
- Directional Distribution (D). A measure of the highest traffic volume in one direction during peak hours, expressed as a percentage of the DHV. This is important principally with relatively high traffic volumes where capacity is being approached.

Information on traffic data, including the recommended number of lanes, is available from the Division of Planning and must be obtained by the designer before starting detailed design of new projects.

Normally, the ADT shown on the title sheet for each project is actually the AADT. On some roads, such as those serving the beach areas, traffic is significantly heavier in one season. For these roads, the design should take into consideration the ADT for the peak months.

2.4.2 TRAFFIC COMPOSITION

Vehicles of different sizes and weights have different operating characteristics, which must be considered in highway design. Traffic composition is a measure of the proportion of heavy trucks in the traffic stream, expressed as a percent of the DHV.

Besides being heavier, large trucks generally are slower and occupy more roadway space than passenger cars. The overall effect of one truck on traffic operations is often equivalent to several passenger cars. Traffic composition is an important factor in determining a facility's ability to carry existing and future traffic volumes, geometrics, cross-sectional elements and the structural design of the pavement needed to withstand the traffic loads for the projected service life of the project.

2.4.3 TRAFFIC PROJECTIONS

For all projects involving new construction or major reconstruction, the design controls normally will be based on the traffic volumes estimated for 20 years in the future, expressed as either ADT or DHV.

For projects where the scope of work is limited to resurfacing, restoration, rehabilitation, and reconstruction improvements, the capacity should be checked against the projected traffic volumes for the forecast year (normally not to exceed 10 years). The forecast year traffic should serve as a design control for geometric standards.

2.4.4 TRAFFIC DATA DOCUMENTATION

The design traffic data to be shown on the title sheet of the plans are:

- ADT Current – (current year specified),
- ADT Projected – (future year specified),
- DHV Projected – (year specified),
- Design Speed — mph [km/h],
- % Trucks – and,
- Direction of Distribution % – in predominant direction.

All of the above information (except design speed) is obtained from the Division of Planning. In addition, the Division of Planning provides current vehicle types and truck patterns common to the project. The designer, based on criteria set forth in this manual and the Green Book, determines the design speed for projects.

2.4.5 HIGHWAY CAPACITY

The term "capacity" is used to express the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point, such as a uniform section of a lane or a roadway, during a given time period under prevailing roadway and traffic conditions.

As part of the traffic data furnished by the Division of Planning, there will be a recommendation on the number of lanes to meet the projected traffic demands. However, the designer needs to be familiar with highway capacity procedures and aware that as a project is developed the original recommendation may need to be reevaluated. Reference is made to the TRB's *Highway Capacity Manual* for technical details and instructions for capacity analyses. Designers should be aware of several general conditions that may warrant a detailed capacity analysis:

- Two-lane roadway sections that exceed 1,400 passenger vehicles per hour (total two-way) should be investigated to see if more lanes or other capacity improvements are necessary.
- The capacity of a two-lane road is greatly reduced where there is limited opportunity for passing slower-moving vehicles. In terrain where it is impracticable to provide adequate passing sight distance, additional lanes or useable shoulder areas may be required.
- The capacity of a two-lane road is significantly affected by long, steep grades with slow-moving trucks. An auxiliary climbing lane may be warranted.
- The lack of shoulders affects vehicle speeds indirectly leading to a decrease in capacity.
- Intersection signalization and frequency may change capacity requiring additional lanes.
- Urban and industrialized areas usually have frequent access points creating roadside conflicts. These areas may need to be studied for their affect on highway capacity.

In the event one or more of these conditions should occur, the designer should work with the Division of Planning and determine its significance. Increasing capacity is not only costly but may be in conflict with the approved scope of work and project intent. Any attempt or request to analyze or justify this type of change should receive approval prior to expending any effort.

Two other key items to compare throughout the design process, particularly at intersections, is the design-hour volume (DHV) versus the service volume (selected design level of service). If the service volume is not equal to or greater than the expected design-hour volume capacity, problems can be expected and other design alternatives need to be considered.

2.4.6 DESIGN VEHICLES

Where turning movements are involved, the geometric design requirements are affected significantly by the types of vehicles using the facility.

Four general classes of vehicles are identified: (1) passenger cars, (2) buses, (3) trucks, and (4) recreational vehicles. The passenger car class includes cars of all sizes, spot/utility vehicles, minivans, vans, and pick-up trucks. Buses include inter-city (motor coaches), city transit, school, and articulated buses. The truck class includes single-unit trucks, truck tractor semi-trailer combinations, and trucks or truck tractors with semi-trailers in combination with full trailers. Recreational vehicles include motor homes, cars with camper trailers, cars with boat trailers, motor homes with boat trailers, and motor homes pulling cars. In addition, where provision is

made for bicycles on a roadway, the bicycle should also be considered a design vehicle.

The specific design vehicles and their relationships to geometric design are discussed in detail in Chapter Seven-Intersections of this manual and Chapter 9 Intersections of the Green Book.

2.5 OTHER DESIGN CONTROLS

Several other factors serve in various ways as controls for geometric design standards.

2.5.1 TERRAIN CHARACTERISTICS

Design standards used in flat areas may not be physically or economically practical in rugged terrain. Traditionally, most highway agencies recognize three categories of terrain characteristics–flat, rolling, and mountainous. Because there are no areas that might be classified as mountainous in Delaware, the Department has designated two categories of terrain characteristics to be considered in relation to design standards.

- Flat terrain. Any combination of gradients, length of grade, or horizontal or vertical alignment that permits trucks to maintain speeds that equal or approach the speed of passenger cars.
- Rolling terrain. Any combination of gradients, length of grade, or horizontal or vertical alignment that causes trucks to reduce their speeds substantially below that of passenger cars on some sections of the highway, but which does not involve sustained crawl speed by trucks for any substantial distance.

Designation of the terrain category involves considerable judgment rather than formalized measurement and criteria. To assure consistency in design, the terrain category should encompass road sections of at least 3 to 6 miles (5 to 10 km). Frequent changes in terrain designation for short sections of road should be avoided. It is better to consider average terrain conditions over a longer length of road. Due to Delaware's size, population distribution and predominant terrain, this manual recognizes that many projects are more influenced by the intensity of roadside development and associated activity than physical terrain features.

2.5.2 FUNCTIONAL CLASSIFICATION

Delaware has adopted a system of classifying and grouping highways, roads and streets as to their purpose and character of service they provide. These recognized functional classifications are shown in Figure 2-2.

Figure 2-2
Functional Classification

URBAN SYSTEM
Interstate Freeways and Expressways Principal Arterial Minor Arterial Major Collector Local
RURAL SYSTEM
Principal Arterial Minor Arterial Major Collector Minor collector Local

In addition to those classifications shown in Figure 2-2, there is a group of local streets classified as "subdivision streets." These are streets that lie within a DelDOT approved residential subdivision that have been determined to be eligible for state maintenance. The various facilities that comprise a designated functional system are shown on the official DelDOT Functional Classification Map available from DelDOT's Mapping Section.

Urban and rural areas differ in land use density, the types of land use, density of street and roadway networks, access and mobility needs, multi-modal needs, and travel patterns.

Because of these diverse needs, urban and rural functional systems are classified separately, as shown in figure 2-2. Urban areas have populations of 5,000 or more. The boundaries are set by the appropriate Metropolitan Planning Organization (MPO) in cooperation with DelDOT. Urban areas are further classified into urbanized areas (population of 50,000 and over) and small urban areas (population between 5,000 and 50,000). Rural areas fall outside the boundaries of urban areas.

2.5.2.1 ROADWAY TYPES WITHIN THE CLASSIFICATION SYSTEM

The following discussion is a general description of the classification of roadways. The total highway system is functionally classified based on a hierarchy on how competing functions of transportation movement and access are satisfied.

Interstates are a specially designated national system of highways serving most state capitols and major population centers for the purpose of national defense and the safe and efficient transportation of high traffic volumes. They are capable of serving larger vehicles carrying all types of goods with heavier loads than permitted on lower class roadways. The design standards are significantly higher in all areas, i.e. full control of access, increased pavement structure, increased lane widths, increased shoulder widths, wider medians, increased vertical clearances to all structures and increased strength requirements for bridges, etc. Essentially interstate facilities are to be strictly designed to the highest possible standards.

Freeways are arterial highways with full control of access having the capacity for high speed and high volume traffic movements over very long distances in an efficient and safe manner. The travel patterns are interstate, interregional, or intercity. Opposing traffic movements are physically separated and access is only provided via grade separated interchanges at selected public roads.

Expressways are similar to freeways but do allow limited access to intersecting state maintained roadways under strictly controlled conditions. They provide high speed, long-distance vehicular service.

Principal arterial roadways have the capacity for safely and efficiently carrying traffic flow at high speeds and high volumes for long distances. The travel patterns include interstate, interregional, and intercity. Access and service to abutting properties is subordinate to providing through traffic movement. Opposing traffic streams are separated by a median, usually non-traversable in urban areas. At-grade intersections are permitted but are controlled both in location and design. Coordination and traffic progression are considered a major issue.

Minor arterials have a capacity for medium to high speeds or medium to high volume traffic movements over medium to long distances safely and efficiently. The travel needs are regional, intercity, and intracity. Direct access to abutting land is subordinate to providing service to traffic movement. Intersecting highways, streets, or access to crossing movements are permitted but must meet spacing criteria, which allows signalization when volumes warrant. Progression in providing signalization is desirable but may be difficult to achieve, particularly in urban areas.

Major collectors have a capacity for moderate travel speeds and moderate traffic volumes for short travel distances providing for intercity and intracity travel needs. Mobility needs are balanced with direct access to provide the desired service. Traffic progression may not be a major concern.

Minor collectors are roadways that provide equal treatment and importance to abutting property access and the movement of traffic. They usually intersect with arterial roadways. The progression of traffic is not considered a major issue.

Local roads provide good access to adjoining residences and businesses but limited opportunity for through movement of traffic. Travel is short and movement is to intersecting roadways, usually of the collector classification.

Because each classification reflects a particular type of service, geometric design standards may vary, even for road sections within the same traffic volume group. These variations are considered in the tables of design standards in this manual. For design purposes the classifications of Principal and Minor Arterials have been combined into one class (Arterials) and the Major and Minor Collectors have been combined as Collectors. The designation of urban and rural arterials and collectors has been maintained.

The designer should be aware that functional classifications are not updated or revised on an annual basis but more likely on a five to ten year frequency. When scoping a project, particularly within the context sensitive design environment, a project area could be considerably different than would be indicated by its current designated functional classification.

2.5.3 MANUAL APPLICATION

There are many types of projects requiring varying levels of design effort and selection of design criteria. Most projects do not involve new alignment, new construction or major reconstruction that would allow significant changes in grades and geometrics permitting the use of full (maximum and desirable) design standards as set forth in Green Book and this manual. The majority of projects are planned and designed to maintain the existing highway system. Many projects are funded to address immediate needs such as interim improvement of the riding surface and improving traffic services and safety. These projects may include some or all of the following items: widening, strengthening of pavement structure, flattening of slopes, minor isolated alignment or grade changes, resurfacing, and minor traffic services improvements, such as traffic signals, curb and gutter, and channelization. These projects may also include creating opportunities for providing alternative modes of travel (transit, bicycles, and pedestrians). The proposed scope of work, available funding and project needs, along with the many issues involved in context sensitive design, must be evaluated in establishing the design standards.

Design standards appropriate for new construction or major reconstruction may be impractical both economically and in the disruption of the social context of a community or project area. The goal is to strive to meet the full geometric and cross sectional design controls in this manual and the Green Book for new construction and reconstruction projects. These two types of projects are generally described as follows:

- ***New Construction***. Projects on new alignment or major reconstruction projects on existing alignment where significant horizontal and vertical changes are included in the scope of work and funded.
- ***Reconstruction.*** Projects typically involving demolition and full depth replacement of the pavement structure, rubbilization or crack-and-seal of existing pavement combined with full depth overlay, removal and replacement of any significant part of the substructure or superstructure, and the total replacement of highway signs, highway lighting, or drainage systems. Safety considerations are an important part of reconstruction improvements.

There is a third type of project. These projects are classified as ***Preventive Maintenance.*** Projects in this category are defined as projects which include restoration or rehabilitation of specific elements of a highway facility. These improvements are recommended as part of an acceptable pavement or bridge management system showing that such

activities are a cost-effective means of maintaining the bridge or pavement life. In general, any work that (1) retains pavement structural capacity (general overlays up to 2 inches or mill and replacement of portions of the pavement structure up to 4 inches); (2) prevents the intrusion of water into the pavement or pavement base (crack or joint sealing, underdrains, restoration of drainage systems); (3) restores pavement rideability (profiling, milling); or (4) prevents deterioration of bridges (cleaning and painting, seismic retrofit, scour countermeasures, deck rehabilitation and repair, deck drain cleaning) are considered to be work which extends the service life of the highway. Please refer to the current FHWA stewardship agreement.

All projects on the National Highway System (NHS), except preventive maintenance, must conform to the full standards, as set forth in AASHTO's Green Book. When the minimum standards are not provided, a design exception must be prepared as provided in the current Delaware Department of Transportation/Federal Highway Administration Stewardship Agreement.

Many projects include a review of safety features and other roadway features. Frequently project intent, funding, environmental concerns and other issues do not permit these projects to be designed in accordance with all the standards in the Green Book or this manual. However, each project should be initially evaluated using the applicable standards. TRB Special Report 214, *Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation* provides a methodology for making decisions on which elements could be addressed to be most cost-effective in improving project safety. Flexibility both in application of this manual and AASHTO allow the designer to select design standards that will provide a safer facility, meeting the project intent, project funding, and user expectations.

2.5.4 ACCESS CONTROL

Access control is the regulation of public ingress and egress to and from properties abutting the highway facilities. Design standards and their application are affected by a facility's designated level of access control. The four basic types of access control are:

- Full Control of Access — Provides access to through-traffic lanes only at selected public roads by means of ramps. Crossings at grade and direct driveway connections are prohibited.
- Partial Control of Access — Gives preference to through traffic movement but provides some direct access connections with selected public roads, either grade separated or at grade.
- Access Management — Provides access to land development while simultaneously preserving the flow on the surrounding road system in terms of safety, capacity, and speed.
- Conventional Highways — Permit access directly to abutting property within the guidelines and criteria established for the location, spacing and geometrics of the access points. Such guidelines are set forth in DelDOT's *Entrance Manual*.

The functional classification system defines a highway's level of access in conjunction with a facility's purpose, importance, and functional characteristics. The basic principles in defining the level of access are:

- Classify the road system by the primary function of each roadway.
- Limit direct access to roads with higher functional classifications.
- Locate traffic signals to emphasize through traffic movements.
- Locate driveways and major entrances to minimize interference with traffic operations.

- Use curbed medians and locate median openings to manage access movements and minimize conflicts.

Controlling and managing access is accomplished through a combination of statutory authority, land-use ordinances, geometric design, and driveway regulations.

2.5.5 PEDESTRIANS

Pedestrians are an important part of the roadway environment. Pedestrian needs are more prevalent and influential on design in urban areas but their needs in rural areas should also be recognized. Pedestrian facilities include sidewalks, traffic control features, refuge islands, curb cuts (depressed curbs and ramped sidewalks), and ramps for older walkers and persons with mobility impairments. Pedestrian facilities are also an important supporting component for transit operations.

The typical pedestrian will not walk more than 1 mile [1.5 km] to work or over 1/2 mile [1.0 km] to catch a bus. Pedestrian actions are less predicable than those of the motorist. This makes it difficult to design a facility for safe and orderly movement of pedestrians. Pedestrians tend to walk in a path that represents the shortest distance between two points. Pedestrians tend to resist changes in grade or elevation when crossing roadways and tend to avoid using special underpass or overpass pedestrian facilities.

A pedestrian's age has an important role in how they use a facility. If the users are predominantly older, several measures have been identified in the FHWA publications *Highway Design Handbook for Older Drivers and Pedestrians* and *Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians.* Several of these design elements are: keep the design simple, assume slower walking speeds, provide adequate median refuge islands at wide intersections, and assure the geometric design is compatible with enhanced traffic control systems. Sidewalk design is more fully discussed in Chapter Ten-Miscellaneous Design.

2.5.6 BICYCLE FACILITIES

Bicycle users are an important element when making decisions defining a project's design parameters. Many existing streets and highway systems can accommodate bicyclists without significantly affecting costs or other impacts. Improvements that will enhance safety and promote increased bicycle use include: paved shoulders, wider outside traffic lanes, bicycle-safe drainage grates, adjusting manhole and utility covers, and maintaining a smooth, clean riding surface. Bicycle facility design details can be found in Chapter Ten-Miscellaneous Design and AASHTO's *Guide for the Development of Bicycle Facilities.*

2.5.7 ECONOMICS

Decisions on alignments, grades, widths, slopes and other items can greatly influence the construction cost. Geometric and structural standards higher than needed for a particular type of facility may cause increased expenditures that might be better spent on improving additional road sections. Use of standards that are too low may be uneconomical by contributing to early obsolescence of the facility.

The standards established by the Department reflect the best judgment as to design criteria for particular conditions. Sometimes the standards are expressed as minimum values but the opportunity often exists to use higher than minimum standards without significant additional costs. Designers should recognize these opportunities. At the same time they should recognize when increased costs for higher standards cannot be justified. Designers must continually be cost conscious within the framework of established criteria, defined scope of work, and project intent.

2.5.8 SAFETY

Safety is a major control in roadway design. Designs should minimize driver decisions and reduce unexpected situations. Designs should strive for uniformity in features and traffic control. Driver safety involves a variety of factors including:

- The number and use (variety and frequency) of access points,
- Operating speed,
- Type and width of median,
- Shoulder width,
- Alignment,
- Grades,
- Roadside design (roadside slopes and unyielding obstacles),
- The uniform and proper application of traffic control devices (signs, markings, and signals), and
- Properly designed intersections, particularly in rural areas.

Established standards generally consider safety factors. Items such as minimum sight distances and limitations on minimum curvature for a particular design speed are generally accepted as minimum fixed values. The safety value of some other items, such as guardrail, shoulder widths, side slopes and lateral clearances is not as clearly defined, and the designer may vary the treatment to suit the specific needs and provide the maximum possible safety where necessary.

It is difficult to completely separate safety and economic considerations. Designers should watch for opportunities to improve safety in the design when little or no additional cost is involved. At the same time, they should carefully evaluate proposed safety features that may result in extremely high costs. A well-documented study may be needed to economically justify the potential safety benefits.

A significant factor contributing to safety is access to the facility. Reducing the number, frequency, and variety of events to which a driver must respond improves safety. All roadways need to provide design features and operating characteristics that will reduce conflicts and minimize the interference between vehicles while still meeting the intended needs of the users.

Speed is often a contributing factor to safety but its role must be related to the accident site. The safest speed depends upon design features, road conditions, traffic volumes, weather conditions, roadside use, spacing of intersecting roads, cross-traffic volumes, and other factors. Design features that reduce the variance in speed of vehicles (such as flat grades, speed-change lanes, shoulders, grade separations, and appropriate signing and markings) improve highway safety.

In addition, it is important to recognize the type and characteristics of the drivers expected to use the facility. Trip purpose directly relates to the mix of vehicles likely to use the roadway. Where trips are of one predominant type, i.e. commuter, recreational or commercial, the facility should be designed to fit this specific need.

The use of medians has been found to increase safety on four-lane facilities. Depending upon the functional classification, funding and project scope, medians vary greatly in width and treatment.

Even though improvements in alignment, grade and traveled way cross section are included in the design, the roadside design itself is an important part of a safe design. Drivers leave the traveled way for a variety of reasons. Most of these occurrences leave the driver without full control of the vehicle. This means that obstacles near the roadway as well as the physical cross section of the roadside become potential contributors to safety. Roadside design is addressed in Chapter Ten-Miscellaneous Design and in greater detail in AASHTO's *Roadside Design Guide.*

Another element in providing a safe design is the use of traffic control devices: signs, markings and signals. Driver expectation and reaction is very dependent upon the communications provided through traffic control devices. Without a uniform, high-quality traffic control system even the best alignments and roadway cross sections may not function in a safe and efficient manner meeting user expectations. The four basics of effective traffic control are: (1) design, (2) placement, (3) maintenance, and (4) uniformity. More detail on the design of traffic control devices is found in Chapter 8-Traffic Services and the FHWA's *Manual on Uniform Traffic Control Devices* (MUTCD).

A major element in providing a safe design in rural areas is at-grade intersections. Safety can be improved by channelizing intersections, providing adequate sight distance (including stopping, decision, and intersection sight distance), proper lighting, signing and traffic control devices and, where necessary, providing for pedestrians (refuge islands and sidewalks). Intersections are an important design element and are discussed in detail in Chapter Seven-Intersections of this manual and Chapter 9 Intersections in the Green Book.

One or more of the factors that have been discussed may be applicable to a project. The designer should carefully review accident records and studies of the location as a guide to identifying locations where some form of safety improvement may be needed.

Statistical records for accidents involving injuries or fatalities are compiled for each road in the state through the Highway Safety improvement Program (HSIP) administered by the Traffic section. For each accident the Road Study Listing includes valuable design information such as:

- Milepost location,
- Intersecting roads, if any,
- The county, city, town or area,
- The highway system,
- The year, month, hour of the day and day of the week,
- The accident report number,
- The severity of the accident–fatality, injury or property damage,
- If alcohol or speed was involved,
- The available light, weather and surface conditions,
- The type of collision, if appropriate,
- The type(s) of vehicle(s) involved,
- The type of traffic control and if it was functioning, and
- The primary contributing circumstances.

Road Study Listings are compiled each year for all the accidents occurring during the year. Designers can request accident reports from the accident Data Coordinator in Planning.

The codes for the accidents are revised periodically and the data from Road Study Listings for prior years may vary somewhat from that currently provided. Designers should obtain copies of both the current accident coding sheets and those used earlier to accurately interpret the data.

Designers should also refer to the current *Traffic Summary* from Traffic to determine the milepost locations for each road in order to reference accident locations to design projects.

The Critical Rate Ratio Report, prepared by Traffic, shows the accident statistics by segment for each road. Designers should review this report to identify any areas with high accident rates within the road section being designed and consider improvements within the design scope to reduce accident frequencies.

2.5.9 ENVIRONMENT

Each project receives a level of evaluation as to its impact on environmental issues as established by state and national goals as well as its affect on community and area values. Projects on new alignment receive the most intense and detailed evaluation.

As a part of the evaluation at the initial planning and funding stages, projects are classified according to their probable impact on the environment and the level of expected impact on any affected communities and the project area. These two elements determine the type of environmental studies and public involvement to be conducted on a project. Projects reaching the design phase have been through some level of environmental studies with many of the important design issues identified. However, it should be recognized that as project designs are developed, scoping meetings are held, and public involvement continues, other important issues will very likely arise.

As a part of a project's development process, the designer will receive one or more of the following types of environmental documents. These are: (1) SEE (Social, Economic and Environmental) Report, (2) Categorical Exclusion (CE), (3) Environmental Assessment (EA), (4) Finding of No Significant Impact (FONSI), (5) Draft Environmental Impact Statement (DEIS), (6) Final Environmental Impact Statement (FEIS) or a Record of Decision (ROD). The level of environmental assessment given to a project is a major consideration in establishing its design controls and standards since commitments made in these assessments must be fully incorporated in the design.

Chapter Three

Design Standards

Designers are called upon to make numerous decisions as to the geometrics and physical characteristics of highway improvements. Without some basic framework of design standards, the judgments of individual designers may vary considerably. The purpose of design standards is to assure that highway improvements are consistently designed with due consideration of appropriate levels of traffic service, safety, and economy, consistent with the environmental and social context of the area (context sensitive).

Selecting design standards that are context sensitive is an important part of the design process. Chapter Ten, Section 10.1, Context Sensitive Design, describes this concept. The designer is reminded that there is flexibility in the standards set forth by AASHTO and this manual that allows choices to be made as the design progresses and complex community and environmental issues are raised. Since there are so many decisions made during the design process affecting design standards, documentation of these decisions is a critical part of the design process. This is particularly important on projects with extensive community involvement and an extended design process where previously discussed and resolved issues continue to be raised. Other reference publications on context sensitive designs are AASHTO's *Context Sensitive Design for Integrating Highway and Street Projects with Community and the Environment*, and FHWA's *Flexibility in Design.*

3.1 BASIS FOR STANDARDS

The concept of design standards has evolved from extensive highway agency field-testing, research, mathematical modeling and the study and documentation of many years of application and experience. The findings and conclusions are documented in many publications that serve as guides for highway designers. The design standards are flexible in that agencies must adopt those that are proven to work best for area(s) over which they have jurisdiction. Elements that influence selecting design standards include topography, geographical location, physical geology, predominant weather conditions, population growth, traffic volumes, predominant types of vehicles, past operational experiences, state and local transportation goals, community interests and other conditions that may affect the area of agency responsibility.

The flexibility to select project specific design standards does not compromise the national goal of maintaining a standard design. The concept of a standard design is reached through the consistent application of design principles. Drivers can reasonably expect transportation agencies to apply the same design principle when encountering similar conditions throughout the United States allowing the driver to be prepared and react in a consistent manner. For instance, all freeway ramps will have deceleration lanes and consistent signing.

3.1.1 AASHTO POLICIES AND GUIDES

The American Association of State Highway and Transportation Officials (AASHTO) is the recognized authority on American highway design policies and standards. Since 1938, AASHTO has been developing and publishing design policies and guides for use by transportation agencies and continues to update the information to reflect new findings and the current state of knowledge.

This manual cannot attempt to cover the total scope of important published information related to highway design policies.

3.1.2 APPLICATION OF STANDARDS

Depending upon the design element being evaluated, AASHTO design criteria are expressed as design values, minimum values or as ranges of values for particular elements and conditions. Design values are empirically derived; any value lower may be unsafe and any value higher may be unnecessary and uneconomical. Minimum values should, depending upon the element being evaluated, not be lowered either because it will create an unsafe condition or, in some cases, will not physically work. Some design elements lend themselves to minimum and desirable values. Although a portion of a project may require the application of minimum values, other sections will allow the designer to use standards higher than the minimum. In evaluating a project for application of standards, user expectation is important. Consistency in application of standards is an important safety feature. Large variances in standards may create unacceptable driver behavior. AASHTO standards are developed to allow agencies to select those that best meet their needs and practices.

Design values presented in this manual are in metric and US Customary units and were developed independently within each system. The relationship between the metric and US Customary values is neither an exact (soft) conversion nor a completely rationalized (hard) conversion. The metric values are those that would have been used had the manual been presented exclusively in metric units; US Customary values are those that would had been used if this manual has been presented exclusively in US Customary units. Therefore, the user is advised to work completely in one system and not attempt to convert directly between the two. Figure 3-1 shows the equivalent US Customary and Metric units for the commonly used design speeds.

Figure 3-1
Corresponding Design Speeds in US Customary and Metric Units

US Customary	Metric
Design speed (mph)	**Corresponding Design speed [km/h]**
15	20
20	30
25	40
30	50
40	60
45	70
50	80
55	90
60	100
70	110

The standards adopted by the Delaware DOT, described herein, adhere to the basic framework of AASHTO design policies. But the specific standards contained herein reflect judgments by the Department as to their proven operational success in Delaware and application to the predominant geographical conditions.

Most standards are related to a facility's functional classification with the interstate system having the highest and local streets having the lowest. For instance, 10 ft [3 m] travel lanes may be acceptable on local streets but completely unacceptable on facilities with high volumes, higher operating speeds, a more diverse mix of vehicle types, and a goal of maintaining or improving system capacity.

3.1.3 DEPARTURE FROM STANDARDS

Most projects are expected to meet at least the minimum standard design values established for the project level. Occasionally there may be conditions that warrant consideration of a lower value. For projects on higher functional classification roadways, it is more critical to strive to meet or exceed all of the applicable standards, primarily because the motorist expects a higher standard and will drive the facility accordingly.

The need for exceptions to the standards must be identified early in the project development phase in order that approvals or denials will not delay completion of the design or require extensive redesign. However, the need to evaluate a lower design value may arise at any time during the design process and needs to be addressed expeditiously. Thorough documentation is essential. The need for exceptions should not be viewed as normal or routine. The forms in this chapter shall be used to document decisions on design criteria and as a basis for developing and documenting requests for exceptions. The primary focus of the request should be highway safety. The design and proposed mitigation should be the best practical alternative that considers whether or not other controlling design elements will be adversely affected.

The major controlling design elements that need to be evaluated for conformity to established standards include:

- Design speed;
- Through lane and auxiliary lane widths;
- Shoulder widths;
- Stopping sight distance on vertical and horizontal curves;
- Horizontal alignment (radius of curve);
- Vertical alignment;
- Minimum and maximum grades;
- Cross slopes;
- Superelevation rate;
- Horizontal clearance;
- Vertical clearance;
- Bridge width; and
- Structural capacity.

An exception for design speed should not be sought as this element establishes most if not all of the other parameters to be met. As discussed in this chapter and in several other sections in this manual, design speed is an achievable speed selected by the designer based on the various factors the designer must consider. Design elements that cannot be met within that selected design speed should be supported by seeking a design exception in accordance with the following guidelines.

All variances from standards need to be documented in the project files, and, in many cases, require approval from the Chief Engineer and FHWA. The level of documentation depends on the project scope, functional classification and other factors. Depending upon the significance of the request, the support information may include some or all of the following:

- Existing roadway characteristics,
- Required and proposed design criteria;
- Cross section or geometric figures comparing the existing and proposed conditions;
- Supporting calculations and cost analysis;
- Analysis of accident records;
- A discussion on the compatibility with adjacent sections;
- Effect on right-of-way;
- Environmental constraints;
- Any proposed mitigation considered to help offset the variance from the design standard;
- Mitigation costs;
- Public support or opposition; and
- Other pertinent factors

There are several project types described in the following sections. Figure 3-2 shows the types of construction that fall under each project type. The type of construction shall be shown on the title sheet of the construction plans.

Figure 3-2
Types of Construction

Project Type	Type of Construction
New Construction and Reconstruction	Bridge Replacement
	New Construction (Road/Bridge)
	4-R
Intermediate	Bridge Rehabilitation
	Bridge Superstructure Replacement
	Intersection Improvements
	Safety Improvements
	Traffic Calming
Preventive Maintenance	Bridge Painting
	Bridge Preservation
	Bridge Preventive Maintenance
	Community Transportation Fund
	Pavement Preservation
	Scour Countermeasures
Miscellaneous Improvement	Advanced Utility
	Bike and Pedestrian Improvements
	Drainage Improvements
	ITS
	Landscaping
	Lighting
	Railroad Crossing
	Signal
	Signing and Striping
	Small Structure Repair
	Transportation Enhancement
	Miscellaneous Improvements (e.g., Wetland Mitigation, DTC Rail, Facilities)

3.1.3.1 NEW CONSTRUCTION AND RECONSTRUCTION PROJECTS

New construction and reconstruction projects on the interstate system and the NHS are expected to be in conformance with the appropriate standards and exceptions should be rare. All projects on the NHS (except preventive maintenance) shall conform to the standards in the current edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book). All projects on the interstate system (except preventive maintenance) shall also meet the design criteria contained in the current edition of *A Policy on Design Standards—Interstate System.* Projects having full federal oversight by FHWA are established in the current *Stewardship and Oversight Agreement* between FHWA and DelDOT. FHWA must approve all design exceptions for projects with full federal oversight.

New construction and reconstruction projects require a formal design exception as described in "Required Design Exception Documentation" (Figure 3-3) to justify the rationale for departure from the established design standards. The Chief Engineer must approve design exceptions for all new construction and reconstruction projects. "Design Exception Request" (Figure 3-4) is a guide format for developing a design exception request. It is important that the designer complete the "Design Control Checklist" (Figure 3-5) and the "Design Criteria Form" (Figure 3-6) for all new construction and reconstruction projects.

Figure 3-3
Required Design Exception Documentation

Project Type	Design Exception Request (Fig. 3-4)	Design Control Checklist (Fig. 3-5)	Design Criteria Form (Fig. 3-6)	Asst. Director, Design, and Chief Engineer Approval	FHWA Approval
New Construction or Reconstruction – Full Federal Oversight	X	X	X	X	X
New Construction or Reconstruction – State Administered	X	X	X	X	
Intermediate Level		X	X		
Preventive Maintenance		X			
Miscellaneous Improvement		X			

3.1.3.2 INTERMEDIATE PROJECTS

The scope of intermediate projects is improvements being retrofitted into the existing infrastructure. At times, it may be necessary to deviate from the design standards. It is important that the designer complete the "Design Control Checklist" (Figure 3-5) and the "Design Criteria Form" (Figure 3-6) for all intermediate projects. If design standards cannot be met due to site conditions, then it shall be noted on the Design Criteria Form. In addition, there must be a memo to the project file explaining the justification for the variances in the design standards signed by the Project Engineer, the Group Engineer and the Assistant Director. The level of documentation can vary depending on the project intent, environmental concerns and other issues. In most cases it is not necessary to provide the same level of documentation as provided in a formal design exception request because the project's scope has already been established. The following are examples of intermediate projects:

- Safety improvement projects
- Addition of channelizing islands with no reduction in existing lane or shoulder width
- Intersection improvements with no reduction in existing lane or shoulder width
- Minor lane or shoulder widening
- Traffic calming features, e.g. speed humps, chicanes, midblock median islands, chokers, or narrowed lanes. (Consult with FHWA for projects on NHS Routes.)
- Superstructure replacement
- Bridge rehabilitation

3.1.3.3 PREVENTIVE MAINTENANCE PROJECTS

Preventive maintenance includes rehabilitation or restoration of specific elements of a highway facility when such activities are a cost- effective means of extending the pavement or bridge life and shall not degrade any existing or geometric aspects of the facility. The majority of the work for these projects is between existing curb lines or outer edges of existing shoulders. These types of projects are not required to provide any documentation for design standards that cannot be met. A general list of preventative maintenance items is below:

- Pavement Preservation - Pavement milling and resurfacing of the same thickness with no reduction in lane widths, or pavement repair (e.g. sawing, sealing, pothole patching)
- Bridge Preservation
    - Bridge painting
    - Bridge Preventive Maintenance
        - Deck rehabilitation
        - Joint replacement or repair
        - Bearing replacement
        - Pile Jackets
    - Scour countermeasures

- Seismic retrofit

3.1.3.4 MISCELLANEOUS IMPROVEMENT PROJECTS

These projects are typically beyond the existing edge of pavement and are intended to improve safety and aesthetics. They may also include improvements within the roadway intended to improve safety which do not degrade the existing highway geometrics. These projects are not required to provide any documentation for design standards that cannot be met. A general list of miscellaneous improvement project work items is below:

- New or replaced curb and/or sidewalk
- Modifying sidewalk to comply with ADA requirements
- Roadside safety enhancements
  - New guardrail installation
  - Guardrail repair/replacement
  - New impact attenuators
  - Impact attenuator repair or replacement
  - Upgrading bridge rails
  - Removal of obstructions
- Median barrier replacement
- Drainage improvements
- Signing
- Small Structure Repair
  - Culvert lining
  - Retaining structure repair
- Large ground-mounted signs
- New sign structures (sign bridge, cantilever and bridge-mounted, provided they meet clearance requirements)
- Existing sign structure rehabilitation
- Striping with no additional lanes nor reduction in existing lane width
- New or replaced raised pavement markers
- New or upgraded signals
- ITS (e.g. fiber optic cable, message signs, cameras, emergency call boxes)
- New or upgraded lighting systems
- Fencing, provided existing stopping sight distance is not degraded
- Glare screens, provided existing stopping sight distance is not degraded
- Repair of structural components resulting from traffic impact
- Advanced utility relocation projects
- Landscape improvements

Figure 3-4
Design Exception Request

State Project No._____Federal-Aid Project No._____

Date: _____**Oversight Project: Yes _____No____**

Design Exception Abstract: (Provide a short summary detailing the nature of the exception, reasons for the request, etc.)

Note:
For all NHS projects, the thirteen controlling criteria to be met are design speed; through lane and auxiliary lane width; shoulder width; bridge width, structural capacity, horizontal alignment; vertical alignment; grades; stopping sight distance, cross-slope; superelevation; horizontal clearance; and vertical clearance.

RECOMMENDATION:

The purpose of this project is to----------.

The most effective method of addressing this is----------.

Based upon the conditions presented, it is recommended that a design exception be approved for the controlling substandard design element as justified.

Recommended By: _____
Squad Manager

Recommended By: _____
Group Engineer

Recommended By: _____
Assistant Director-Transportation Solutions

Recommended By: _____
Assistant Director-Design

Approved By: _____ **Date**: _____
Chief Engineer

Approved By: _____ **Date**: _____
Federal Highway Administration (NHS oversight projects only)

Enclosures: (Include design criteria, figures, calculations, etc. to document request.)

Figure 3-5
Design Control Checklist

PROJECT DATA

Squad Leader/Project Manager: _____

Project Title: _____

Contract No.: _____

Federal Aid Project No: _____

Project Limits: _____

Type of Construction:_____

Project Scope and Initial Estimate: _____

_____

_____

_____

DESIGN DATA

Functional Classification:_____

Current AADT (Year)_____

Projected AADT (Year):_____

Projected DHV (Year):_____

% Trucks_____

Directional Distribution (%):_____

Design Speed _____

Design Vehicle_____

Design Level of Service _____

Clear Zone_____

Recommended By: _____

Squad Manager

Recommended By: _____

Group Engineer

Recommended By: _____

Assistant Director-Transportation Solutions

Approved By: _____

Director-Transportation Solutions

Figure 3-6
Design Criteria Form

Design Criteria		
Design Factor	**As per Road Design Manual**	**Provided**
Design Speed*		
Width of Through Lanes*		
Width of Auxiliary Lanes*		
Width of Outside Shoulder*		
Width of Inside Shoulder*		
Cross Slope*		
Width of Median		
Stopping Sight Distance*		
Minimum Horizontal Curve Radius*		
Minimum K (Crest)*		
Minimum K (Sag)*		
Maximum % Grade*		
Maximum front slope (Unprotected Section)		
Maximum back slope		
Barrier Offset		
Superelevation Rate (%)*		
Bridge Width*		
Vertical Clearance*		
Structural Capacity*		
Horizontal Clearance *		
Width of clear zone		

General Notes:

- Use this form primarily for new construction or reconstruction projects.

* The Chief Engineer must approve design criteria deviating from the requirements of the Road Design Manual through the use Figure 3-4 "Design Exception Request."

Recommended By: _____
Project Manager

Recommended By: _____
Group Engineer

Approved By: _____
Assistant Director-Transportation Solutions

3.1.4 DETERMINATION OF STANDARDS

Figure 3-7 graphically defines the nomenclature used when describing the various elements that establish a roadway's cross section. The dimensions and geometrics needed to design this roadway section are known as a project's design standards. The basic information needed before beginning the process of establishing a project's design standards is:

- The ***functional classification*** of the road section to be improved. A roadway's classification is shown on DelDOT's current functional classification map but should be verified with the Division of Planning. A part of the initial field review and scoping meeting should be to verify that the area and roadway section being considered are truly representative of the designated classification.

- The ***scope of work*** proposed for the project under consideration is in the Project Initiation Form, project development documentation and other supporting data furnished to the designer.

- The ***traffic data*** for the road section is obtained from the Division of Planning and includes current traffic, projected traffic, percent trucks, accident history, etc.

With this information, the designer can proceed with the process of selecting the design speed.

3.2 STANDARDS BASED ON DESIGN SPEED

3.2.1 SELECTION OF DESIGN SPEED

The design speed establishes basic criteria for certain design elements. Two design standard considerations are related directly to the design speed:

- Curvature and superelevation, and
- Required sight distances.

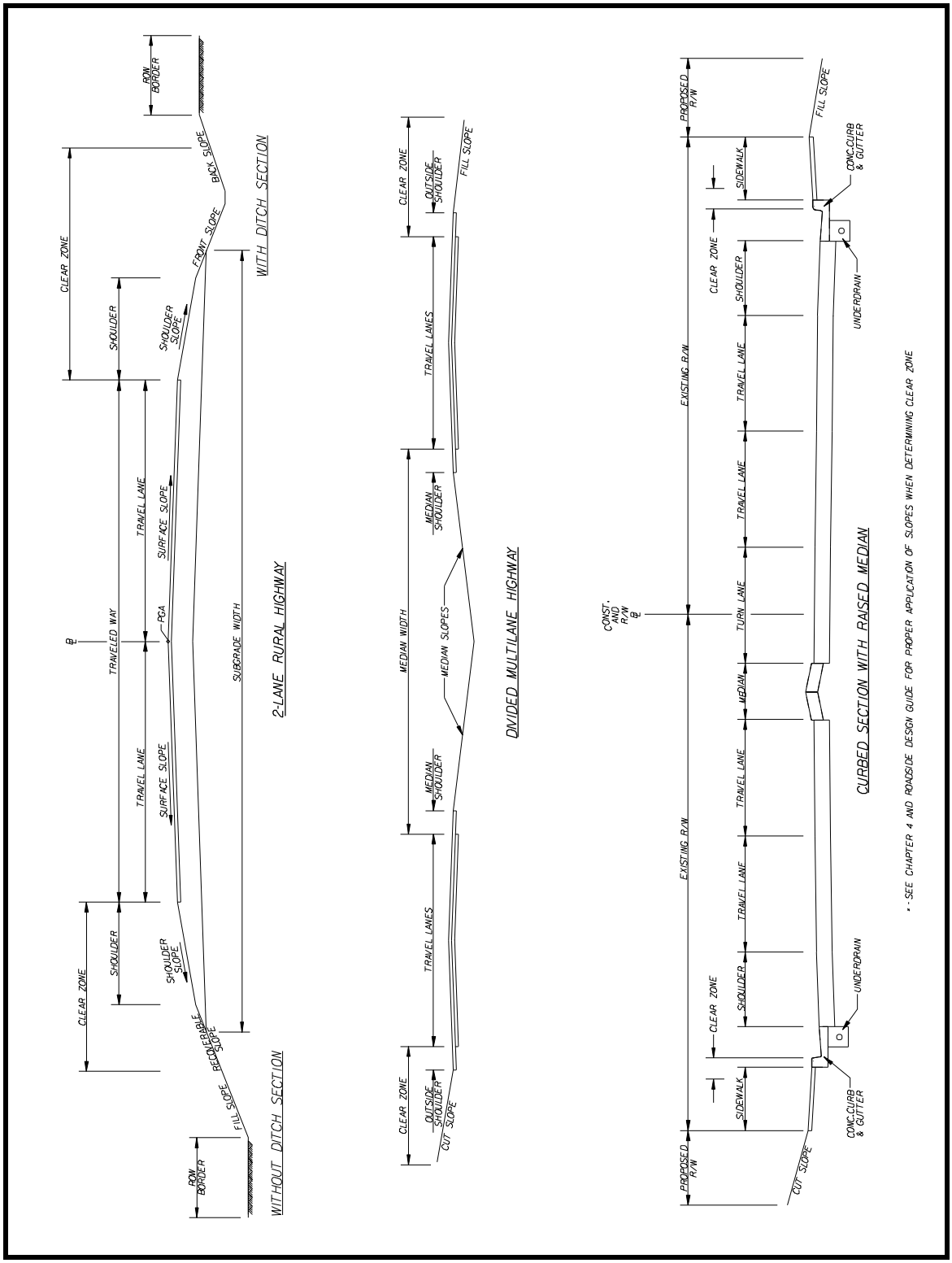
The designer's goal is to provide at least the minimum values, and preferably larger values, for these standards, regardless of traffic volumes, functional classification or any other consideration. These design elements are very closely related to traffic safety and should not be compromised.

A first step in determining the appropriate design standards is to establish a reasonable and realistic design speed. Since the majority of design controls are related to the design speed, this decision needs to be based on more factors than a roadway's functional classification and traffic volume.

The design speed selected should accommodate a high percentage of drivers, including the reasonable and prudent driver. Other considerations include topography, anticipated operating speeds, driver expectations, volume and mix of vehicles, the volume and type of non-vehicular traffic, driver familiarity, level of congestion reasonably acceptable to the motorists, and community values.

Once the design speed is selected, the pertinent highway features need to be related to obtain a balanced design. Some design features, such as curvature, superelevation, and sight distance, are directly related to, and vary with, design speed. Other features, such as lane and shoulder widths and clearances to highway appurtenances, although not directly related to design speed, affect the driver's comfort level and are reflected in vehicle operating speeds.

Figure 3-7
Typical Section Nomenclature



Designers should evaluate any unique conditions that might indicate a practical need for a higher or lower design speed. For example:

1. Design speeds should be selected as high as economically and physically practical.
2. The highway section may be legally posted for a relatively low operating speed; selecting a higher design speed may result in considerable added cost. Therefore, it would be appropriate to accept a lower design speed which is 5 mph [10 km/h] above the posted speed.
3. Extensive roadside development and proposed land-use changes, intersection spacing and frequency of entrances may influence decisions on design speed.
4. The need to preserve historic sites and districts may be a controlling factor.
5. The impact on the social context of the affected project area should be evaluated. This is particularly important when a project involves a rural setting and extends into a town center type of environment.
6. The impact on environmentally sensitive areas are part of the decision making process.
7. Whether or not the 85th percentile speed criteria should be used will have to be evaluated.

Keep in mind, however, that lowering the design speed will not necessarily lower operating speed without also lowering the legal posted speed limits. Before a final decision is made on the design speed, the adjacent road sections should be evaluated in terms of current operating speed characteristics and the potential for future reconstruction work. To the extent practicable, it is desirable to have consistent design speeds over longer sections of highways, where the roadway and roadside characteristics are also consistent and similar. If the adjacent roadside characteristics, traffic mix, and user activities vary dramatically within a project's limits, it may be more reasonable to use several design speeds This would be applicable when entering a business district or other activity center involving increased pedestrian use and cross traffic.

Since design speed selection is one of the most significant decisions, it is important to document the basis for making the selection and obtain approval before proceeding with the design. As the design process proceeds there may be issues raised that will call for a reevaluation of the design speed decision.

In addition to the design speed, a facility's projected traffic volume and functional classification influence the selection of traveled way (lane) and shoulder widths. The designer should refer to the Green Book in establishing traveled way and shoulder widths. The following is a guide to help locate this information.

- Local Roads and Streets — page 384, Exhibit 5-5;
- Collector Roads and Streets (Rural) — page 425, Exhibit 6-5;
- Collector Roads and Streets (Urban) — page 433;
- Arterials (Rural) — page 448, Exhibit 7-3;
- Divided Arterials (Rural) — page 455;
- Urban Arterials — page 472;
- Freeways — page 504.

Determining lane and shoulder widths is a very critical step in project design. The Design Criteria Form, Figure 3-6, is used to document and obtain approval for the selected lane and shoulder widths.

3.2.2 CURVATURE AND SUPERELEVATION

Establishing the proper relationship between design speed and curvature, as well as their joint relationship with the proper amount of superelevation on the curve is an important decision. Although these relationships are derived from laws of mechanics (speed, centrifugal force and side friction factor), the actual values for use in design depend on practical limits and factors determined empirically over a range of variables. For example, the

maximum permissible rate of superelevation is based on a practical consideration that a high operating speed can be accommodated on a relatively sharp curve if the superelevation is steep enough, but highways must serve vehicles traveling at a wide range of speeds. Slow moving vehicles or stopped vehicles would be adversely affected with excessively steep superelevation, particularly in ice and snow conditions.

AASHTO suggests maximum superelevation rates in the range of 4 to 12 percent. Delaware's roadways are subject to the effects of ice and snow during the winter. These conditions have resulted in poor operational and accident history on roadways using a superelevation rate higher than 8 percent. Therefore, DelDOT strives to use a maximum superelevation rate of 6 percent. However, for rural roadways it may be appropriate to use a superelevation rate of 8 percent. In urban areas, it is more practical to use a rate of 4 percent. This rate allows for smoother pavement tie-in at entrances and intersecting streets.

The selected superelevation rate sets the limitations on curvature. It is desirable to use curves flatter than the minimum values wherever conditions permit. When approved by the Chief Engineer, curves sharper than the minimum may be used on reconstruction projects. The designer has design alternatives to mitigate the effect of introducing sharper curvature by widening pavement, providing advance warning signs, providing wider clear zones, increasing vertical or horizontal sight distances, etc.

Tables of superelevation rates for various combinations of design speed and curvature are shown in the Green Book, pages 167 to 174, and figures in Chapter Five - Alignment and Superelevation in this manual. Both of these should be referred to for a more detailed discussion of the application of superelevation and transition methods for entering and leaving horizontal curves.

3.2.3 STOPPING SIGHT DISTANCE

Sight distance is the length of roadway ahead of the vehicle that is visible to the driver. The available sight distance must always be sufficient to enable a vehicle traveling at or near the design speed to stop before reaching an object on the roadway. Factors that influence the required stopping sight distance include:

- The speed of the vehicle;
- The height of the driver's eyes;
- The height of the object on the road;
- The driver's reaction time before braking;
- The surface condition; and
- The distance necessary to stop the vehicle after applying the brakes.

Reference should be made to Chapter 3 Elements of Design in the Green Book, pages 109-117, for a thorough explanation of the concepts and procedures for defining stopping sight distances. Attention is also drawn to AASHTO's discussion of the concept of 'decision sight distance' and its possible application to the project under design.

Vertical curvature, horizontal curvature, roadside obstructions, or any combination of these elements can restrict sight distance. Procedures for checking available sight distances are described in the Green Book, pages 127-131.

3.2.4 PASSING SIGHT DISTANCE

Consideration of passing sight distance is limited to two-lane, two-way highways on which vehicles frequently overtake slower-moving vehicles and the passing operation must be accomplished on a lane used by opposing traffic.

Passing sight distance for design is determined on the basis of the length needed to accomplish the passing maneuver. Derivation of the required distance is described in the Green Book, pages 118-126. AASHTO recommends

that, "In designing a highway these distances should be exceeded as much as practical ..."

These distances for design should not be confused with other distances used as warrants for placing no-passing pavement markings on completed highways. Values shown in the MUTCD are substantially less than the design distances and are derived from traffic operation control needs based on assumptions different from those for design.

Because of vertical and horizontal sight limitations, nearly all two-lane highways have some no-passing restrictions. In rolling terrain, the proportionate amount of no-passing sections usually becomes greater. Normally it is impracticable to attempt to provide passing sight distance throughout the entire length of a project. The principal design consideration is to try to provide adequate passing opportunities as frequently as possible.

There are no fixed values for the frequency of passing sections. Experience shows that highway capacity is measurably reduced when a significant percentage of a section of highway is restricted to sight distances less than 1500 ft [500 m]. Highways with high traffic volumes will require a higher proportion of passing opportunities than those with low traffic volumes. Where an analysis shows that a lack of passing sight distances has reduced capacity to near or below the expected traffic volumes, it is necessary to consider adjustments in the alignment and grade, or to provide additional lanes.

3.3 STANDARDS BASED ON TRAFFIC VOLUMES

Standards not directly related to design speed are influenced primarily by traffic volumes. Tables for these standards shown in the tables at the end of this chapter reflect variations by traffic volume ranges.

3.3.1 NUMBER OF LANES

The number of lanes required for any highway is directly related to the facility's traffic volume and desired level of service. But there are no simple, fixed criteria for these relationships. The recommended number of lanes is normally obtained through the project development process.

The *Highway Capacity Manual* gives two very general guidelines for determining the need for additional lanes. These numbers are based on long sections of roadway with uninterrupted traffic flow having the highest standards for design controls (horizontal and vertical geometrics and cross-sectional elements), ideal weather conditions, daylight, etc.

1. Under ideal conditions, a two-lane rural highway can accommodate about 900 passenger vehicles (two-way) per hour with a reasonably high level of service if there are adequate passing opportunities and no long, steep grades. Considerably more vehicles can be accommodated if motorists are willing to accept a lower level of service, a greater degree of congestion and lower operating speeds.
2. Under ideal conditions, a multi-lane highway can accommodate about 900 passenger vehicles per lane per hour. Again, considerably more vehicles can be accommodated, if lower levels of service can be tolerated.

Most roadways do not meet the ideal conditions. The HCM defines the ideal roadway as follows: (1) meets or exceeds design speed; (2) has 12 ft [3.6 m] travel lane widths; (3) has shoulder widths greater than 6 ft [1.8 m]; (4) has minimal no passing zones; (5) carries predominantly passenger cars; (6) has evenly distributed traffic flow; (7) has minimum crossing and entering traffic interference; and (8) has level terrain.

Although all these elements are rarely available within a project's limit, capacity is

usually not a problem on most of the rural roadways in Delaware. Exceptions are some of the principal arterial routes, particularly in the vicinity of urban areas.

Most proposed improvements will be in traffic volume ranges where the existing number of lanes will be adequate without the need for detailed study. However, capacity may be influenced where the traffic volume exceeds about 900 DHV or where there are unusual conditions of alignment, grade or signalization.

Designers working with Traffic and the Division of Planning should identify the need for additional through lanes or, if applicable, auxiliary climbing lanes. Such a change after project initiation is a major decision affecting all aspects of a project from cost to environmental and social impacts.

3.3.2 SURFACED LANE WIDTHS

The traveled way designated for vehicle operation (excluding shoulders) normally consists of two or more surfaced traffic lanes. The impact of providing adequate lane widths is wide ranging and includes maintaining and/or enhancing driver safety, driver comfort, the level of service, capacity, and the frequency and extent of shoulder and pavement surface maintenance.

For all new construction and reconstruction on arterial and collector roads, the desirable surfaced travel lane width is 12 ft [3.6 m]. If the scope of work is limited, speeds are low, truck volumes are light or there are no defined safety problems, surfaced lane widths of 11 ft [3.3 m] may be acceptable, particularly in urbanized areas with restricted right-of-way and increased pedestrian activity. However, for urban arterials with higher speeds, predominantly free-flowing conditions, and higher traffic volumes, surfaced lane widths of 12 ft [3.6 m] are desirable. For local roads and streets, surfaced traffic lanes normally should be 11 feet [3.3 m] wide but AASHTO allows lane widths of 9 [2.7 m] or 10 ft [3.0 m] where there is restricted or low truck use, low traffic volumes and low operating speeds. See Section 3.2.1 for information on selecting lane and shoulder widths.

For pavements on new construction or major reconstruction projects with existing or projected high concentrations of truck traffic, a wider pavement provides more edge strength and has been found to be structurally better for heavy loads. Consideration should be given to widening the pavement an additional 2 ft [0.6 m] under these circumstances. The lanes should be striped for 12 ft [3.6 m] lanes to keep trucks away from the edge of the pavement. The extra width can be considered part of the shoulder. If the mainline and shoulders are constructed of Portland cement concrete and the shoulders are structurally tied to the mainline, this additional width is not normally necessary. For divided highways, the widening should be adjacent to the outside shoulder; on two-lane roadways the widening should be equally divided on each side.

3.3.3 SHOULDER WIDTH

The total shoulder width is the distance from the edge of the traffic lane to the intersection of the shoulder slope with the front slope, or to the face of curb. In sections without curbs there are two terms used to describe the shoulder area. The “graded” width of shoulder is that measured from the edge of the traveled way to the intersection of the shoulder slope and the front slope. The “useable” width of shoulder is the actual width that can be used when a driver makes a stop.

Having a sharp break at the point of intersection of the edge of the graded shoulder and the front slope is not a good practice. Instead a rounding of 4 to 6 ft [1.2 to1.8 m] with a front slope 4:1 or flatter is the best practice. This rounding improves the general safety of the roadside by reducing the likelihood of encroachment, thus giving the errant driver more chance to regain control. Other considerations are that rounding may reduce rollovers and the possibility that the vehicle may become air-

borne. A portion of the rounding (usually one half) can be considered part of the "useable" width. Where front slopes are steeper than 4:1, the rounding should occur outside of the useable shoulder width.

Well-designed and maintained shoulders are necessary on rural highways with any appreciable traffic volumes. Shoulders and their widths are an important consideration in establishing a project's design standards. The benefits of including a shoulder include:

- Providing a refuge when a driver makes an emergency or parking stop.
- Providing lateral recovery areas for vehicles that inadvertently leave the traffic lane.
- Providing improved sight distance in cut areas.
- Providing areas for maintenance operations, including snow removal and storage.
- Providing for alternative modes of travel by pedestrians, bicyclist, joggers, transit operations, etc.
- Structurally improving the service life of the pavement by increasing the stability of the roadway's base and surfacing materials at the edge of the through traffic lane.
- Providing the opportunity for improved subgrade drainage designs.

Section 3.2.1 discusses the selection of shoulder widths. Normally shoulder widths of 10 ft [3.0 m] are used on new construction projects for arterial highways with relatively high traffic volumes. Where truck traffic exceeds 250 DHV it is desirable to have a paved shoulder width of 12 ft [3.6 m]. AASHTO allows narrower shoulder widths on most roadways with lower traffic volumes. However, wider shoulders widths should be provided on these projects when practical. Where bicyclists and pedestrians are to be accommodated on the shoulders, a minimum useable shoulder width, clear of any rumble strips, of 4 ft [1.2 m] should be used. On highways with three or more lanes in each direction a median shoulder width of 10 ft [3.0 m] is desirable. This provides a refuge area for disabled vehicles without affecting roadway capacity and flow.

3.3.4 SURFACED SHOULDER WIDTH

The surfaced shoulder width is that part constructed to provide better all-weather load support than is afforded by natural soils or stabilized materials. The paved portion of the shoulder also protects the edge of the traffic lane pavement from deterioration and raveling. More discussion on shoulder surfacing is in Chapter Nine - Pavement Selection. Normally the shoulder's structural design, including surface material, is recommended by the Materials and Research Section.

3.3.5 SIDE SLOPES

Various cross section slopes are identified in Figure 3-7. Four of these slopes are described below.

- **Front Slope.** The slope extending outward and downward from the shoulder to the ditch line.
- **Back Slope**. The slope extending upward and outward from the ditch line to intersect the natural ground.
- **Fill Slope**. The slope extending outward and downward from the shoulder to intersect with the natural ground; it may include a ditch section.
- **Cut Slope**. The slope extending outward and upward from the shoulder, intersecting the ditch slope and then extending upward from the ditch back slope to natural ground.

It is generally desirable that these slopes be 6:1 or flatter. Often, from a practical standpoint, they must be steeper. There is a distinct relationship between the steepness of side slopes, operating speeds and the desirable widths of clear zones. Chapter Four discusses

the relationship of these slopes to establishing a roadway's clear zone. General criteria for side slopes are presented in Figure 4-4, in terms of both desirable slopes and maximum slopes. The desirable slopes should be provided wherever feasible.

3.3.6 HORIZONTAL CLEARANCE AND CLEAR ZONE

3.3.6.1 HORIZONTAL CLEARANCE

Horizontal clearance is the lateral distance from edge of traveled way to a roadside feature or object for a roadway with barrier curb. Roadways having curbed sections should be provided with a minimum horizontal clearance of 1.5 ft [0.5 m] beyond the face of curb, with wider offsets (if possible to the full clear zone width) provided where practical since most types of curbs provide little help in redirecting an errant vehicle. Please see the Green Book for more information regarding horizontal clearance and AASHTO's Roadside Design Guide for more information regarding the clear zone width. If the minimum horizontal clearance cannot be provided in curbed areas, then a design exception is required.

3.3.6.2 CLEAR ZONE

The clear zone is defined in AASHTO's Roadside Design Guide as "the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry." This border area includes any shoulders or auxiliary lanes. Adequate lateral clearance between the edges of traffic lanes and roadside obstructions has been shown to be a very important safety factor. Vehicles leaving the roadway should have a reasonable opportunity to recover control and return to the roadway without overturning or colliding with roadside obstacles such as trees, poles, headwalls or other large objects. The combination of a relatively flat slope and an obstacle-free roadside within the prescribed clear zone helps this situation.

The determination of a clear zone is a function of speed, volume, curvature and embankment slope. The most current edition of AASHTO's *Roadside Design Guide* should be used for determining clear zone widths. For low-speed rural collectors and rural local roads, a minimum clear zone width of 10 ft [3.0 m] should be provided.

Some roadside appurtenances, such as guardrail, breakaway light poles and signs using breakaway posts, are permitted within the specified clear zone, due to their crashworthiness. They should be placed in the safest available location, minimizing their use when possible. Please refer to the *Roadside Design Guide* for more information. For guardrails within the clear zone, it is desirable to maintain a minimum 2 ft [0.6 m] lateral clearance between the outer edge of the usable shoulder and the face of the rail. At bridge approaches, guardrail should either match the width of the bridge or taper to meet the bridge rail.

The width of clear zone is included on the Design Control Checklist (Figure 3-5), the Design Criteria Form (Figure 3-6) and the title sheet of construction plans. Deviations from the clear zone criteria will have to be approved by the appropriate assistant director.

3.3.7 GRADES

Design standards for maximum grades are not as precise and objective as the standards for other geometric elements. AASHTO has established recommended maximum grades based principally on analyses of vehicle operating characteristics. Criteria for maximum grades are related principally to design speed, traffic volumes and terrain characteristics.

When it is necessary to design grades at or near the maximum values for relatively long distances, designers should investigate the effect on lane capacity. The lane capacity problem may be further complicated where there

are steep grades accompanied by considerable no-passing distances.

More detailed guidelines and criteria for the design of grades, including critical lengths of grades and minimum and maximum grades are presented in Chapter Five - Alignment and Superelevation and the Green Book, pages 231-250. The maximum grades should be used infrequently, only as dictated by severe terrain conditions. When it is necessary to use maximum grades, the designer should check other design criteria and roadside features that may be improved to minimize the impact of using the higher design grade.

3.3.8 BRIDGES

The designer should coordinate with the Bridge Design Section when determining vertical clearances. A minimum vertical clearance for roads over interstate, U.S. and state routes is 16.5 ft [5 m]. Pedestrian bridges and overhead sign structures must have an extra 1 foot [0.3 m] of clearance, a total of 17.5 ft [5.3 m]. These clearances allow for a 4 in. [100 mm] future resurfacing.

3.3.9 MEDIANS

Geometric criteria for medians on multilane divided highways are discussed in Chapter Four.

Chapter Four
Cross Section Elements

Many of the basic geometric criteria for design of the various cross section elements were described as design standards in Chapter Three. This chapter provides more detailed instructions for practical application of these criteria, along with guidelines for cross section elements not previously discussed.

The term "cross section" is used to define the configuration of a proposed roadway at right angles to the centerline. Typical sections show the width, thickness and descriptions of the pavement section, as well as the geometrics of the graded roadbed, side ditches, and side slopes.

Criteria are presented in two general categories: (1) those related to surfacing elements, and (2) those related to grading elements.

4.1 SURFACING ELEMENTS

The surfacing cross section includes the pavement for the traffic lanes, the shoulders, and the base and subbase courses that are placed on the graded roadbed, as well as curbs that may be used adjacent to the pavement.

4.1.1 SURFACE TYPE

The type of pavement usually is determined by analysis of the volume and composition of traffic, the soil conditions, the availability of materials, the initial cost, the desired service life and the estimated cost of maintenance.

Recommendations on surface type and structural thickness are prepared by the Materials and Research Section and are used in the typical sections prepared by designers. A general discussion of criteria and procedures for selecting the type of pavement, structural design of the various surfacing courses and different pavement rehabilitation techniques can be found in Chapter Nine-Pavement Selection.

The texture of the type of surface to be used has an influence on the prescribed cross slopes for pavement surfaces and for shoulders. For this reason, three general types of surfacing are recognized:

- **High type**–hot-mixed asphalt concrete or Portland cement concrete on a prepared subbase with an improved subgrade;
- **Intermediate type**–hot-mixed asphalt concrete on a prepared subbase; and
- **Low type–**surface treatment on prepared subbase material.

4.1.2 LANE AND SHOULDER WIDTHS

Criteria for widths of traffic lanes and shoulders are in Chapter Three. The basic policy is to provide 12 ft [3.6 m] traffic lanes on all arterials and collectors. This lane width is desirable in that it provides sufficient clearances between large commercial vehicles in the opposite lane on two lane roadways or four-lane undivided roadways. Lane widths affect level of service, operating speeds, and driver comfort. If the lanes are too narrow for the traffic volume and composition, it can result in reduced driver comfort and create erratic operations. Narrower lanes are permitted on some lower classifications of roads with rela-

tively low traffic volumes. Narrower lanes may also be permitted on some higher level roadways where the scope of work is limited, right-of-way is restricted, there is extensive adjacent development, significant pedestrian traffic is present, and other constraints may apply.

Prescribed shoulder widths vary widely, depending on the functional classification of the highway, the traffic volume, and the type of improvement. For new construction on most arterial highways, the shoulder should have a width of 10 ft [3.0 m], while narrower shoulders may be acceptable for lower class roadways having lower traffic volumes and a low percentage of trucks. Unless local conditions significantly increase costs, designers should provide at least a 4 ft [1.2 m] shoulder and preferably an 8 ft [2.4 m] shoulder. Shoulders provide safety benefits, protection of the structural integrity of the edge of the travel lane pavement, and accommodate bicycle and pedestrian use.

Shoulders are usually not required adjacent to auxiliary turning lanes.

4.1.3 MEDIAN SHOULDERS–DIVIDED HIGHWAYS

In the case of divided highways, left shoulders (or median shoulders) are also to be provided but the criteria are different. The left shoulder need not be quite as wide since the purpose is to keep vehicles from rutting the edge of the traveled way and aid in recovery if the driver leaves the traveled way.

- The inside shoulder width for a depressed median should be at least 4 ft [1.2 m];
- The inside shoulder on six lane facilities should be 10 ft [3.0 m]
- Where there is a concrete or guardrail barrier in the median, an additional 2 ft [0.6 m] clearance from the outer edge of the shoulder to the face of the barrier is needed.
- A minimum of 1 ft [0.3 m] and preferably 2 ft [0.6 m] clearance from the edge of the traffic lane to the face of the curb is needed with curbed medians.
- If drainage inlets are to be installed, the shoulder may have to be widened to keep the basins/grates out of the running path. Since this is a major design and economic decision, the designer should obtain approval early in the design process.

4.1.4 CROSS SLOPES

It is important to enable surface water to drain from traffic lanes and shoulders as quickly as possible. Accumulations of water (ponding) cause hazards by reducing surface friction and vehicle stability. Sufficient cross slope is needed for adequate drainage, but too great a slope adversely affects vehicle operation. In addition, good drainage minimizes moisture penetration at the pavement/shoulder joint thus increasing stability and ensuring the mainline pavement will meet its projected service and design life.

The type of surface greatly influences surface drainage characteristics. Dense, smooth surfaces (concrete or high-type asphalt) require a lesser

Figure 4-1
Pavement Cross Slopes for Traveled Way

	Pavement Cross Slope (%)	
Surface Type	**New Construction/ Reconstruction**	**Preventive Maintenance**
Portland Cement Concrete or Asphaltic Concrete	2.0	1.0–3.0
Surface Treatment	N/A	2.0–4.0

slope for adequate cross drainage than is required for gravel or a coarse-textured, intermediate-type asphalt surface. The Department has adopted the cross-slope values shown in Figure 4-1 for standard practice on tangent sections of highways.

The range of cross slope values for projects other than "new construction" permits slight variations in the slope where the scope of work is primarily resurfacing. If surface drainage is a problem, cross slopes up to 2.5% on high-type pavements may be justified. However, new construction slopes should be provided wherever practical. For two-lane highways or multi-lane undivided highways, the cross slope normally goes downward both ways from a crown point at the highway center line.

On divided highways, each one-way pavement may be crowned separately, as on two-lane highways, or each may have a one-direction cross slope across the entire width of pavement-usually downward to the outer edge. Surface drainage on roadways with three or more lanes in one direction can cause problems if the pavement slopes uniformly in one direction at the rate of slope recommended for one- and two-lane roadways. The designer has two options for cross slopes on a three-lane roadway: (1) slope the inside lane to the median and the other two lanes to the outside, or (2) slope all three lanes to the outside and increase the slope of the outside lane.

A cross section with each roadway crowned separately, such as the first option above, has an advantage in rapidly draining the pavement during rainstorms. Disadvantages are that more inlet and underground drainage lines are required, and treatment of at-grade intersections is more difficult because of several high and low points on the cross section. Sections having no curbs and a wide depressed median are particularly well suited for this design. With a crowned section, cross slopes should not exceed 2% because the rollover effect, when changing lanes, is then 4%. "Rollover" is the algebraic difference between the two slopes.

Roadways that slope in only one direction are more comfortable to drivers because vehicles tend to be pulled in the same direction when changing lanes. This design is generally desirable for divided highways with a narrow curbed median. The cross slope of the third lane (outside lane) of a three-lane roadway where the cross slope is all one direction should be increased by 0.5% to 1.0% to improve surface drainage.

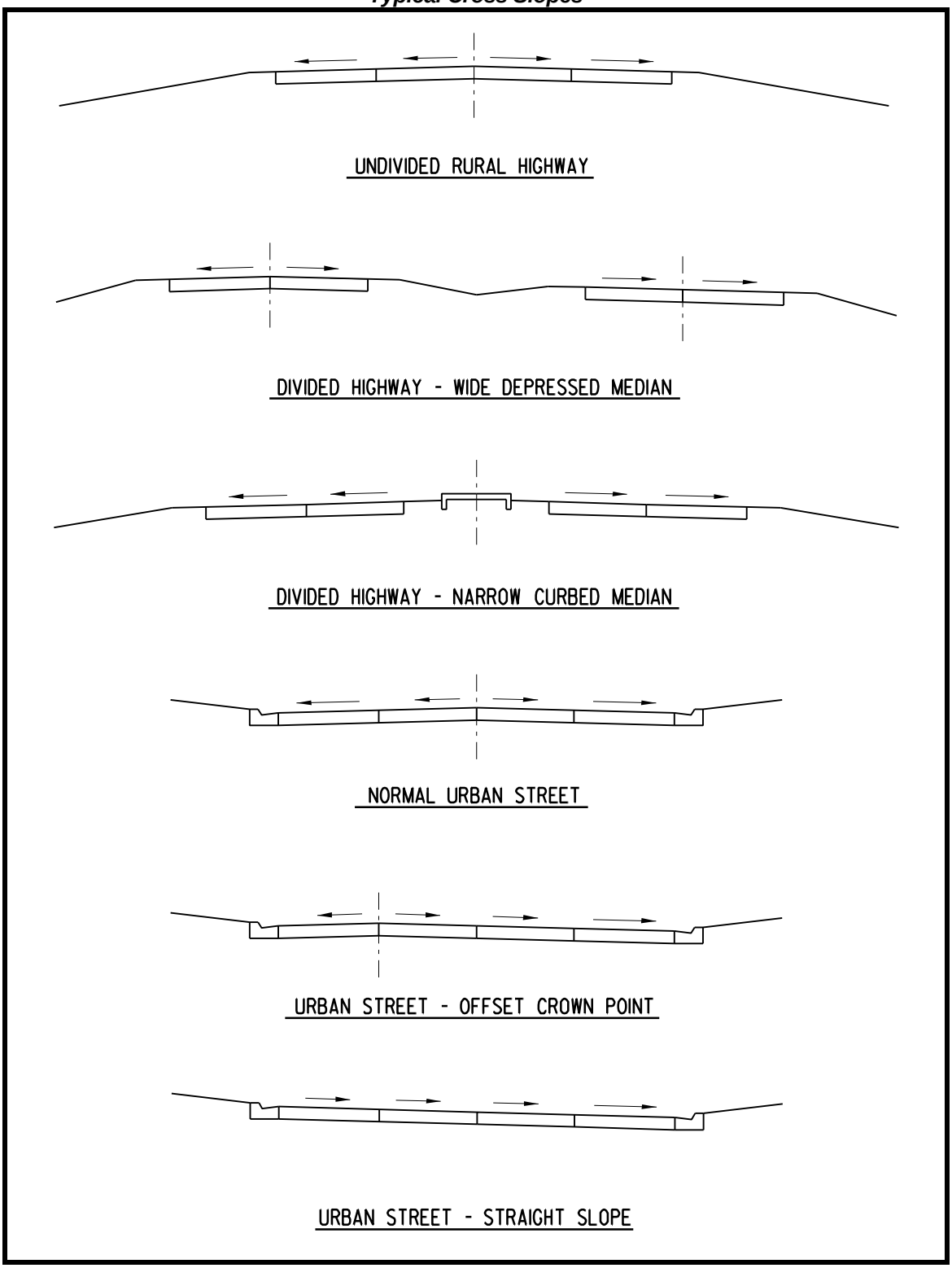
In the design of urban highways and streets, it may sometimes be found that adjacent property developments dictate that the curb on one side must be higher than the curb on the other. Two options are available. The cross slope can be in one direction for the full width of the street, or the crown point can be offset from the centerline toward the high side of the street. The latter option usually is preferable with the offset crown point corresponding to an edge of travel lane, out of the wheel path and with a maximum of 4% rollover.

Typical cross slope designs are illustrated in Figure 4-2.

4.1.5 SHOULDER CROSS SECTIONS

Shoulders should be flush with the roadway surface and should abut the edge of the traffic lane. All shoulders, including median shoulders on divided highways, normally should be sloped to drain away from the traveled way. However, in the case of a raised narrow median, the median shoulders may slope in the same direction as the traffic lanes, but consideration should be given to sloping the shoulders toward the median and providing inlets and underground drainage to alleviate problems with snow and ice. Slightly sloping shoulders steeper than the traffic lanes assure rapid surface drainage, reduce the chance of ponding, and minimize subgrade penetration of moisture through the edge joint. Paved shoulders normally should slope at a rate of 4%, and unpaved shoulders should be sloped at a rate of 6%.

Figure 4-2
Typical Cross Slopes



Special attention must be given to shoulder slopes in relation to superelevation on curves. Shoulder slopes that drain away from traffic lanes on the outside (high side) of a superelevated curve should be designed to avoid too great a cross slope break at the pavement edge. The rollover should not exceed 8%; differences greater than this tend to pull the vehicle toward the shoulder and may result in difficulty for the driver to regain control. For example, with a superelevation rate of 6% and a shoulder slope of 4%, the rollover would equal 10% which is unacceptable. The shoulder slope should be reduced to 2% but not less than 1% along the high side of the curve; this is acceptable since there is no stormwater discharge to the shoulder from the pavement and there is little opportunity for ponding or shoulder erosion damage.

Standard shoulder slopes should be used on the inside (low side) of superelevated curves unless the rate of superelevation exceeds the rate of normal shoulder slope. In this case, the shoulder slope should be the same as the superelevation slope.

4.1.5.1 GRASS SHOULDER

Vehicles often drift off the roadway and cause rutting of the shoulder edge. This is particularly noticeable at intersections and on the inside of sharp curves. Several options of grass shoulders are available. Vehicles may also park on the grass shoulder. Grass shoulders should be used only for special project needs. A grass shoulder shall not be used directly under a bridge overpass. Consider the following:

1. Use inground pavers, geogrid or a combination, which provide stability and allow grass to grow up through them. This is suitable for the area where parking is anticipated. Consider this where trash pick-up or mail delivery could rut the shoulder.
2. Provide a two foot wide paved shoulder adjacent to the grass shoulder, which may or may not be stabilized.
3. Provide a pavement edge line to determine the edge of the travel lane.
4. In a heavily salted area, alkaligrass may be used. Contact the Roadside Environmental Administrator.
5. The time of year for seeding and the length of time after seeding before the shoulder is subject to traffic are important. Phasing work should provide time for growth after seeding, and the shoulder should be protected until the grass is firmly established.

4.1.6 CURBS

Curbs are closely related to other surfacing cross section elements. They generally serve several purposes including drainage control, pavement edge delineation, delineation of pedestrian walkways, and control of entrances to roadside development. Curbs are used extensively on various types of urban highways and streets. In the interest of safety, curbs should be omitted on high-speed rural highways when the same objectives can be attained by other acceptable means. Curbs may be considered an obstruction, increase project cost and design effort. When using curbs, positive drainage of paved areas, particularly the traveled way, is necessary. This normally requires the installation of a closed drainage system with drainage inlets, positive outfalls and extensive ditching. Therefore, the need and use of curbs should be given appropriate study.

Curbs may be designed as a separate unit or integrally with the pavement structure. Separate curbs usually are a combination curb and gutter. Sometimes the curb is constructed alone without the gutter section.

The two general classes of curbs are barrier curbs and mountable curbs. Barrier curbs tend to, but do not always, prevent vehicles from crossing the curb line; mountable curbs permit such vehicle crossings without much difficulty. The types of curbs used most commonly are in DelDOT's *Standard Construction Details*. Refer to Chapter Ten for criteria for curb installations.

4.2 GRADING CROSS SECTION

The geometric elements of the grading cross section include the width and shape of the graded roadbed that consists of either suitable natural material or specified imported material. The top surface of the roadbed soil is defined as the subgrade. The pavement structure is placed on the prepared roadbed. The pavement structure includes any required selected subbase materials, base materials and the various layers of paving courses. The subgrade includes the various cut and fill slopes related to grading operations, including side ditches, to prepare a surface for constructing the pavement structure.

4.2.1 SUBGRADE CROSS SLOPES

The cross slope of the bottom of pavement box (top of subgrade) should parallel the cross slopes of the finished traffic lanes for the full width of the roadbed, including shoulders. This allows the pavement structure to drain through the porous material into side ditches or, if necessary, an underdrain system. The subgrade slope should not be broken to parallel the steeper finished shoulder slope. The parallel relationship between the subgrade and the finished traffic lanes applies to both normal crown slopes on tangent sections of highways and superelevated sections on curves.

4.2.2 SUBGRADE WIDTH

The design width of the subgrade should be shown on the roadway typical section(s). The total subgrade width should be the sum of the widths required for travel lanes, shoulders, median area, and any side slopes necessary to meet the subgrade.

For ease of computation and field staking, the design width may be rounded off to the nearest 1 ft [0.3 m]. This will result in a slope from the outside of the finished shoulder to the subgrade shoulder slightly different from the slope prescribed by the standards, but the variations will be negligible.

4.2.3 SUBGRADE WIDENING FOR GUARDRAIL

The subgrade should be widened at locations where guardrail is to be installed. The purpose is to provide the required horizontal clearance from the edge of the normal shoulder to the face of the guardrail, ensure the stability of the guardrail posts when placed in the embankment, and reduce maintenance.

Normally, 6 feet [1.8 m] of widening is required. This width includes 2 feet [0.6 m] from the normal shoulder line to the face of rail and 4 feet [1.2 m] behind the face of rail to a newly established edge of shoulder (the point of intersection of the front slope with the stabilized shoulder subgrade). Widening requirements are shown in DelDOT's *Standard Construction Details* for guardrail including special details for tapered flares for end treatments.

Refer to Chapter Ten for criteria for guardrail installations for various conditions relating to high embankments, non-traversable hazards and bridge ends.

4.2.4 SIDE SLOPES

A roadway's cross section includes side slopes as illustrated and identified in Figure 4-3. Side slopes are important in maintaining the stability of the roadbed and pavement structure as well as providing an area for the safety of errant vehicles. Side slopes are constructed in both fill (embankment) areas (those falling above the natural ground level) and cut areas (those falling below the natural ground level). As a general reference, slopes in embankment areas are commonly referred to as fill slopes or front slopes. When it is determined that no parallel ditch section is needed the front slope is graded to meet natural ground. In cut areas, side slopes are referred to as front slopes and back slopes, the back slope being necessary to bring the roadway cross section back up to meet the natural ground level. Ditch sections included as part of either fill or cut sections have a front slope, a ditch bottom with a defined shape and width, and a back slope. Criteria for rates of

these slopes (by road classes) are shown in Figure 4-4. The application of the criteria is very important in selecting a safe cross section. This application is discussed in this section; a full understanding of the concepts presented in AASHTO's *Roadside Design Guide* is critical to the proper application of the criteria.

All three slopes depend upon a lateral horizontal area measured from the edge of outside travel lane, called "clear zone". Consideration must be given to the lateral clear zone requirements when applying the criteria in selecting all side slope and ditch sections for the design cross section.

4.2.4.1 SIDE SLOPES WITHIN THE CLEAR ZONE

A roadway's "clear zone" is the total roadside border area, starting at the edge of the inside traveled way, that is considered available for safe use by errant vehicles. In addition to any shoulder area, the clear zone area may consist of a combination of a recoverable slope, a non-recoverable slope, a traversable slope, a clear run-out area and a critical slope. These slopes are defined as follows:

- A recoverable slope is flatter than 4:1 and an errant driver has a high probability of being able to recover control of the vehicle.
- Non-recoverable slopes are embankment areas with slope ratios from 3:1 to 4:1 on which the vehicle will continue to the bottom of the slope.
- A traversable slope has a slope ratio between 3:1 and 4:1. Slopes in this range, if properly graded and clear of obstructions, will not allow the driver to recover control and steer back onto the roadway but will permit the vehicle to slow down and stop safely.
- A clear runout area follows a non-recoverable slope and is graded, shaped and made free of hazards (traversable) wide enough to allow an errant vehicle to safely stop.
- Critical slopes have a slope ratio of 3:1 or steeper and will require barrier treatment to protect an errant vehicle

Please note that in the Department's presentation of slope ratios it uses horizontal to vertical while the *Roadside Design Guide* uses the ratio of vertical to horizontal, e.g. DelDOT's 4:1 versus the *Roadside Design Guide's* 1:4.

The "forgiving roadside" concept recognizes that motorists do run off the roadway and that serious accidents and injuries can be lessened if at least a traversable recovery area is provided. The concept calls for a clear, unobstructed, relatively flat roadside area providing drivers an opportunity to recover control if their vehicle accidentally leaves the pavement surface. It may not be possible or practical to provide an area with flat slopes large enough to permit the driver to regain control of the vehicle. Where these areas can not be provided every attempt should be made to have an appropriate area clear of obstructions. The desired width of a project's clear zone varies based on several factors: (1) operating speeds, (2) traffic volume, (3) the steepness of slopes, (4) changes in slopes, (5) horizontal curvature, and (6) the accident history.

Any decisions on clear zone width obviously will influence the geometrics of the cross-section design, including design of side slopes. Since funds available for roadway improvements are limited, designers must consider the benefits and costs of alternate design treatments to provide the optimum clear zone design for any specific location. The proposed improvements for some projects do not take into consideration the clear zone based on the scope of work, such as minor improvements projects like pavement rehabilitation.

Table 3.1 of the *Roadside Design Guide* was developed to determine suggested roadside recovery area or clear zone distances for selected traffic volumes and speeds. The numbers are not precise since they are based on limited empirical data extrapolated to provide information for a wide range of conditions. Keep in mind

site-specific conditions, design speeds, rural versus urban locations, project scope and practicality. The clear-zone distances from Table 3.1 for horizontal curvature may be modified by using Table 3.2. These modifications are normally considered where accident history indicates a need or a specific site investigation shows definite accident potential which could be significantly lessened by increasing the clear zone width in a cost effective manner.

For relatively flat and level roadways, the clear-zone concept is simple to apply. However, it becomes somewhat less clear when the roadway is in a fill or cut section where roadside slopes may be either positive, negative, or variable, or where a ditch exists near the traveled way. Consequently, these features must be discussed before a full understanding of the clear zone concept is possible.

A basic understanding of the clear zone concept is critical to its proper application. As previously mentioned, the numbers obtained from Table 3.1 of the *Roadside Design Guide* are based on limited empirical data and extrapolated to provide information for a wide range of conditions. Thus, the numbers represent a reasonable measure of the degree of safety suggested for a particular roadside, but they are neither absolute nor precise. In some cases, hazards outside the clear zone may require removal or shielding depending on the severity of the hazard, the projected ADT, projected construction costs, and the classification of the roadway. The selection of an appropriate clear zone distance amounts to reaching a compromise between balancing user safety, construction costs, land use and social impacts, environmental concerns and the many other constraints that influence project decisions. Appropriate application of the clear zone concept will often result in more than one possible solution.

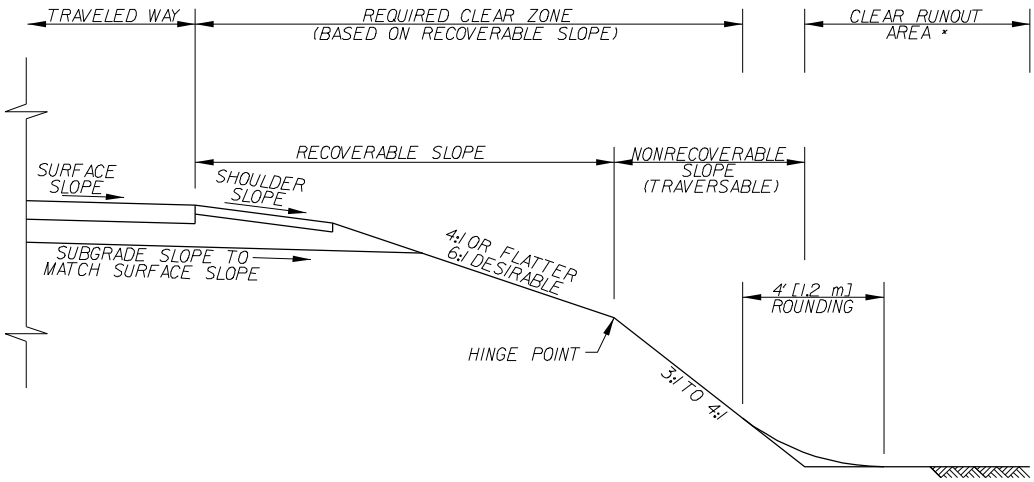
Chapter 3 of the *Roadside Design Guide* states: "The guidelines found in this chapter may be most applicable to new construction or major reconstruction." For other types of projects the guide recognizes that it may be not within the scope, not within the available funding, too environmentally disruptive, or impractical to achieve the recommended clear zone widths. Projects at this level are evaluated on an historical basis, identifying safety problems and obvious obstructions with emphasis placed on correcting these, if clear-zone related. TRB's *Special Report 214 Designing Safer Roads* should also be referred to when designing these types of projects.

In Delaware, experience has shown that the amount of lateral clear zone that should be provided varies from location to location. For each specific project a lateral clear zone is established by considering the following factors:

- Run-off-the-road accident experience,
- Design speed,
- Operating speed,
- Traffic volume,
- Steepness of side slopes,
- Profile grade,
- Horizontal curvature,
- Amount of roadside development,
- Sight distances,
- Level of improvement,
- Policy on removal and/or preservation of trees,
- Policy on installation of above ground utilities and
- Severity and location of the hazard.

After consideration of these factors and the clear zone requirements, the designer recommends a lateral clear zone or zones for each project. Clear zone widths generally will be uniform throughout the project except where widened for curvature. Lateral clear zone width decisions are an important design issue and are fully documented as described in Chapter 3.

Figure 4-3
Cross Section Side Slopes

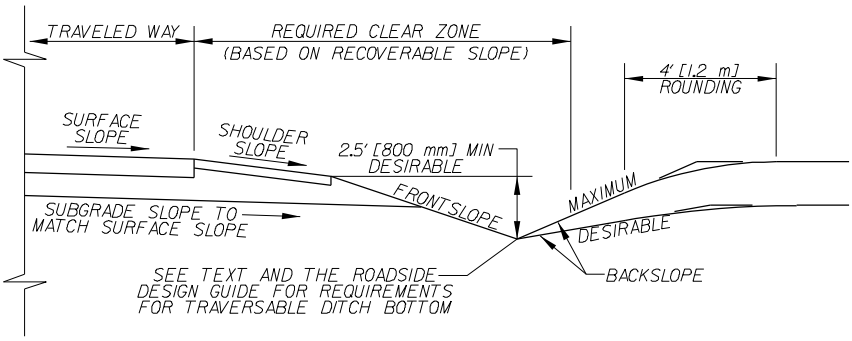


FILL SLOPES

* - THE CLEAR RUNOUT AREA IS ADDITIONAL CLEAR ZONE SPACE THAT IS NEEDED BECAUSE A PORTION OF THE REQUIRED CLEAR ZONE FALLS ON A NON-RECOVERABLE SLOPE. THE WIDTH OF THE CLEAR RUNOUT AREA IS EQUAL TO THAT PORTION OF THE CLEAR ZONE DISTANCE THAT IS LOCATED ON THE NON-RECOVERABLE SLOPE. THE CLEAR RUNOUT AREA SLOPE SHOULD BE RECOVERABLE OR AT LEAST TRAVERSABLE.

NOTES:

1. SEE TEXT SECTION 4.2.4.1 FOR DEFINITIONS OF CLEAR ZONE COMPONENTS.
2. A FILL SECTION USING A FRONTSLOPE CROSS SECTION THAT INCLUDES A NON-RECOVERABLE BUT TRAVERSABLE SLOPE WITHIN THE REQUIRED CLEAR ZONE IS COMMONLY REFERRED TO AS A "BARN ROOF DESIGN."



CUT SLOPES

Figure 4-4
Side Slope Criteria

Cut Slopes						
Road Class	**Front Slopes**		**Back Slopes**			
	Desirable	**Maximum**	**Desirable**	**Maximum** [1]		
				Depth of Cut		**Slope Ratio**
				US Cus-tomary	**Metric**	
Arterial	6:1	4:1	6:1	0 to 5 ft 5 to 10 ft 10 ft+	0 to 1.5 m 1.5 to 3 m 3 m +	4:1 3:1 2:1
Collector	6:1	4:1	4:1	0 to 5 ft 5 to 10 ft 10 ft +	0 to 1.5 m 1.5 to 5 m 3 m +	4:1 3:1 2:1
Local	6:1	4:1	4:1	0 to 5 ft 5 to 10 ft 10 ft +	0 to 1.5 m 1.5 to 5 m 3 m +	4:1 3:1 2:1

Fill Slopes						
Road Class	**Within Clear Zone**		**Outside Clear Zones**			
	Desirable	**Maximum**	**Desirable**	**Maximum**		
				Depth of Fill		**Slope Ratio**
				US Cus-tomary	**Metric**	
Arterial	6:1	4:1	6:1	0 to 5ft 5 to 10 ft 10 to 15 ft 15 ft +	0 to 1.5 m 1.5 to 3 m 3 to 5 m 5 + m	6:1 4:1 3:1 2:1
Collector	6:1	4:1	6:1	0 to 5 ft 5 to 10 ft 10 to 15 ft 15 ft +	0 to 1.5 m 1.5 to 3 m 3 to 5 m 5 m +	6:1 4:1 3:1 2:1
Local	4:1	3:1	4:1	0 to 3 ft 3 ft +	0 to 1.5 m 1.5 m +	3:1 2:1

Note:

Refer to the text and Tables 3.1 and 3.2 in the *Roadside Design Guide* for proper application of side slope design and clear zone requirements.

[1] The maximum back slope ratio outside the clear zone may be increased because of right-of-way restrictions.

In urban areas where curbs are often utilized, the space available for clear zones is generally restricted. In areas where barrier curbs are used, the clear zone shall extend to a minimum of 2 ft [0.6 m] beyond the face of curb, with wider clear zones provided where possible. In locations with mountable curbs, the clear zone width provided shall be as determined in *Roadside Design Guide*'s Table 3.1 as adjusted by Table 3.2.

If the clear zone width requirements, as determined by the above procedure, are not practicable because of local conditions or are inadequate because of specific safety problems, the designer should consider adjustments to the highway geometry or the installation of barriers. Refer to Chapter Ten for policies and criteria for barriers.

Utility poles are considered an obstruction and are not permitted within the clear zone without an engineering study, including accident history, and proper documentation. A design exception may be necessary. The use of breakaway utility poles can be considered as an alternative to moving or burying the utility where there is documented accident history.

4.2.4.2 FRONT SLOPES

Slopes within the selected clear zone must have a slope rate that is relatively flat. Normally a 6:1 slope (or flatter) should be used outward and downward from the edge of the finished shoulder to the outer limits of the lateral clear zone. Steeper slopes, up to a maximum of 3:1, may be used for low traffic volume roads and for conditions where flatter slopes would cause inordinately high costs. With slopes steeper than 4:1 the horizontal distance of the steeper slope cannot be used to meet the clear zone requirements.

For relatively low embankment heights, fill slopes extending outward and downward from the outer limits of the lateral clear zone to the natural ground normally should be the same.

Under conditions of high fills and/or right-of-way restrictions, steepening the fill slopes to a maximum of 2:1 beyond the clear zone may be considered, but they should be designed as flat as practical within the constraints of local right-of-way conditions. Slopes steeper than 3:1 cannot be mowed with conventional mowers. The cost of flattening slopes versus the cost of guardrail is also a consideration as guardrail is deemed a roadside obstruction and can be a continuing maintenance problem.

Slopes that parallel the traveled way can be recoverable, non-recoverable, traversable or critical. For recoverable slopes that are smooth and traversable with slopes of 4:1 or flatter, the suggested clear zone may be taken directly from the *Roadside Design Guide*'s Table 3.1, adjusted by Table 3.2 as necessary. Motorists who encroach on recoverable slopes can generally stop their vehicles or slow them enough to safely return to the roadway. Fixed object hazards, such as culvert headwalls, should not extend above the embankment either within the clear zone or beyond if the embankment is traversable to the bottom.

Slopes between 3:1 and 4:1 are considered non-recoverable slopes for which most motorists would be unable to safely stop or return to the roadway. It is very important that these embankments be traversable since a high percentage of encroaching vehicles will reach the toe of these slopes. The clear zone cannot logically end on the slope. Fixed object hazards should not be constructed along such slopes, and a clear runout area should be provided at the base of the slope. The runout area is a relatively flat clear area wide enough to allow the vehicle to stop. Figure 4-3 shows an example of such a clear zone. The clear zone width is the sum of the widths of the shoulder, any recoverable slopes, and the clear runout area, but excludes the non-recoverable slope.

Front slopes steeper than 3:1 are considered critical slopes and a driver will most likely lose control. If a slope steeper than 3:1 begins closer to the traveled way than the suggested clear zone for that specific roadway and the slope

cannot readily be flattened, a barrier may be warranted. Barrier warrants are included in Chapter Ten-Miscellaneous Design.

In developing the proposed roadway cross section, the design cross section may consider a shape that resembles and is referred to as a "barn roof" section, (see Figure 4-3). This design provides a relatively flat recovery area adjacent to the roadway for some distance, followed by a steeper traversable down slope. This cross section may be more economical than providing a continuous flat slope from the edge of the traveled way to the original ground line and is generally perceived as safer than a continuous steeper slope.

In cut sections, the slope extending outward and downward from the finished shoulder to the side ditch should desirably be 6:1. For low traffic volume roads and unusual local conditions, the front slopes may be as steep as 3:1. On federal-aid projects the "desirable" criteria shown in Figure 4-4 shall be used. It is desirable that the front slope extend outward far enough to provide a side ditch flow line elevation at least 2.5 feet [800 mm] below the elevation of the paved or finished shoulder. The purpose of this minimum ditch depth is that it will place the ditch bottom below a normal pavement box allowing any moisture trapped under the pavement to travel through a porous subbase into the ditch. In addition, when combined with the proper slope, it will provide a depth that allows for some temporary ponding but will quickly remove any roadway runoff before it saturates the pavement structure.

4.2.4.3 BACK SLOPES

Back slopes extending upward and outward from side ditches to intersect the natural ground desirably should be 6:1 for principal arterials, but may be slightly steeper for lower classes of roads.

Under conditions of deep cuts and/or right-of-way restrictions, steepening the back slopes to a maximum of 2:1 may be considered, depending on the depth of cut and the class of road. Recommendations for slopes in rock cuts will be made for individual projects based on studies of local conditions.

When a roadway is in a cut section, the back slope may be hazardous depending upon its relative smoothness and the presence of fixed object hazards. If the slope is traversable (3:1 or flatter) and obstacle-free, it may not be a significant hazard. However, steep back slopes or those with obstacles such as rock cuts within the clear zone may require shielding if they cannot be flattened or the obstacle removed. Warrants for barriers are discussed in Chapter Ten.

4.2.4.4 TRANSVERSE SLOPES

Common obstacles along roadsides are transverse slopes created by median crossovers, drainage structures, driveways and intersecting side roads. These are generally more critical to errant motorists than front slopes or back slopes because they are typically struck by run-off-the-road vehicles travelling parallel to the roadway and impact the feature head on. Transverse cross slopes of 6:1 or flatter are suggested for high-speed roadways, particularly for the section of the embankment that is located immediately adjacent to traffic. This slope can then be transitioned to a steeper slope as the distance from the traveled way increases.

Embankment slopes (including the ends of any drainage structures) of 10:1 are desirable; however, their practicality is limited by width restrictions and the maintenance problems associated with long tapered pipe ends. Embankment slopes that are steeper than 6:1 may be considered for urban areas or for low-speed facilities. Safety treatments of drainage structures are discussed in Chapter Six-Drainage.

4.2.5 ROADSIDE DITCHES

The two principal functions of roadside ditches (hydraulically defined as open channels) are: (1) to drain water from the subgrade and (2) to collect surface water either from the roadway

surface or adjacent roadside areas and remove it before entering the subgrade. Moisture in the subgrade and the frequency and magnitude of pavement loads are the most destructive forces to the roadbed and pavement structure. In addition, roadside ditches are an important element in reducing the environmental impact of a project on the adjacent landscape. Ditch designs can play a major role in managing stormwater runoff, removal of sediment, controlling erosion, and reducing the impact of roadway pollutants on watercourses.

Insofar as practical, ditch cross sections should be traversable within the clear zone. Figures 4-5 and 4-6 show preferred front slopes and back slopes for basic ditch configurations. Cross sections that fall within the shaded area of each figure are considered to have traversable cross sections. Ditch sections that fall outside the shaded area are considered less desirable; their use should be limited where high-angle encroachments, such as the outside of relatively sharp curves, can be expected. Ditch sections outside the shaded area may be acceptable for projects with one or more of these characteristics: restrictive right-of-way, rugged terrain, low traffic volume low, operating speed, or projects involving resurfacing, restoration, or rehabilitation, particularly if the ditch bottom and back slopes are traversable and free of any fixed objects. If practical, ditches with cross sections outside the shaded areas and in vulnerable locations may be re-shaped and converted to a closed system or shielded with traffic barriers.

Side ditches are particularly important to control surface drainage through cut sections in order to maintain the design integrity of a freely draining pavement structure. If the excavated material is of adequate quality it usually is used in the construction of adjacent fill sections. Figure 4-3 indicates that ditches may not be required at the toes of fill sections to drain the subgrade. However, in addition to the previously discussed environmental concerns, there may be other reasons to provide ditches to control runoff at the toes of fills to carry the flow to natural drainage channels thereby minimizing real or perceived damage to adjacent properties.

The two commonly used geometric configurations for side ditches are trapezoidal and v-ditch.

4.2.5.1 TRAPEZOIDAL DITCH

The preferred design is a ditch that is trapezoidal in shape with relatively flat front slopes and back slopes and a wide, flat bottom. (See Figure 4-5.) The general configuration of the trapezoidal ditch section does graphically show sharp breaks at the intersection points. However, constructing these breaks in the field is not always practical and they are normally graded in a more rounded shape making this ditch type more easily traversable than most other shapes.

4.2.5.2 V-DITCH

The V-ditch is a less desirable ditch design. Safety features are reduced because of the sharp break in the slope between the front slope and back slope. (See Figure 4-6.) This type of ditch section is more easily constructed and requires less right-of-way. However, it is not the best choice for traversability or maintenance.

4.3 MEDIANS

Medians are provided on divided multi-lane highways to provide a separation of opposing traffic lanes, a recovery area for out-of-control vehicles and an area for emergency stops. Besides these safety benefits, medians also can provide space for:

- Left-turn lanes,
- Snow storage,
- Collecting surface drainage,
- Refuge for pedestrians at crosswalks,
- Installation of traffic control devices, and
- Adding future lanes.

Figure 4-5
Trapezoidal Ditch Section

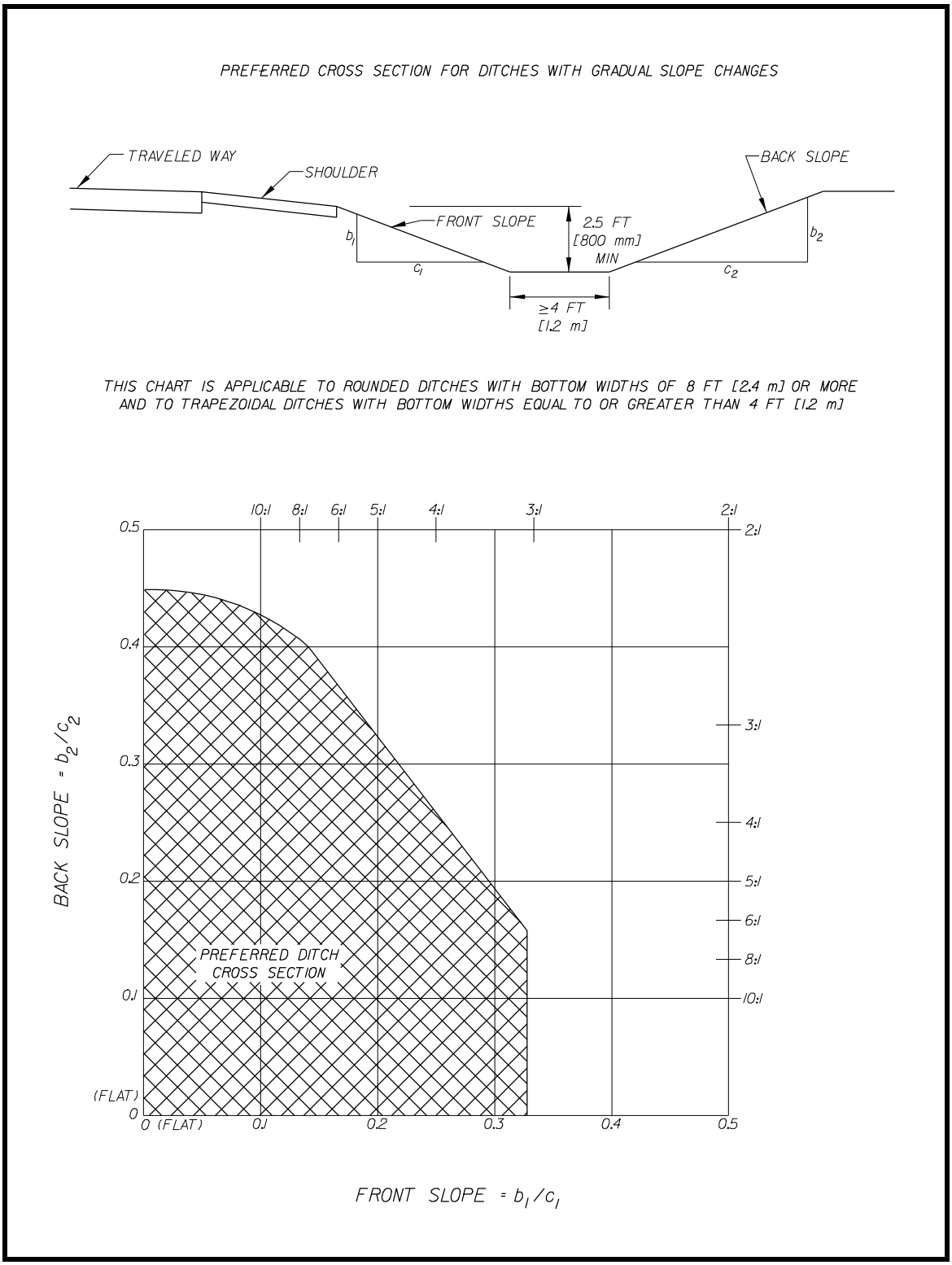
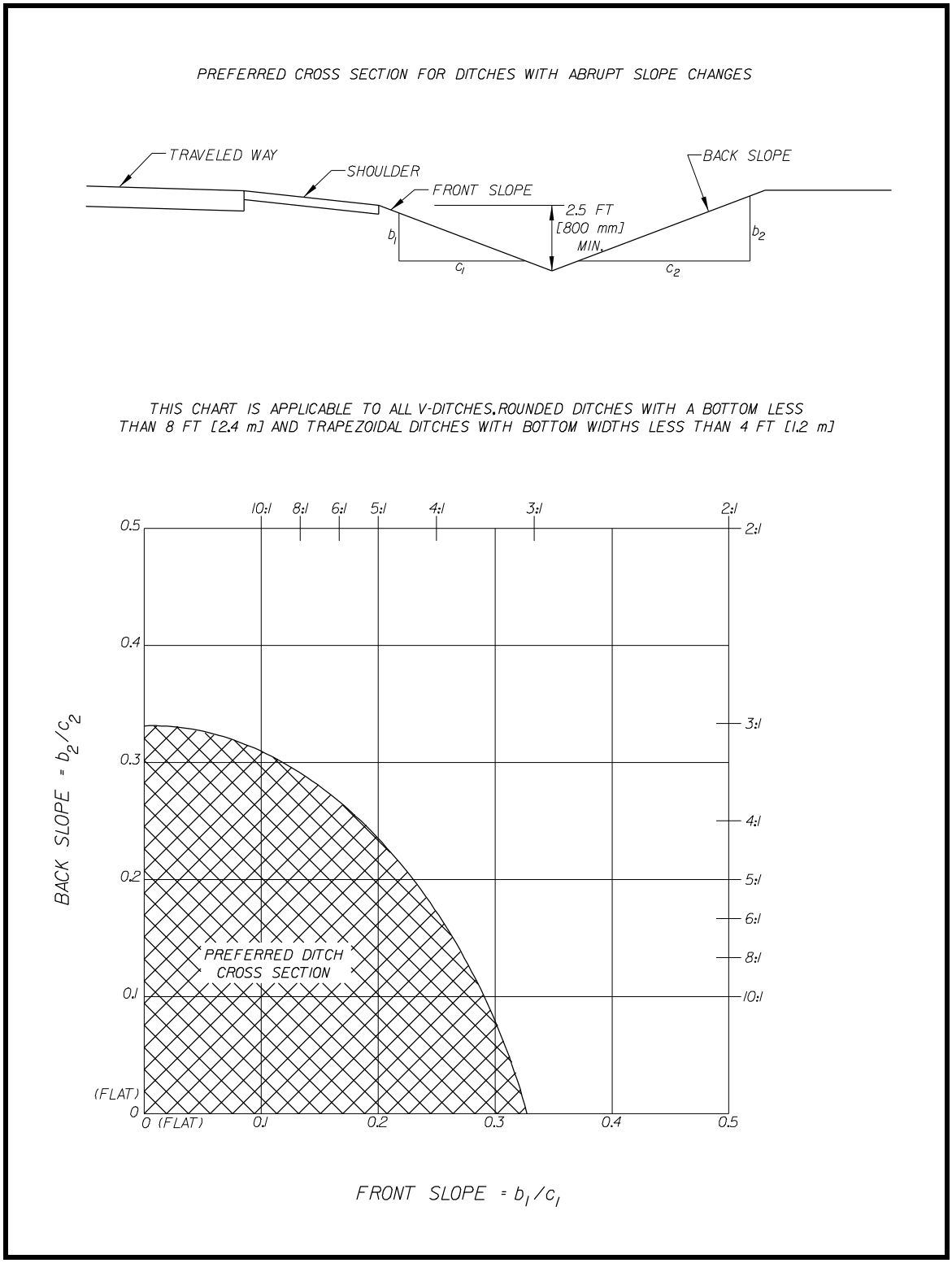


Figure 4-6
V-Ditch Section



Median widths are always measured between the inside edges of opposing travel lanes. Medians operate best when they are highly visible during the day or night and are at a width that provides for the predominant usage. There are three basic types of medians:

- Flush medians,
- Curbed (raised) medians, and
- Depressed medians.

The use of medians in providing for U-turn movements, auxiliary lanes and intersection design is further discussed in Chapter Seven-Intersections.

4.3.1 FLUSH MEDIANS

Flush medians consist of a relatively flat paved area separating the traffic lanes with only painted stripes on the pavement. This type is generally used only for lower-speed urban arterials. Painted medians need frequent repainting to maintain their visibility at night and under inclement weather conditions.

To accommodate painted left-turn channelization, flush medians should be at least 16 ft [4.8 m] wide and desirably 18 ft [5.4 m]. Flush medians should be either slightly crowned to avoid ponding of water in the median area or slightly depressed (with median drains) to avoid carrying all surface drainage across the driving lanes.

4.3.2 CURBED MEDIANS

Curbed raised medians are most commonly used on lower-speed urban arterials. They have the same basic advantages and characteristics of flush medians except the separation is more clearly defined than for painted lines, do not need frequent repainting and are more easily seen at night and during inclement weather.

Typical widths of raised medians range from 4 to 22 ft [1.2 to 4.1 m]. A raised median of 4 to 6 ft [1.2 to 1.8 m] in width with a paved surface may be used under restricted conditions on urban streets, but they have limited advantages. Although they provide a positive separation between opposing traffic and an opportunity to collect drainage, they offer no opportunity to introduce left turn lanes, are too narrow to provide a desirable pedestrian refuge and do not adequately serve as an area for installing traffic control devices.

The absolute minimum median width is 12 ft [3.6 m] for introducing left-turn lanes on low-speed arterial streets with restricted conditions and minimal truck use. Any size truck (as well as many passenger car drivers) could not use this lane without infringing on the adjacent travel way. A median width of 16 ft [4.8 m] is the normally accepted minimum in urban areas to adequately serve a mix of drivers and vehicles without having erratic movements. This width provides for a 10 ft [3.0 m] turn lane and a 6-ft [raised median]. This width does not provide any curb offset so there will be a tendency for drivers to shy away from the median into the adjacent travel way.

The two preferred urban median widths, where frequent left turns are to be accommodated with a diverse traffic mix, are 20 ft [6.0 m] or 22 ft [6.6 m]. A 20 ft [6.0 m] median width allows for a 12 ft [3.6 m] left turn lane, 2 ft [0.6 m] clearance from the edge of traffic lanes to the face of the curbed island, and a 4 ft [1.2 m] wide island to provide space for traffic control devices. However, in high pedestrian use areas, the preferred width is 22 ft [6.6 m] that will allow for a 6 ft [1.8 m] raised median for pedestrian refuge.

4.3.3 DEPRESSED MEDIANS

Depressed medians are most commonly used for high-speed expressways, freeways and rural arterials. Depressed medians are uncurbed grass areas with flat slopes drained by open ditches and flush drainage inlets. Normally, the widths of depressed medians are considerably greater than for either flush medians or raised medians. Smoother traffic operations and improved traf-

fic safety are observed advantages of wide, depressed medians.

Designing a relatively narrow depressed median creates problems. The result is that the longitudinal drainage ditch in the center of the median is too shallow, or the transverse slopes from the roadways to the ditch are too steep.

Median side slopes of 6:1 or flatter, for a distance of at least 30 ft [9 m] from the edge of the traffic lanes, are preferred. Other median slopes (for median crossovers, ditch blocks, etc.), that might be in the path of an out-of-control vehicle, should be 6:1 as a minimum and preferably 10:1 or flatter as a safety feature.

A width of 40 ft [12.0 m] or more for depressed medians permits adequate drainage design with flat slopes. A median width of at least 50 ft [15.0 m] can safely store a school bus. Wider medians are desirable where right-of-way permits allowing for the placement of a median bridge pier or overhead sign structure without the need for barrier protection. Wider medians should also be considered where there is a potential for adding travel lanes in the median to meet future traffic demand. Also see the *Green Book* pages 460 and 461 for further discussion on this subject.

Where flat longitudinal slopes on the roadway are encountered, the cross slopes of the median may be varied to increase the longitudinal slope of the median ditch. For example, the cross slope may be kept very flat (10:1 or flatter) at the upper end of the drainage area and steeper (6:1) at the lower end.

4.3.4 MEDIAN BARRIERS

For divided highways with large traffic volumes and high operating speeds, a wide, depressed median is the best choice. Under some conditions this is not practicable, and a flush or raised median must be provided. But in this case, some type of physical barrier must be placed in the median to prevent out-of-control vehicles from crossing into opposing traffic lanes.

Several types of physical median barriers can be designed. Refer to Chapter 10 and the *Roadside Design Guide* for criteria for median barriers.

4.3.5 MEDIAN OPENINGS

Refer to Chapter 7 for the design of median openings and channelization for left turns.

Chapter Five

Alignment and Superelevation

A facility's horizontal and vertical alignments establish the general character of a roadway, perhaps more than any other design consideration. The configuration of line and grades affects safe operating speeds, sight distances, opportunities for passing, and highway capacity. Decisions on alignment have a significant impact on construction costs, and social and environmental issues. Alignment is defined by several factors, including: the length of tangent sections; the transition into horizontal curves; the degree of curvature (radius) for horizontal curves; the transition out of the curves; the rate of superelevation applied to the horizontal curves; and the rate of grade change for any vertical curves.

Basic design controls and standards related to alignment were presented in Chapters Two and Three, including criteria for design speed, maximum curvature, superelevation and sight distances. This chapter provides more detailed explanations and discusses practical application of the criteria.

5.1 HORIZONTAL ALIGNMENT

Horizontal alignment of a roadway is defined graphically using a series of straight-line tangents with transition sections into and out of horizontal curves. Many factors, including terrain conditions, physical features and right-of-way considerations, affect the design of tangent and curve sections. The two types of curves used in designing horizontal alignments are simple circular curves (a single constant radius) and compound curves (a series of symmetrical or asymmetrical radii). The two methods of transitioning from a tangent section to full curvature are tangent-to-curve and spiral

For most horizontal alignment designs, the Department has adopted the use of simple circular curves and the tangent-to-curve transition method. Placement of the superelevation runoff length is an important design consideration when using the tangent-to-curve transition method. The location where the runoff length begins and ends has an effect on a vehicle's lateral velocity and motion. The result can be operational problems due to driver tendency to over steer to compensate for the increase in side friction; see Section 5.3.3 for more detailed discussion.

5.1.1 GENERAL CRITERIA

Design speed is the principal factor controlling horizontal alignment design. Several geometric standards related to design speed are very specific. Other criteria cannot be defined as specifically and require that judgments be made in consideration of local conditions. The following guidelines outline some of these decisions.

- Alignment should be consistent and as directional as possible. Frequent changes in alignment with many short curves should be avoided except in cases of very low design speed and low traffic volume in rough terrain. Sharp curves should not be introduced at the ends of long tangents or flat curves. Sudden changes from areas of slight curvature to sharp curvature should be avoided. Where sharp curvature must be introduced, it should be gradually approached by successively sharper curves.

- Avoid the use of the minimum curve radii shown in Figures 5-1 and 5-2. Generally, curves should be as flat as practicable for the conditions. On the other hand, avoid the use of excessively long, flat curves on two-lane highways where considerable passing opportunities are needed. Many drivers are reluctant to pass on a curve, even though the sight distance may be adequate. It may be better to use a shorter curve, thus lengthening the adjacent tangent section and increasing the passing opportunity.
- A horizontal curve is not required for radii of deflection angle of 0.25° or less. Curves with a small deflection angle should be long enough to avoid the appearance of a "kink."
- The minimum length of horizontal curves on primary roadways should be about 15 times the design speed in mph [3 times the design speed in km/h]. On high-speed controlled access highways, a desirable minimum length of curve would be about 30 times the design speed in mph [6 times the design speed in km/h].
- Care should be used in the design of compound curves. Preferably, their use should be avoided where the curves are sharp. Compound curves with large differences in curvature introduce the same problems that arise with a tangent approach to a circular curve. Where compound curves must be used, the radius of the flatter curve should not be more than 50 percent greater than the radius of the sharper curve.
- Avoid abrupt reversals in reverse curve alignments by providing enough tangent distance between the curves to ensure adequate superelevation transition for both curves and sufficient distance for adequate signing.
- Avoid "broken-back" curves (short tangent sections between two curves in the same direction). Use of spiral transitions, compound curves or a single longer curve is preferable because they provide some degree of continuous superelevation. When broken-back curves are necessary, there should be a tangent distance 500 to 1500 ft [150 to 450 m] between the curves, depending on the design speed.
- On long, high fills, tangents should be used wherever feasible. If curvature is needed, it should be kept as flat as possible. Without visual "cues" such as cut slopes, shrubs and trees above the roadway, it is difficult for drivers to perceive the extent of curvature and adjust their operation to the conditions.

It is desirable that bridges be located on tangent sections of the alignment. When it is necessary to locate a bridge on a curve, care should be used to avoid beginning or ending a curve on the bridge. This can be hazardous under slippery surface conditions, and also adds complications to bridge design and construction. Where curves are necessary on road sections near bridge ends, the beginnings and endings of curves should be located so that no portion of the superelevation transition extends onto the bridge.

5.1.2 CONTROL LINE LOCATIONS

The relationships between the construction baseline and the profile grade must be clearly identified on the typical section sheets. The following general criteria should apply:

1. For two-lane roads and undivided multi-lane roads, the horizontal and vertical controls should be located at the centerline of the typical section.
2. For multi-lane divided highways with relatively narrow medians, the horizontal control should be at the centerline of the median, and a single profile grade should be located at the median edge of pavements for both traveled ways.

Figure 5-1
Minimum Radius for Open Highway Conditions and Superelevation Rate of 4%

US Customary		Metric	
Design Speed (mph)	Radius (ft)	Design Speed [km/h]	Radius [m]
15	42	20	8
20	86	30	22
25	154	40	47
30	250	50	86
35	371	60	135
40	533	70	203
45	711	80	280
50	926	90	375
55	1190	100	492
60	1500		

Figure 5-2
Minimum Radius for Open Highway Conditions and Superelevation Rate of 6%

US Customary		Metric	
Design Speed (mph)	Radius (ft)	Design Speed [km/h]	Radius [m]
15	39	20	8
20	81	30	21
25	144	40	43
30	231	50	79
35	340	60	123
40	485	70	184
45	643	80	252
50	833	90	336
55	1060	100	437
60	1330	110	560
65	1660	120	756
70	2040	130	951

3. For multi-lane divided highways with independent roadways or relatively wide medians, independent horizontal and vertical controls are established at the centerline of each roadway.

The relationships between these control line locations and the pivot points for superelevation of horizontal curvature are described in Section 5.3.

5.1.3 TYPES OF CURVES

The types of curves used in designing horizontal curvature may be simple circular curves, spiral transition curves or compound curves. Circular curves use a uniform radius for the entire distance between adjacent tangent sections. Spiral transition sections more closely replicate the vehicle and driver's behavior when entering a curve. They are introduced at each end of the circular curve to gradually ease the driver into and out of curves without a sharp break at the tangent sections. This is particularly noticeable with relatively sharp curves and higher vehicle operating speeds. Compound curves are most commonly used for turning roadways where it is necessary to fit the curve to the inside edge of the design vehicle's swept path. When the design speed of a turning roadway is 45 mph [70 km/h] or less, compound curvature can be used to form the entire alignment of the turning roadway. However, the exclusive use of compound curves can increase the right-of-way impacts.

Although circular curves are normally used in the design of Delaware roadways, using spiral transitions may be considered as described in the Green Book, pages 184 to 192.

Where spiral transition curves are to be used, right-of-way lines should not be defined as a spiral curve paralleling the centerline. Instead, the right-of-way should be described with a circular curve or compound circular curve of a similar shape. A practical guide for the length of a spiral is the length required for superelevation runoff.

5.1.4 SIGHT DISTANCE ON HORIZONTAL CURVES

An important element in ensuring driver safety and maintaining a roadway's operational efficiency is providing adequate sight distance—the length of roadway ahead visible to the driver. Sight distance applies to four conditions that may arise when setting a project's horizontal alignment:

(1) Is adequate distance available to stop?

(2) Is there adequate opportunity and length available for passing on two-lane roadways?

(3) Is there adequate distance for drivers to react when approaching complex decision points?

(4) Has the selected criteria for measuring these distances been applied to the selected design?

Providing adequate sight distance is also important in the design of intersections, in particular, those in rural areas. These locations tend to be less safe than urban ones, primarily because of higher speeds and lack of driver awareness. Providing at least the minimum sight distance will play an important role in reducing these occurrences.

5.1.4.1 STOPPING SIGHT DISTANCE

The designer must check sight distance across the inside of horizontal curves. Sight obstructions such as walls, concrete safety barriers, bridge parapets, cut slopes, vegetation and buildings may limit sight distance on curves. Where these obstructions cannot be removed or permanently controlled, adjustment in the normal cross section or a change in alignment may be required to provide and assure continuation of adequate sight distance. For areas within a project that may cause confusion or delay a driver's reaction time i.e. multiple decision points, it may be necessary to check the decision sight distance also.

Minimum stopping sight distance for each design speed is shown in Chapter Three-Design Standards. The sight line is a chord of the curve. The applicable stopping sight distance is checked by measuring along the centerline of the inside lane around the curve. See the Green Book, pages 224-228 for the design and evaluation of stopping sight distances on horizontal curves. Horizontal sight distance is based on the formula:

$$HSO = R\left[\left(1 - \cos\frac{28.65\,S}{R}\right)\right]$$

Where:

S = Stopping sight distance, ft [m]
R = Radius of curve, ft [m]
HSO = Horizontal sightline offset, ft [m]

Where the obstruction is a cut slope on the inside of the curve, it is necessary to know the critical height of vegetation on the slope for measuring the middle ordinate distance. Because the height criteria for stopping sight distance are 3.5 ft [1,080 mm] for the eye and 2 ft [600 mm] for the object, a height of 2.75 ft [840 mm] may be assumed as the midpoint of the line of sight where the cut slope usually obstructs sight. In some cases, retaining walls, concrete median safety barriers, and other similar features constructed on the inside of curves may be sight obstructions and need to be checked for stopping sight distance.

Solutions to sight distance problems on horizontal curves might be removal of obstructions, flattening the curves and flattening or benching cut slopes. It should be kept in mind that stopping sight distances greater than the minimum should be used for design. Minimum stopping sight distance values may be used only if greater values cannot be obtained without undue costs. On new construction, the stopping sight distance at any loca-

tion shall never be less than the minimum standard for stopping sight distance for the selected design speed. Designs for new construction and reconstruction projects that do not meet these standards must have a design exception approved by the Chief Engineer.

5.1.4.2 PASSING SIGHT DISTANCE

The minimum passing sight distance for a two-lane road is about four times greater than the minimum stopping sight distance at the same design speed. To provide the greater passing sight distance, clear sight areas on the insides of curves must be considerably wider. Often this is not practicable. It is necessary to acknowledge and accept no-passing zones.

Passing sight distance depends on the eye height of 3.5 ft [1,080 mm] and object height of 3.5 ft [1,080 mm]. The sight line to the center of the area inside a curve is about 0.75 ft [240 mm] higher than the stopping sight distance.

Perhaps the simplest way to measure passing sight distance is directly from the plans, using a straightedge. Potential obstructions are plotted on the plans. In the case of cut slopes, a dotted line is plotted for the horizontal distance from the centerline of the inside lane to a point on the cut slope 4 ft [1.2 m] above the traffic lane. Because vegetation also blocks vision, its anticipated height must be included in the 4 ft [1.2 m]. The straight edge is placed along the edge of the obstruction (or dotted line), and the intersection with the centerline identifies the sight distance.

Where horizontal curves and vertical curves occur at the same general location, the sight distances for each must be considered together. At least the minimum stopping sight distance must be provided for each. Efforts to provide passing sight distance for one might be completely negated by a no-passing zone situation for the other.

For more information see the Green Book, pages 118 to 131.

5.1.4.3 DECISION SIGHT DISTANCE

Drivers frequently are called on to make decisions concerning vehicle operations. Occasionally, the characteristics of the horizontal alignment can adversely affect the ability to make these decisions. Examples of this include:

- ***Proximity to a Curve***. It is important that the driver has a complete or partial view of the curve ahead to indicate the direction of curvature. With some combinations of vertical and horizontal curvature, the curve may come as a surprise and the driver may have difficulty reacting properly.
- ***Curve Signing***. To be effective, curve signing must be located a considerable distance ahead of the curve. The use of short tangents between curves results in inadequate length for proper signing. Where the design speed of the curve is equal to or greater than the legal posted speed, the length of the tangent should be at least 300 ft [90 m] plus the required distance for superelevation transition.
- ***Route Continuity***. When a driver approaches a diverging roadway situation, such as a Y intersection, an exit ramp on a curve, or a flat-angle intersection, the main route should be distinctly emphasized with sufficient sight distance to eliminate any uncertainty on the part of the driver.

The Green Book, pages 115-117, provides more details and tables of calculated values for checking decision sight distance.

5.1.5 COORDINATION WITH VERTICAL ALIGNMENT

Curvature and grades should be in proper balance. Emphasis on a tangent alignment is not desirable when it results in extremely steep or long grades. An emphasis on flat grades is not desirable when it results in excessive curvature. A compromise between the two extremes is the best approach.

Several general criteria should be kept in mind:

- Sharp horizontal curvature should not be introduced near or just beyond the top of a pronounced crest vertical curve. This condition makes it difficult for drivers to perceive the horizontal change in alignment, especially at night.
- Sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve. This is aesthetically undesirable and can be hazardous because vehicle-operating speeds, particularly for trucks, often are higher at the bottoms of grades.
- On two-lane roads and streets with considerable traffic volume, safe passing sections must be provided at frequent intervals and for an appreciable percentage of the length of roadway. In these cases, it is necessary to work toward long tangent sections to secure sufficient passing sight distance rather than the more economical combination of vertical and horizontal alignment.
- Both horizontal curvature and the profile should be as flat as feasible at intersections where sight distances along both roads and streets is important and vehicles may have to slow or stop.

5.2 VERTICAL ALIGNMENT

The profile grade line defines the vertical alignment for construction in terms of straight grades and parabolic curves. This section provides guidelines and criteria for the design of profile grades.

5.2.1 GENERAL CRITERIA

As with other design elements, the characteristics of vertical alignment are influenced greatly by basic controls related to design speed, traffic volumes, functional classification and terrain conditions. Within these basic controls, several generally accepted criteria must be considered.

- Use a smooth grade line with gradual changes, consistent with the type of highway and character of terrain, rather than a line with numerous breaks and short lengths of tangent grades.
- The “roller coaster” or “hidden dip” type of profile should be avoided. Often they are proposed in the interest of economy, but they are aesthetically undesirable and extremely hazardous. A driver cannot avoid or compensate for a hazard that cannot be seen.
- Avoid “broken back” grade lines (two crest or sag vertical curves separated by a short tangent). One long vertical curve is more desirable.
- Avoid very long crest vertical curves if passing sight distance cannot reasonably be attained. A shorter vertical curve may permit more passing opportunity on adjacent tangent grades.
- On a long grade it is preferable to place the steepest grade at the bottom and flatten the grade near the top.
- Maintain moderate grades through intersections to facilitate turning movements. For the design of vertical alignment through intersections, refer to Chapter Seven-Intersections.
- Consider auxiliary lanes where passing opportunities are limited and it is probable that slow–moving vehicles will affect operating speeds and the desired level of service.

5.2.2 MAXIMUM GRADES

Criteria for maximum grades are based mainly on studies of the operating characteristics of typical heavy trucks. Although design values have been determined and agreed upon for many highway features, few conclusions have been reached on roadway grades in relation to design speed.

Refer to the Green Book for the maximum grades permitted for various combinations of functional classification, traffic volume and terrain. The maximum grades should be used only where absolutely necessary. Grades much flatter than maximum normally should be used.

For short grades less than about 500 ft [150 m] in length, the maximum gradient may be one percent steeper than the values shown in the tables.

5.2.3 MINIMUM GRADES

Minimum grades are primarily related to the need for adequate drainage. For uncurbed pavements that are adequately crowned to drain laterally, relatively flat or even level profile grades may be used. With curbed pavement, the minimum longitudinal grade in usual cases should be 0.5 percent. With a high-type pavement accurately crowned on a firm subgrade, a longitudinal grade of about 0.35 percent may be used. Even on uncurbed pavements, it is desirable to provide a minimum of about 0.35 percent longitudinal grade because the lateral crown slope originally constructed may subsequently be reduced as a result of irregular swell, pavement structure consolidation, maintenance operations or resurfacing. Use of flatter grades may be justified in special cases.

5.2.4 MINIMUM DITCH GRADES

Special attention should be directed to minimum ditch grades. Any ponding of water in the side ditches, particularly on expansive soils, has a very detrimental effect on the subgrade. To ensure continuing flow, ditch grades should be sloped at least 0.5 percent–preferably steeper. This may require some special warping of ditch grades where the roadway profile cannot be adjusted accordingly. A minimum depth of ditch has been established at 2.5 ft [800 mm] below the elevation of the hinge point between the shoulder and frontslope to assure proper drainage of pavement base and subgrade. In superelevated sections both the ditch grade and bottom width may have to be adjusted in order to prevent water ponding onto the shoulder or traveled way.

5.2.5 CRITICAL LENGTH OF GRADE

From the standpoint of vehicle operating characteristics and the effect on highway capacity, the steepness of the grade is not the only factor to be considered. The length of the grade can become a critical factor and must also be considered.

The term “critical length of grade” is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For a given grade, lengths less than critical ones result in acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than critical ones, design adjustments such as change in location to reduce grades or addition of extra lanes should be made. It is recommended that a 10 mph [15 km/h] speed reduction be used as the general guide for determining critical lengths of grades. The Green Book, pages 242 and 243, provides curves showing the critical lengths of grade resulting from various combinations of percent upgrade and designated speed reductions.

On roads with moderate to heavy traffic volumes, where critical lengths are approached or exceeded, and passing opportunities are limited, long lines of smaller vehicles will accumulate behind the slower vehicles. This reduces both the operating speed and highway capacity and, consequently, the level of service. Consideration should be given to providing climbing lanes. A capacity analysis should be conducted to determine whether the addition of a climbing lane is warranted. Procedures for such an analysis are shown in Chapter Ten of the *Highway Capacity Manual*. Factors considered in the analysis include:

- Desired level of service,
- Lane widths and lateral clearance,
- Percent of trucks and buses,
- Passing sight distance,

- Steepness and length of grades,
- Volume/capacity ratio, and
- Service volume.

5.2.6 CLIMBING LANE CRITERIA

The need for climbing lanes in Delaware is seldom warranted. The Green Book pages 241 to 250 gives a through explanation for the design of these lanes.

5.2.7 VERTICAL CURVES

Vertical curves are used to effect gradual changes between tangent grades at their point of intersection. They have the properties of a simple parabolic curve. The vertical offsets from the tangent grade vary with the square of the horizontal distance from the curve end (point of tangency).

Vertical curves that are offset below the tangent are termed crest vertical curves. Those that are offset above the tangent are termed sag vertical curves. Examples of each curve type are shown in Figure 5-4.

The minimum lengths of crest vertical curves are determined mainly by sight distance requirements. These lengths generally are satisfactory from the standpoint of safety, comfort and convenience. An exception may be at decision areas, such as intersections and approaches to ramp exit gores, where adequate sight distance requires longer lengths.

Passing sight distance seldom can be attained on a crest vertical curve simply by lengthening the curve. Excessively long vertical curves often reduce the length of passing opportunities on the adjacent tangent sections on either side of the crest. They also can adversely impact roadway and roadside ditch drainage systems.

Sag vertical curves use four different criteria for determining their lengths:

(1) headlight sight distance,

(2) passenger comfort,

(3) drainage control, and

(4) general appearance.

The primary control used in design is headlight sight distance.

5.2.8 VERTICAL CURVE DESIGN

The principal concern in designing vertical curves is to ensure that at least the minimum stopping sight distance is provided. The values set forth in the design standards for stopping sight distance are also applied to vertical curves. Refer to the Green Book pages 265 to 280 for more design detail.

For crest vertical curves, the design eye height is 3.5 feet [1,080 mm] and the object height is 2.0 ft [600 mm]. The crest of the curve should not obstruct the line of sight.

Nighttime driving conditions govern sag vertical curves. The sight distance control is the height of headlight and the distance illuminated to an object rather than driver eye height. The distance illuminated is that of a headlight beam with an assumed upward divergence of 1 degree and headlight mounting height of 2 ft [600 mm]. Equations found in the Green Book are used to determine these values for various design speeds. For overall safety, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. The values in Figure 5-6 were developed using the design stopping sight distance as the light beam distance.

For passing sight distance, the controls are different than for stopping sight distance. The design height of the eye remains at 3.5 ft [1,080 mm], but the height of the object (oncoming car) is increased to 3.5 ft [1,080 mm].

By analyzing the requirements relating to sight distances and the characteristics of the curve, determinations can be made as to the minimum permissible length of curve for particular situations. A ride control criterion for vertical curve length of not less than three times the design speed in mph [0.6 times the design speed in km/h] is recommended for comfort.

The minimum length of a vertical curve is computed by the following formula:

$$L = KA$$

Where

L = minimum length of vertical curve in feet [meters],

A = algebraic difference in grades in percent, and

K = a constant value for the design speed.

The rate of change of grade along a vertical curve is constant, and is measured by dividing the algebraic difference between the grades by the length of the curve in feet [meters]. This value gives the percent change in grade per horizontal feet [meters] of curve.

The reciprocal of this value represents the horizontal distance required to effect a one percent change in the gradient along the curve. The expression L/A is termed K, and is useful for determining minimum lengths of vertical curves. Based on the geometrics of each sight distance condition and assumption, formulas are used to compute values of K for each design speed.

Established values of K are shown in Figure 5-5 for the calculated minimum stopping sight distance. Similar K values are shown as minimum criteria for passing sight distance. Curve lengths should be based on greater than minimum stopping sight distance wherever possible. Usually the length is rounded off to the next multiple of 100 ft [30 m].

Generally, crest vertical curves in rural areas should be longer than the minimum to avoid the appearance of an angular break in the alignment, even though sight distance criteria may permit a shorter curve. On long sag vertical curves, special attention may be required for drainage features.

5.2.9 PASSING SIGHT DISTANCE MEASUREMENT

Vertical curves designed in accordance with the criteria in the preceding section will, in all cases, provide adequate stopping sight distance. But often it is not practicable to provide passing sight distance. Passing sight distances are 4 to 5 times the corresponding length for stopping sight distance. On some projects with relatively high traffic volumes, the lack of sufficient passing opportunities can cause problems with traffic operations and levels of service. A key factor in identifying these potential conditions is the percentage of passing sight distance 1,500 ft [450 m] or greater available within the limits of the roadway section.

A practical approach for this determination is to measure the length of all “non-passing” locations (sight distances less than 1,500 ft [450 m]) and subtract this length from the total length of the roadway section. The remainder is available for passing and is the basis for computing the percent of passing sight distance. Separate calculations are needed to check both directions where roadways carry two-way traffic.

Measurements of vertical curve non-passing lengths can be made directly from the profile using a straightedge and the prescribed criteria of 3.5 ft [1,080 mm] for the height of the eye and 3.5 ft [1,080 mm] for the height of the object. These measurements should be coordinated with similar measurements of horizontal sight distance restrictions before computing the percent available passing sight distance. As a general guide, the restricted passing lengths (less than 1,500 ft [450 m] sight distance) should not exceed the values shown in Figure 5-3 or the level of service will drop below an acceptable level.

Figure 5-3
Restricted Passing Sight Distance Criteria

ADT	Percent Restricted Passing
Under 250	70
250 - 500	60
500 – 1000	40
1000 – 1500	20
Over 1500	10

5.2.10 GRADELINE ELEVATIONS

For many reconstruction projects in urbanized areas there is little opportunity for any major variation in gradeline elevations. The elevation controls usually are fairly well fixed by the existing facility and any adjacent roadside development.

However, for new construction or major reconstruction in rural areas there may be greater opportunity to adjust grades, particularly in relatively low areas with level terrain. Structural problems often occur where the pavement structure or upper portions of the embankment become saturated with water. Extremely high water may encroach on the roadway surface or shoulders.

Generally, the finished grade elevation should be set above the surrounding terrain to minimize the chance of ponding water or a natural high water table from saturating the pavement box. Evidence of occasional standing water in adjacent ditches or fields is a good indication that the gradeline should be high enough to assure drainage of the pavement structure at high-water levels.

Adjusting the profile grade up or down in rolling terrain is one way of balancing the earthwork so excavation within the roadway prism will be adequate to construct the designed embankments. In some instances, this is a practical approach as long as the previously mentioned criteria for gradeline elevations are not compromised. It is generally desirable to have a substantial portion of the roadway designed as an embankment, with fewer excavation areas and longer balance points. Consideration should be given to deeper cuts, wider side ditches, and daylight in cut areas where feasible. It may be more economical to use borrow material than to attempt to balance a project within the roadway prism.

Figure 5-4
Types of Vertical Curves

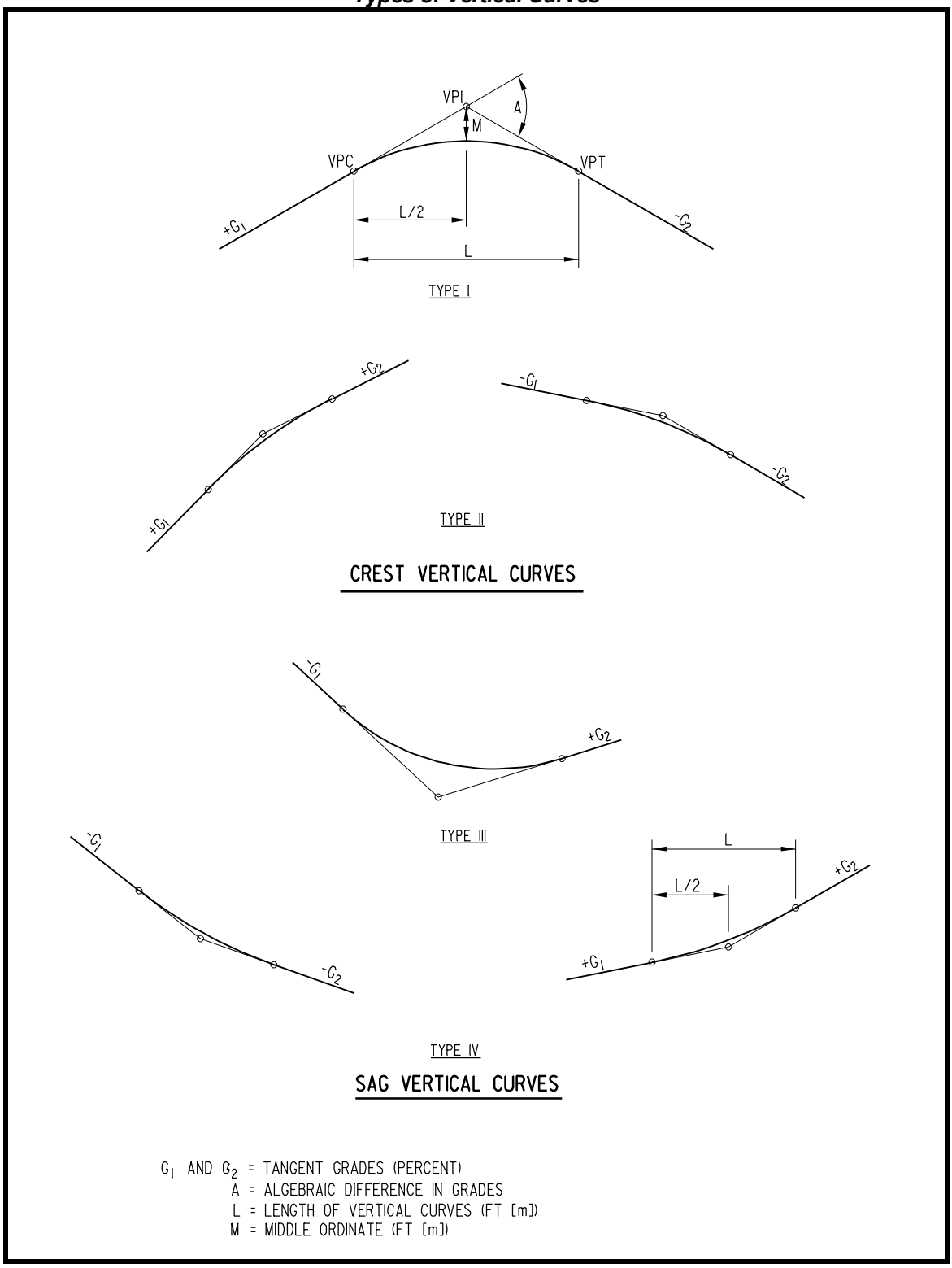


Figure 5-5
Criteria for Crest Vertical Curve Design
U S Customary [Metric]

Design Speed mph [km/h]		30 [50]	35	40 [60]	45 [70]	50 [80]	55 [90]	60 [100]	65	70 [110]
Based on Minimum Stopping Sight Distance	Distance in ft [m]	200 [65]	250	305 [85]	360 [105]	425 [130]	495 [160]	570 [185]	645	730 [220]
	K Value	19 [7]	29	44 [11]	61 [17]	84 [26]	114 [39]	151 [52]	193	247 [74]
Based on Minimum Passing Sight Distance	Length of curve ft [m]	1090 [345]	1280	1470 [410]	1625 [485]	1835 [540]	1985 [615]	2135 [670]	2285	2480 [730]
	K Value	424 [138]	585	772 [195]	943 [272]	1203 [338]	1407 [438]	1628 [520]	1865	2197 [617]

Note: Length of minimum crest vertical curve in feet [m] required to meet criteria is computed by multiplying the algebraic difference in grades by the value of the coefficient "K." Longer curves are desirable. Normally the length should be rounded to the next multiple of 100 feet [30 m]. When K>167 [51], drainage should be more carefully designed.

Figure 5-6
Criteria for Sag Vertical Curve Design
U S Customary [Metric]

Design Speed mph [km/h]		30 [50]	35	40 [60]	45 [70]	50 [80]	55 [90]	60 [100]	65	70 [110]
Based on Minimum Stopping Sight Distance	Distance in ft [m]	200 [65]	250	305 [85]	360 [105]	425 [130]	495 [160]	570 [185]	645	730 [220]
	K Value	37 [13]	49	64 [18]	79 [23]	96 [30]	115 [38]	136 [45]	157	181 [55]

Note: Length of minimum sag vertical curve in feet [m] required to meet criteria is computed by multiplying the algebraic difference in grades by the value of the coefficient "K." Longer curves are desirable. Normally the length should be rounded to the next multiple of 100 feet [30 m]. When K>167 [51], drainage should be more carefully designed.

5.2.11 URBAN GRADE DESIGN

For most projects in urbanized areas there is little opportunity for any major variation in gradeline elevations. The elevation controls usually are fairly well fixed by the existing facility and/or adjacent roadside development.

The design of vertical alignment on urban projects frequently involves consideration of special problems such as existing street intersections and adjacent property development.

Vertical curves are not required when the algebraic difference in grades is less than 0.5 percent. Because long vertical curves tend to create drainage "flat spots," the vertical curves

on street sections usually are considerably shorter than for comparable locations on rural roads.

Where controlling factors are not severe, the normal practice of carrying the profile grade on the centerline or on the median edges of pavement will work satisfactorily. Where outside controls are significant, it may be necessary to supplement the main profile with other elevation controls, such as gutter-line profiles or top-of-curb profiles. Where this is necessary, the supplemental controls should be clearly shown on the typical sections, profiles, and grades and geometrics sheets.

Special attention must be given to existing features when designing urban grades. This is particularly true in the case of private driveways when a street is being widened. With even moderate driveway grades, up or down, angular breaks must be kept flat enough with adequate clearance so that the undercarriage or bumpers of vehicles will not drag. Reference should be made to the Department's publication *DelDOT Entrance Manual*.

Where roadside development is extensive and the general elevation is higher on one side than on the other, an unsymmetrical section may be required. The crown point (and profile grade) may be offset from the centerline so the total drop from the crown line to the gutter line will be more than normal on one side and less than normal on the other. However the location of the crown point must be at the edge of the travel lane.

5.3 SUPERELEVATION

The transitional rate of applying superelevation into and out of curves is influenced by design speed, degree of curvature and number of lanes. Introducing superelevation permits a vehicle to travel through a curve more safely and at a higher speed than would be possible with a normal crown section. For a given degree of curvature, a steeper superelevation is required for a higher design speed than is needed for a lower design speed. For a given design speed more superelevation is needed through sharp curves than for relatively flat curves.

The maximum rates of superelevation used on roadways are controlled by four factors:

(1) Climate conditions (i.e. frequency of ice and snow);

(2) Terrain conditions (i.e. flat or rolling);

(3) Type of area (i.e. rural or urban); and

(4) Frequency of slow-moving vehicles.

Basic design controls for superelevation are presented in Chapter Three. Rural roadways are usually designed with a maximum superelevation rate of 6 percent but it may be appropriate to use a rate of 8 percent. Urban roadways are normally to be designed with a superelevation rate of 4 percent. Superelevation may be omitted on low-speed urban streets subjected to severe constraints. The selected superelevation rate establishes the minimum permissible radius of curve based on a project's design speed.

This section discusses practical application of superelevation criteria, with particular attention to:

- The rates of superelevation to be used for various combinations of design speed and curve radius,
- The manner of transition of slope between normal tangent sections and superelevated sections on curves, and
- Special criteria for superelevation of shoulders and auxiliary lanes.

5.3.1 RATES OF SUPERELEVATION

The Green Book, pages 165 to 174, sets forth the basic design criteria based on design speeds for the normal design superelevation rates of e_{max} = 4 and 6 percent as well as other values ranging up to 12 percent. The criteria shown includes the minimum radius of curvature, crown treatment and superelevation runoff lengths (L), all of which are related to the number of lanes to be rotated. The minimum rate of cross slope for a traveled lane is determined by drainage requirements.

The preferred cross slope (normal crown line) for high type pavement surfaces is 2 percent. The data in the Green Book exhibits are based on this cross slope. Very flat curves will not require superelevation. In the table, these are identified with the symbol "NC" meaning that the normal crown slopes used on tangent sections can be carried through the curve.

For slightly sharper curves, some superelevation is required and the normal practice is to remove the adverse crown on the outside lanes and carry the normal crown slope across all lanes. In the table, this is identified with the symbol "RC" meaning removal of the normal crown.

The indicated superelevation rates are applicable regardless of the number of lanes. The runoff lengths are for two-lane and four-lane highways. For three-lane undivided pavements, the runoff length should be 1.25 times the length shown for two-lane pavements.

On new construction and reconstruction projects, every effort shall be made to provide the prescribed superelevation rate of curves. Where this is not practicable, advisory speed signs shall be provided indicating the maximum safe speed for the curve at the existing superelevation.

5.3.2 SUPERELEVATION TRANSITION

Two terms related to superelevation transition are defined below.

- ***Superelevation runoff*** is the length of roadway needed to transition the outside-lane cross slope from zero (flat) to full superelevation, or vice versa.
- ***Tangent runout*** is the length of roadway needed to transition the outside-lane cross slope from the normal cross rate to zero (flat), or vice versa.

These two elements are applicable to superelevation on both simple circular curves and spiral transition curves, but the manner of application is somewhat different for each. General criteria for application of runoff and terminology for both types of curves are shown in Figure 5-8.

In order to achieve driver comfort and safety in operating a vehicle into, through and out of a curve, superelevation should be introduced and removed uniformly over a length adequate for the anticipated travel speeds. Figure 5-7 shows AASHTO's recommended desirable proportion of runoff on the tangent to minimize lateral acceleration and a vehicle's lateral motion. AASHTO does allow agencies to adopt a single value for all design speeds and rotated widths. For simplicity, DelDOT has adopted a runoff proportion of two-thirds in the tangent section and one-third into the curve. However, where conditions permit, the desirable values shown in Figure 5-7 should be used.

In the case of simple curves, the superelevation runoff distance (from Figures 5-9 and 5-10) is applied with one-third on the curve itself and two-thirds (or preferably as per Figure 5-7) on the tangent adjacent to the curve. This is a compromise between placing all the transition on the tangent section (where superelevation is not needed) and placing the transition on the curve (where full superelevation is needed throughout the entire length). Thus, full superelevation is not reached until slightly past the P.C. and starts to reduce shortly before reaching the P.T.

Where spiral transition curves are used, the superelevation runoff is always coincident with the spiral length (T.S. to S.C. or C.S to S.T.) and the designated full superelevation is provided between the S.C. and the C.S. The geometrics for spiral curves provide for a natural introduction of superelevation without the compromise necessary for circular curves.

In the case of both simple curves and spiral curves, the tangent runout is placed outside the superelevation runoff sections. The tangent runout length is not a critical factor. The only criterion is that the longitudinal slope of the outside edge of the traffic lanes (compared to the profile grade) should not exceed a rate of about 1:200 during the tangent runout.

Where more than two lanes are to be rotated, the runout distance should be lengthened so the 1:200 longitudinal slope limitation is not exceeded.

Figure 5-7
Runoff Locations that Minimize Vehicle Lateral Motion

Design Speed (mph) [km/h]	Portion of runoff located prior to the curve in percent		
	No. of lanes rotated		
	1.0	1.5	2.0-2.5
15-45 [20-70]	80	85	90
50-80 [80-130]	70	75	80

5.3.3 AXIS OF ROTATION

In the design of superelevation, it is necessary to select a point (axis of rotation) on the cross section around which the cross slope will be rotated to gradually change to the specified superelevation slope. The location of this point varies with the basic characteristics of the typical section and the characteristics of the project area such as drainage, adjacent land elevations, and roadside development.

The available options for rotating traveled ways to attain superelevation are graphically shown in Figures 5-11 to 5-13. These are:

1. Revolving a traveled way with normal cross slopes about the centerline profile.
2. Revolving a traveled way with normal cross slope about the inside-edge profile.
3. Revolving a traveled way with normal cross slopes about the outside-edge profile.
4. Revolving a straight cross-slope traveled way about the outside-edge profile.

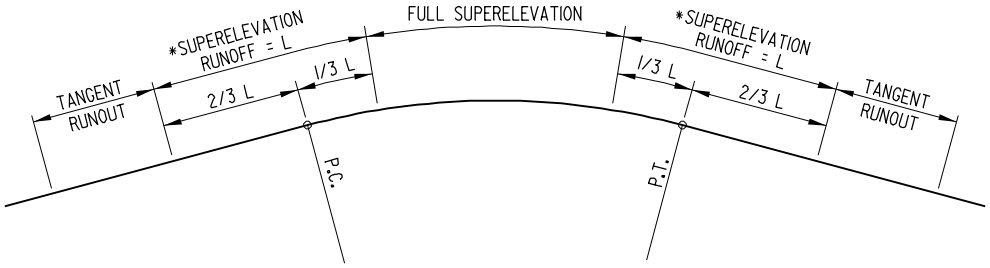
The most commonly used method for transitioning into a superelevated section is rotating the traveled way about the centerline as shown in Figure 5-11. This method provides a more uniform appearance with a less distorted edge of traveled way.

Figure 5-12 shows the transition method most commonly used for two-lane one-way roadways where the axis of rotation coincides with the edge of the traveled way adjacent to the highway median.

There are many possible profile arrangements. Selecting the most appropriate is based on drainage requirements, avoidance of critical grades, aesthetics and making adjustments to fit the roadway section into the adjacent topography.

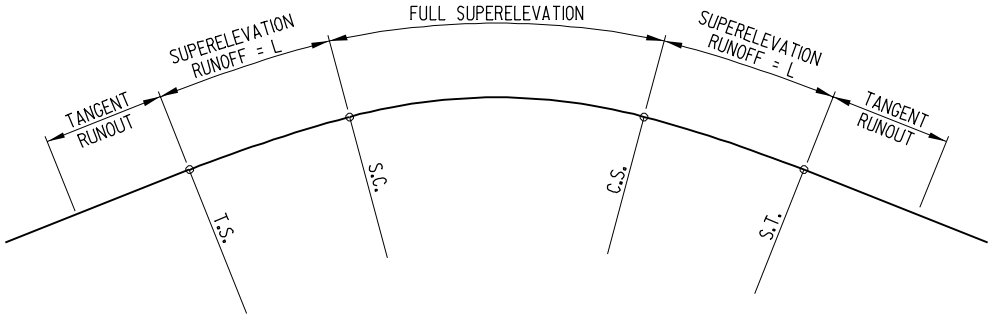
The methods of attaining superelevation shown result in angular breaks in the profiles of the pavement edges. For appearance and safety, these breaks should be rounded in final design by using vertical curves. The simplest method is by graphically plotting the center line or edge profile on a vertical scale. Then by means of a spline (a flexible straight edge), ship curve, curve template, or circular curve, draw smooth-flowing lines to approximate the straight-line controls. Once the edge profiles are drawn in the proper relationship, elevations can be read as needed from the plot. Vertical curves can also be used. The minimum length of vertical curve in feet [meters] can be used as numerically equal to the design speed in mph [km/h]. The angular break for transitioning about the centerline is less pronounced than for other transitions and the vertical curve may be adjusted accordingly.

Figure 5-8
Superelevation Runoff Elements



SIMPLE CURVE

* - SEE TEXT AND SUPERELEVATION ATTAINMENT FIGURES



SPIRAL TRANSITION CURVE

Figure 5-9*
Minimum Superelevation Runoff and Tangent Runout Lengths (US Customary)

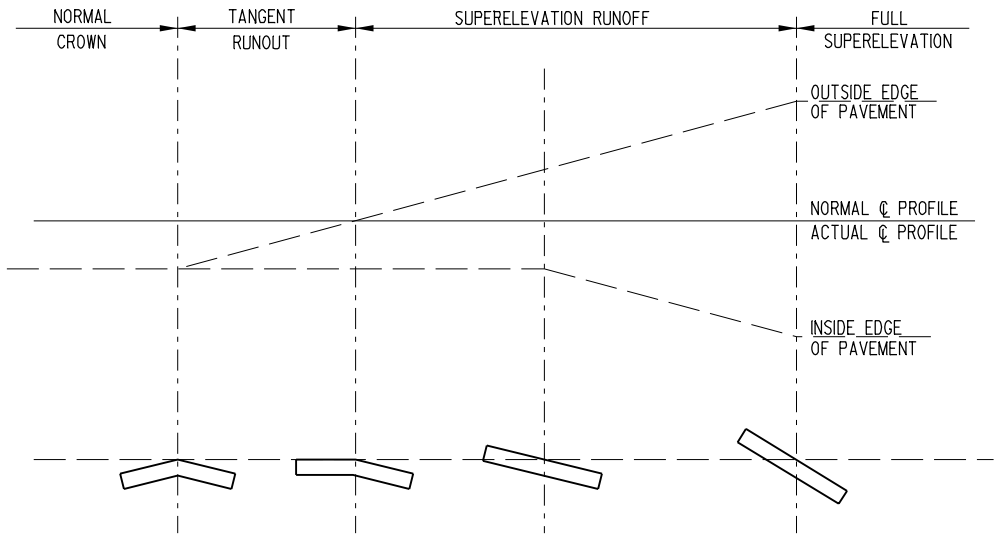
Minimum runoff and runout length (ft)				
Design speed (mph)	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
25	34	69	103	34
30	36	73	109	36
35	39	77	116	39
40	41	83	124	41
45	44	89	133	44
50	48	96	144	48
55	51	102	153	51
60	53	107	160	53
65	56	112	167	56
70	60	120	180	60
Two lanes rotated				
25	51	103	154	51
30	55	109	164	55
35	58	116	174	58
40	62	124	186	62
45	67	133	200	67
50	72	144	216	72
55	77	153	230	77
60	80	160	240	80
65	84	167	251	84
70	90	180	270	90

Figure 5-10*
Minimum Superelevation Runoff and Tangent Runout Lengths [Metric]

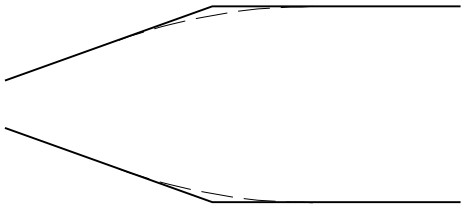
Minimum runoff and runout length [m]				
Design speed [km/h]	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
20	9	18	27	9
30	10	19	29	10
40	10	21	31	10
50	11	22	33	11
60	12	24	36	12
70	13	26	39	13
80	14	29	43	14
90	15	31	46	15
100	16	33	49	16
110	18	35	53	18
Two lanes rotated				
20	14	27	41	14
30	14	29	43	14
40	15	31	46	15
50	17	33	50	17
60	18	36	54	18
70	20	39	59	20
80	22	43	65	22
90	23	46	69	23
100	25	49	74	25
110	26	53	79	26

*Note: Figures 5-9 and 5-10 are based on 12-ft [3.6 m] lanes and 2.0% normal cross slope

Figure 5-11
Superelevation Attainment
Traveled Way Rotated about Centerline



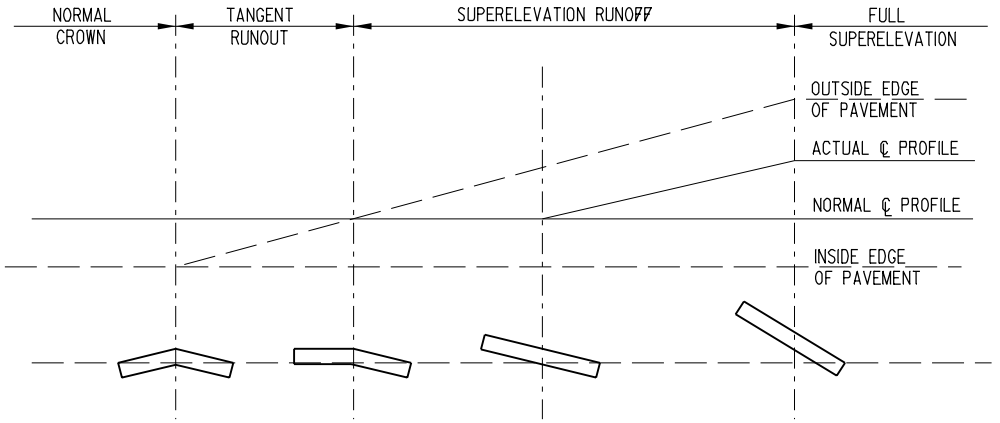
TRANSITION A - PAVEMENT REVOLVED ABOUT CENTERLINE



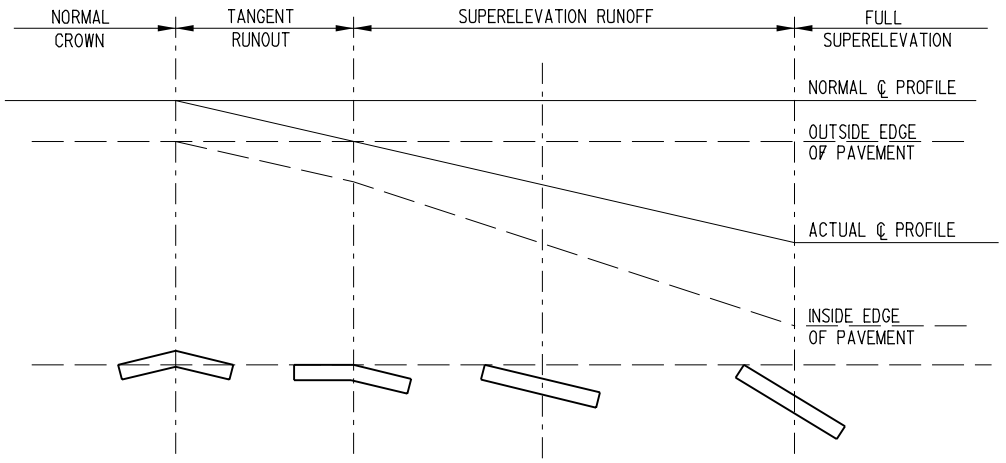
NOTE : ANGULAR BREAKS TO BE APPROPRIATELY
ROUNDED AS SHOWN BY DASHED LINE.

ROUNDING

Figure 5-12
Superelevation Attainment
Traveled Way Rotated about Inside and Outside Edge



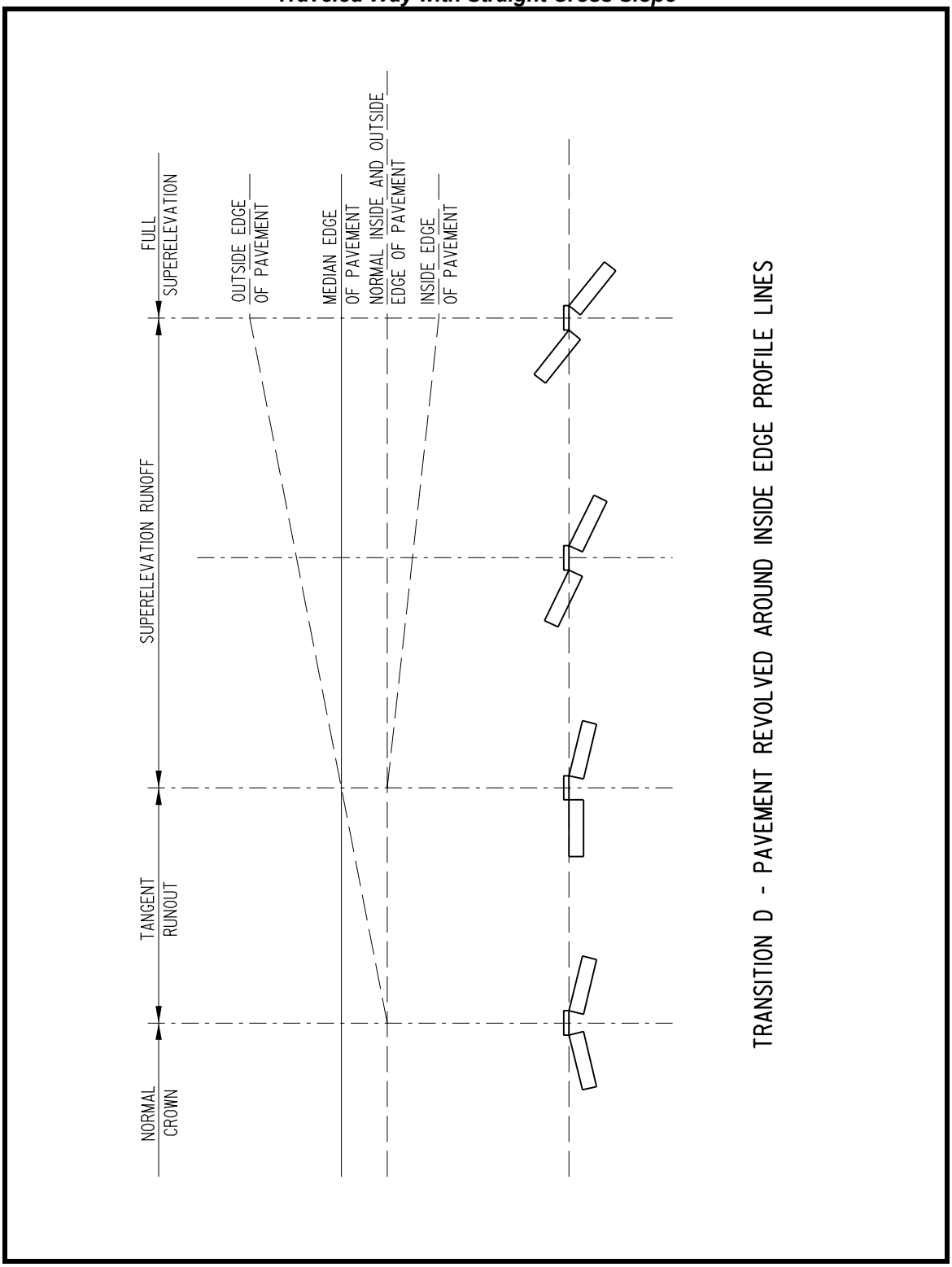
TRANSITION B - PAVEMENT REVOLVED ABOUT INSIDE EDGE



TRANSITION C - PAVEMENT REVOLVED ABOUT OUTSIDE EDGE

NOTE : USE TRANSITION A FOR ALL UNDIVIDED HIGHWAYS.
FOR DIVIDED HIGHWAYS WITH MEDIAN, USE TRANSITION A FOR TRAFFIC LANES ON OUTSIDE OF CURVE AND TRANSITION B FOR TRAFFIC LANE ON INSIDE OF CURVE.

Figure 5-13
Superelevation Attainment
Traveled Way with Straight Cross Slope



Chapter Six

Drainage and Stormwater Management

6.1 INTRODUCTION

Adequate drainage is essential in the design of highways since it affects the highway's serviceability and usable life, including the pavement's structural strength. If ponding on the traveled way occurs, hydroplaning becomes an important safety concern. Drainage design involves providing facilities that collect, transport and remove stormwater from the highway. The design must also consider the stormwater reaching the roadway embankment through natural stream flow or manmade ditches.

This chapter deals with drainage policies, procedures and guidance to be followed in achieving cost-effective design and construction within DelDOT's Highway System. The information contained herein is compiled from various federal and national publications, textbooks, and drainage manuals. The information provided is of a general nature with the inclusion of methods, criteria and references specifically applicable to DelDOT projects.

Source documents are listed in the introduction to each section. It is presumed that the designer is familiar with the basic theory and methods of analysis and design in both hydrology and hydraulics. The information provided herein will have to be supplemented with hard copies or on-line access to the referenced documents.

The regulatory environment related to drainage design is ever changing and continues to grow in complexity. Engineers responsible for the planning and design of drainage facilities must be familiar with Federal, state, county and local regulations, laws, and ordinances that may impact the design of storm drain systems.

Many federal laws have implications that affect drainage design. These include laws concerning:

- Flood insurance and construction in flood hazard areas,
- Navigation and construction in navigable waters,
- Water pollution control,
- Environmental protection,
- Protection of fish and wildlife, and
- Coastal zone management.

Federal agencies formulate and promulgate rules and regulations to implement these laws. Highway hydraulic engineers should keep informed regarding proposed and final regulations.

Some of the more significant federal laws affecting highway drainage are:

- **The Department of Transportation Act** established the Department of Transportation and sets forth its powers, duties, and responsibilities to establish, coordinate, and maintain an effective administration of the transportation programs of the Federal Government.
- **Federal-Aid Highway Acts** provide for the administration of the Federal-Aid Highway Program. Proposed Federal-aid projects must meet existing and probable future traffic needs and conditions in a manner conducive to safety, durability, and economy of maintenance, and must be

designed and constructed according to standards best suited to accomplish this and to conform to the needs of each locality.

- **Federal-Aid Highway Act of 1970** provides for the establishment of general guidelines to ensure that possible adverse economic, social and environmental effects relating to any proposed Federal-aid project have been fully considered in developing the project. In compliance with the Act, the Federal Highway Administration (FHWA) issued process guidelines for the development of environmental action plans as contained in the Federal-Aid Highway Program Manual (FHPM) and in 23 CFR 795 et seq.
- **Federal-Aid Highway Act of 1966 amended by the Act of 1970** requires the issuance of guidelines for minimizing possible soil erosion from highway construction. In compliance with these requirements, the FHWA issued guidelines that are applicable to all Federal-aid highway projects. These guidelines are included in the FHPM. Regulatory material is found in 23 CFR 650.201.

All Federal-aid projects shall conform to FHWA guidelines. FHWA policy includes the following:

- **FHWA Policy 23CFR635 Subpart D—General Material Requirements**
- **FHWA Policy 23CFR650 Subpart A—Location and Hydraulic Design of Encroachments on Floodplains** establishes policy affecting any project that includes an encroachment on a base floodplain.
- **FHWA Policy 23CFR650 Subpart C—National Bridge Inspection Standards** defines the national standards for the proper safety inspection and evaluation of all highway bridges including the evaluation of bridges for scour susceptibility in accordance with the guidance in FHWA Technical Advisory T5140.23.
- **FHWA Policy 23CFR650 Subpart H—Navigational Clearances for Bridges** requires coordination with the United States Coast Guard (USCG) and United States Army Corps of Engineers (USACE) in providing adequate vertical and horizontal clearance for navigation on navigable waterways.

Other federal laws affecting hydraulic tasks, analyses, design, or construction include those formulated under the following legislative acts:

- The Coastal Zone Management Act of 1972
- The Federal Water Pollution Control Act (Clean Water Act)
- The Fish and Wildlife Act of 1956
- The Fish and Wildlife Coordination Act
- The Flood Disaster Protection Act of 1973
- The National Environmental Policy Act (NEPA) of 1969
- The Rivers and Harbors Act (1899)
- The Safe Drinking Water Act of 1974
- Water Quality Act of 1987
- Wild and Scenic Rivers Act

6.2 DESIGN RESPONSIBILITIES

Responsibilities for drainage design are divided between the Bridge Design and Project Development sections based primarily on the size of the drainage basin. Bridge Design is responsible for all watersheds over 300 ac and locations where the design discharge capacity opening(s) exceeds 20 ft^2. Project Development is responsible for:

- All locations requiring pipe culverts not exceeding 20 ft^2 of waterway opening;
- Closed drainage systems involving storm drains;
- Roadside drainage including swales and ditches;
- Erosion and sediment control for transportation projects;

- Stormwater quantity and quality management.

The section responsible for design of the drainage structure shall also design the related stream bank stabilization and erosion control.

The scope of this chapter is limited to the drainage design responsibilities of the Project Development Section. Refer to Chapter Three of the Bridge Design Manual for information on drainage design for structures. Structural designs are coordinated through the Bridge Design Section.

6.3 DESIGN CRITERIA

The Department has established basic design criteria for design storm frequency depending upon roadway type, and the design of open channel flow, pipe flow, pavement drainage, and storm drains (closed drainage systems). These criteria are summarized in Figures 6-1 through 6-3.

6.4 DESIGN PROCEDURES

The goal of drainage design is to economically design systems that limit the exposure of adjoining upstream and downstream properties, the newly constructed roadway and the traveling public to an acceptable flood risk during high flows. Attachment B has several example problems that illustrate the design procedure for the most commonly used drainage facilities used on a roadway project. AASHTO's *Model Drainage Manua*l is one of the best resources for preparing a project's drainage design. In addition, DelDOT has an Internet Web Site (http://www.deldot.gov/) that has additional drainage references.

Below is a brief description of drainage design steps normally followed when designing roadway facilities. Each of these is expanded in sections referenced in the steps.

Step 1. Data collection (Section 6.5) includes:

a. Establish agency requirements and define design criteria and parameters.

b. Access the applicable online drainage design resources (hard copies, if necessary), other design references, environmental controls, drainage criteria, etc.

c. Prepare a drainage area map using USGS maps, topographic maps and aerial photography.

d. Determine soil characteristics using the county soil survey map from the US Natural Resource and Conservation Service (NRCS).

e. Obtain As-Built or record plans from previous projects in the area.

f. Contact the area maintenance office for existing flooding problems.

g. Request soil borings to establish the typical water table elevation. Saturated soil conditions can adversely impact the drainage system design; require expensive dewatering to build; and require permits. Therefore this condition should be identified early in the process.

h. Obtain existing and future land use information.

i. Visit the anticipated drainage area site to verify collected data, locate existing structures and visually observe any problems with their past performance, signs of past high water levels, stream erosion problems, currently developed areas, existing stormwater management facilities, etc.

j. Request a video inspection of any drainage facility to remain in service.

k. Obtain existing and proposed underground utility locations.

Figure 6-1
Design Criteria – Frequency
(Return Period in Years)

Functional Classification	Type of Drainage Installation [1]			
	Pipe Culverts	Storm Drains	Roadside Ditches	Median Drains
Interstate, Freeways and Expressways	50	10 [2]	50	50
Arterials	50	10 [2]	25	25 [2]
Collectors	50 [3]	10 [2]	25 [4]	10 [2]
Local Roads and Streets including Subdivision Streets	25	10 [5]	10	10 [5]

[1] For Stormwater Management see Section 6.10.

[2] Use a 50-yr frequency at sag points, i.e. underpasses or depressed roadways, where ponded water can be removed only through the storm drain system.

[3] Use a 25-yr frequency for rural collectors.

[4] Use a 10-yr frequency for rural collectors.

[5] Use a 25-yr frequency at underpasses or depressed roadways where ponded water can be removed only through the storm drain system.

Figure 6-2
Design Criteria – Allowable Spread on the Pavement Cross Section

Functional Classification	*Allowable Water Spread*
Interstate, Freeways, Expressways	Full shoulder width
Arterials with full shoulder or parking lane Arterials with less than full shoulder or parking lane	Full shoulder or parking lane One half of adjacent driving lane
Collectors Design speed < 45 mph Design speed ≥ 45 mph	 One half of driving lane Full shoulder width
Local Roads and Streets including Subdivision Streets	One half of driving lane
Sag Points (all facilities)	Full shoulder width

Figure 6-3
Design Criteria – Miscellaneous

Ditches	
Ditch flow line below edge of shoulder Ditch water surface elevation below edge of shoulder Minimum ditch grade	≥2.5 ft preferred 0.5 ft minimum; 1 foot preferred 0.003 ft/ft (preferred 0.005)
Pipes	
Minimum size - cross road pipe / culvert Minimum size - storm drain	18 in 15 in
Minimum full flow velocity	3.0 ft/s
Maximum outlet velocity (determined by design parameters)	Based on scour, erosion, risk potential of discharge channel, and mitigating measures such as energy dissipators
Maximum continuous distance between storm drain structures (without clean-out access)	300 ft
Reinforced Concrete Pipe Bedding	Class C, unless specified otherwise
Personnel Grate for Pipe Inlet	All pipes 12 in and larger with an open inlet that is not a straight run to the outlet without full daylight visible when looking through the pipe to the other end
Minimum Pipe Cover	3 ft preferred at profile grade line, 1 ft at the shoulder, depending upon the structural requirements of the pipe material, bedding type and fill height at the installation location; see supplemental figures on DelDOT's web site and Table 12.6.6.3-1 of the *AASHTO LRFD Bridge Design Specifications*
General Criteria - Storm Drain System	
Hydraulic Grade Line (HGL)	≥ 1 ft below top elevation of all manhole covers or top of any inlet
Required manhole or inlet locations	Intersection of two or more storm drains Pipe size change Alignment change Grade changes and sag points in curbed sections
Outfall pipe invert elevation	≥ 0.2 ft below lowest incoming pipe elevation (preferred)
Discharge pipe	Invert higher than the outfall elevation
Inlet clogging factor of safety	1.5 with curb and 2.0 without curb 1.0 for curb opening inlet

Step 2. Hydrology (Section 6.6)

The principles of a hydrologic study for a watershed are common to most design methods. The watershed is defined and divided into subareas contributing to proposed stormwater outlets. Water flows through the watershed as sheet flow, shallow concentrated flow, swales, open channels, pavement drainage, roof drains, and storm drains The existing and future surface runoff characteristics of these subareas are determined and a coefficient or runoff curve number is assigned. The flow parameters are than determined including the hydraulic length of the reaches, the types of flow, and average slope. Using this data flow velocities are determined and the flow times of each type of flow is determined and combined into a time of concentration (t_c) for the watershed. A design rainfall frequency and intensity is selected. Using this data the peak discharge is determined.

Available methods of hydrological analysis include the Rational Method; TR-20 Project Formulation Hydrology Program System; TR-55 Urban Hydrology for Small Watersheds and its Windows version WINTR-55; and the U.S. Geological Survey (USGS) Method which has developed regression equations for Delaware. Each of these procedures has its own methodology, assumptions, limitations, and applications.

The Rational Method is most frequently used for estimating small, homogenous or highly impervious drainage areas which are smaller than 50 acres, including gutter flow, drainage inlets, stormwater systems, small ditches, swales and culverts. This method is not recommended for storage design, i.e., stormwater management.

TR-20 is a computer hydrological model simulating runoff hydrographs using natural and synthetic rainfall events over a large complex watershed up to 2000 acres with multiple subareas, channel reaches, and varied confluence points that are routed through downstream reaches and structures.

Data generated by TR-20 has been used to develop a more user-friendly method for analysis of small watersheds (1 to 300 acres) referred to as TR-55 and WINTR-55. Except for pavement drainage, these are commonly used when analyzing stormwater flow for watersheds and their subareas under both pre- and post-development conditions. TR-55 can be applied to designing most of a project's drainage design features, including stormwater management facilities.

USGS has developed regression equations for Delaware based upon years of data from stream gauging stations. This method is most applicable for estimating the magnitude and frequency of flood-peak discharges and flood hydrographs for large streams, channels and culverts. These equations relate peak discharge to independent variables describing the physical and climatic basin characteristics. Refer to the Bridge Design Manual for more details.

Step 3. Hydraulics

Using the hydrologic data and other acquired information, lay out an initial drainage system to accommodate this flow. Drainage system may include various types and shapes of open channels (i.e., ditches, swales, and gutters), storm drains, median drains, and cross road pipe culverts. The system profile should begin at the farthest discharge point downstream, i.e., a natural or artificially created channel, or an existing drainage system. To minimize the expense of pipe excavation, pipe slopes should conform to the surface slope wherever possible.

Roadside ditches and culverts pose a roadside hazard and should be designed considering safety as a design parameter. Use the principles found in AASHTO's *Roadside Design Guide* to minimize their potential as roadside hazards.

a. Open Channel Design (Section 6.7): There are several types of open channels used in drainage design including roadside ditches, toe-of-slope ditches, intercepting ditches, chutes (stone-lined or pipes) used on step

embankments, and gutters used in curbed sections.

i. For design of roadside channels, hydraulic conditions usually are assumed to be steady and uniform. Flow under these conditions has an energy line approximately equal to the average ditch grade, and a flow depth that changes gradually over time. This allows for determining normal flow values where velocity, depth of flow, surface roughness and slope is constant over the design section.. The capacity or discharge of this open channel is determined by using the modified Manning's equation (which combines Manning's and the continuity equations). Resources are referenced in Section 6.7 that detail open channel design procedures.

ii. For non-uniform flow where either the depth of flow, velocity, or cross section changes, the principles and equations for unsteady, non-uniform flow are used, the computer program HEC-RAS is recommended.

iii. Filter strips, biofiltration swales, bioretention areas and infiltration ditches are designed using DURMM, DNREC's green technology computer model based on a combination of computer routines from TR-22 and TR-55.

b. Pavement (Storm) Drainage System Design (Section 6.8): There are several reasons to provide adequate pavement drainage design, including maintaining the service level during stormy conditions; protecting the user from ponding water that could cause loss of vehicle control; protecting adjacent natural resources, development and property; and maintaining the structural integrity of the pavement section. A typical pavement/storm drainage system consists of curb, gutter, inlets, drainage structures (i.e., junction boxes and manholes), storm drains (pipes), and an outfall structure into an acceptable discharge point.

The design of a project's drainage system may require installing new pipe culverts, or upgrading existing pipe culverts. In this instance see Step 4.

i. The **hydrologic analysis** should be consistent with Step 2. The rational method is used to calculate the peak discharge to be carried by the gutter and storm drains.

ii**. Gutters** are designed using the principles for open channel flow by establishing the rate of flow or discharge, the depth of flow, the velocity, the surface roughness and the slope using a modified form of Manning's equation.

iii**. Inlets** can be grate inlets, curb-opening inlets, (a combination inlet involving a curb opening combined with a grate inlet), or occasionally slotted inlets. Refer to DelDOT's *Standard Construction Details* for the detailed description of their shape and dimensions. See Figure 6-2 and 6-3 for the recommended allowable spread on the pavement and clogging capacity (efficiency). Inlet design consists of determining the interception capacity of the type of inlet under consideration and its location on a continuous slope or in a sag point.

The locations of inlets are a function of the roadway geometrics and cross section, in addition to hydraulic considerations. The underground connecting pipes have to be accessible for maintenance. The physical constraints from using available inspection and cleanout equipment are a limiting factor in the maximum distance between inlets. The volume of inlet bypass flow combined with the normal flow calculated to reach each downstream inlet affect the inlet spacing. Other factors include features such as curb ramps, crosswalks, driveways, intersecting streets, points of superelevation transition, and roadway profile low points or sag vertical curves. Refer to Section 6.8 and Example 5, Attachment B, for a description of manually determining inlet spacing. It is important to note that inlet design in sag locations (both for vertical curves and superelevated sections) is critical due to ponding that might encroach on the roadway.

iv. **Storm drains (pipes)** are used in areas of limited right-of-way and in curbed sections, normally in combination with inlets, junction boxes, manholes and outfall structures. Sections of smaller pipe are referred to as segments or runs. The shorter runs usually discharge into a larger main storm drain called a trunk line discharging into an outfall. Locating storm drains, particularly manholes or other drainage appurtenances, outside of the roadway (especially the traveled way) should be considered for safer maintenance operations and minimizing traffic disruption. In addition, if there is pavement settlement due to compaction problems or erosion of the subgrade from joint separation, the repairs can be made with a reduced effect on the flow of traffic.

Pipe criteria includes the minimum allowable pipe size, the material type based on the allowable service life for the project and location being designed, and any special installation or structural requirements specified by the *Standard Specifications* or manufacturer. Refer to DelDOT *Design Guidance Memorandum Number 1-20R*.

Placement with respect to underground and above ground utilities is to be considered throughout the system design. Utility relocations and/or their protection are expensive. The cost may be fully paid by the project or shared with the utility provider and their customers.

Storm drain capacity design is based on the principles of open channel flow, i.e., steady, uniform flow with a constant average velocity within each pipe run. This is accomplished by assuring the HGL is below the top of pipe. To complete this analysis the designer needs to have a working plan showing the layout and profile of the storm drainage system with the location of storm drain runs and trunk lines, direction of flow, location and numbering of drainage structures and manholes, and location of utilities. Starting with the upstream storm drain run, the designer performs the hydrological calculations for the system. Storm drains are sized using Manning's equation, varying the slope and size needed for the design discharge. Losses in the system at the various drainage structures, bends, etc. are calculated. The final step is determining the system HGL to ensure open channel flow exists throughout the system. The design should be evaluated and adjusted to avoid pressure flow conditions. See Section 6.8 for a more detailed discussion.

Median drainage is an important consideration on divided roadways, especially with high-speed lanes. Ponding water and its spread onto the travel lane or shoulder need to be minimized and the pavement's structural integrity protected from saturation leading to failure. This is difficult since the shape and width of the median is primarily controlled by safety considerations and not by the preferred hydraulic characteristics. Therefore medians are usually wide, shallow, and depressed with a swale with relatively flat side slopes and drainage inlets to control the flow.

The principles of open channel flow are used for locating inlets to limit the depth of water to an elevation of 0.5 ft minimum, one ft preferred, below the outside edge of shoulder. Inlets in sags may function either as a weir or orifice depending upon the type of grate and depth of water, so sag locations must be given special consideration. The preferred method of ensuring adequate drainage in a sag location is to install inlets (flanker basins) upstream from the low point.

Drainage structures include manholes, junction boxes and outlet structures. Each of these has a particular function in the drainage system and a role in the efficiency, capacity and future maintenance of the system. Refer to the *Standard Construction Details* for the size, shape, etc. of these structures. The designer should check the structure dimensions versus the storm drain size to ensure they are compatible. Of primary concern is allowing for the losses in flow capacity these features introduce into design of the system. HDS-5 and HEC-22 discuss this subject.

Designing a storm drainage system for the roadway section is normally an iterative process with several data inputs; the use of

available drainage software is recommended. Available references include HEC-22 *Urban Drainage Design Manual*; HEC-12 *Drainage of Highway Pavements; The State-of–the Art, Storm Drain Systems* (Volume IX of the AASHTO *Highway Drainage Guidelines*); and the AASHTO *Model Drainage Manual*.

Step 4. Culverts (Pipe Culverts Section 6-9)

Culverts are closed conduits (pipes) placed through an embankment as a part of the drainage system to convey runoff collected by roadside channels or natural watercourses such as streams.

Frequently a roadway project will involve installation of a new cross pipe culvert and/or upgrading an existing culvert. For culverts requiring an area of 20 square feet or more, refer the design to the Bridge Design Section.

Hydraulic design considerations include establishing the design storm frequency, the allowable headwater, the allowable tailwater, the minimum allowable pipe diameter, and the type of pipe material (considering allowable service life and location with respect to location under the pavement or the embankment, and the anticipated design load). Headwater and tailwater levels depend upon the roadway typical section and profile, acceptable upstream and downstream flooding, stormwater management interests and other environmental concerns.

Refer to the resources listed in Section 6.9 for detailed design procedures for culverts. Culvert placement and end treatments affect roadside safety.

Step 5. Erosion and Sediment Control (Section 6-10)

Erosion and Sediment Control is an integral part of designing and constructing a drainage system. The erosion and sediment plan has to be prepared concurrently with the drainage plan. The requirements of this plan are in DelDOT's *ES_2M Design Guide* and the *Delaware Sediment and Stormwater Regulations*.

Step 6. Stormwater Management (Sections 6-11 and 6-12)

Managing highway runoff involves designing systems to address stormwater quantity and quality. The design of these facilities is covered in more detail in Sections 6.11 and 6.12; the *ES_2M Design Guide*; and the *Delaware Erosion & Sediment Control Handbook;* and DNREC's *Green Technology*: *The Delaware Urban Runoff Management Approach DURMM: The Delaware Urban Runoff Management Manual.*

The goal of runoff quantity management is to minimize excessive downstream flooding and property damage. The basic principle is that any proposed land development (including roadways) should not increase the downstream flow above what existed prior to the changes. Two methods used to control this flow are:

1. Retention facilities are used to control stormwater runoff and dissipate it through evaporation and infiltration. They are commonly used when there is no nearby natural watercourse or body of water for the release of the stored water.

2. Detention facilities are used to reduce the peak discharge and slowly release this runoff. Normally they are designed to release all of the stored water after the storm has passed. Frequently detention and retention facilities are combined to: manage pollution and sediment; control stormwater quantity discharge; and as an aquifer recharge basin.

A second goal of stormwater management is to maintain water quality. Runoff from developed areas can degrade water quality. Designing for water quality is referred to as Non-Point Source Pollution Control. Legislation referred to as the National Pollutant Discharge Elimination System (NPDES) requires that the pollutants in runoff from storm systems be reduced. Pollutions are reduced using Green Technology BMP's including biofiltration grass swales, biorentention areas, infiltration ditches and area filter strips, forebays at pipe outfalls

and/or retention ponds with slow release outlets.

Step 7. Drainage Report:

The final product of the drainage design is the H&H (Hydrology and Hydraulic) Report (Stormwater Management Report). This report should clearly compare the analysis results to the approved design criteria including whether or not the proposed design meets the appropriate criteria.

This report should include the following:

- **Executive Summary** with an overview of the project intent, drainage methodology and stormwater management approach.
- **Project Description** with detailed information of project scope and proposed construction.
- **Hydrological and Hydraulic Criteria** including methodology used for the drainage design of open channels, pavement drainage and culverts.
- **Stormwater Management Approach** with geographical description of project and design criteria for stormwater quality and quantity management.
- **Watershed Descriptions** discussing the various flow characteristics of drainage areas noting existing drainage facilities and assessment of offsite drainage (beyond survey data collection). Each area is given an identification code as shown on the drainage plans.
- **Conclusions** describing the proposed drainage for each area with a summary comparing the pre- and post-project conditions.
- **Special Conditions** requiring any special design measures to meet permit requirements, water quality standards, etc.
- **Backup Data** plans, maps, quadrangles, assumptions, hand calculations, computer output, etc.

The report is normally prepared in two phases. The preliminary report would include an initial hydrologic analysis, a preliminary drainage plan and, most importantly, the proposed design criteria and methodology to be used in performing the detailed drainage analysis. The final report would contain this data along with the final drainage calculations and final drainage plan. Refer to DelDOT's *ES_2M Design Guide* for the required contents of a stormwater management report. Additional guidance and samples are available from the Quality Section.

6.5 DESIGN PROCESS

The following describes the process for performing an acceptable drainage design.

6.5.1 LEGAL REQUIREMENTS AND AGENCY COORDINATION

Highway construction encroaching flood plains, subaqueous lands, wetlands, lakes, ponds, and natural streams shall conform to prevailing governmental regulations. Strict attention shall be given at the planning stage of the project to avoid potential legal problems. Design activities shall be closely coordinated with DelDOT's Environmental Studies Section and other agencies for obtaining necessary permits complying with the regulations and the project schedule.

Sediment and stormwater regulations were enacted into State Law in 1990 and 1991. The designer should be familiar with the provisions and ensure that the design conforms to the requirements.

6.5.2 DATA COLLECTION

6.5.2.1 INITIAL PHASE

The initial phase of the design is to acquire the relevant data and information. This information is available from various sources including federal, state, and county agencies. See Attachment A for resource references.

Site investigations and field surveys are absolutely necessary and should include:

- Watershed characteristics, soil surveys, U.S.G.S. map topography, and land use;
- Stream course data—profile and cross-sections within the right-of-way and in the

vicinity. Note high water marks when possible. Obtain FEMA maps (http://msc.fema.gov), if available, for 100-year flood plain location and elevation;

- Existing drainage facilities with design details, if available, and field data;
- Current aerial photographs;
- Wetlands locations from field survey data and DelDOT's Environmental Studies Section; and
- Any affected resource protection areas from available maps.

When evaluating existing facilities within the project area, the designer should:

- Video and investigate the need for replacing existing drainage facilities due to inadequate strength or functional deficiency;
- Check utilities for possible impact and constraints on the drainage design; and
- Verify the ditch and stream bank elevations.

The designer should plan on removing and replacing any existing damaged pipe. It is also important that all pipes are relatively free of silt and debris in order to properly inspect and ensure the bottom of the pipe is intact.

6.5.3 DRAINAGE PLANS

The drainage plans are prepared in two phases—preliminary and final. The preliminary drainage plans are included in the preliminary construction review package. The final drainage plans are included in the semi-final construction plan review package. Based upon the comments from the two reviews, the final H&H Report is prepared and submitted.

6.5.3.1 PRELIMINARY DRAINAGE PLAN

The preliminary drainage plans include the following information related to drainage:

- Cut areas with beginning and end stations indicating depths of cut;
- Fill areas with beginning and end stations indicating depths of fill;
- Stations and elevations of high points;
- Stations and elevations of low points;
- Limits of relatively flat sections (grades flatter than 0.5%);
- Drainage areas and subareas indicating basin divides from cross sections and topographic sheets;
- Preliminary locations of proposed drainage facilities;
- Limits of steep profiles where erosion protection is anticipated;
- Locations of existing utilities which may impact highway drainage;
- Potential drainage storage and outfall locations; and
- Sediment and stormwater management project review and design checklist items.

6.5.3.2 FINAL DRAINAGE PLAN

The preliminary drainage plan and previously gathered information are reviewed in the field in developing the final drainage plans.

In fill areas the designer should:

- Investigate the need for roadside ditches to carry surface runoff and indicate their approximate locations on the working drawings;
- Estimate the extent of possible erosion qualitatively and indicate where preventive measures may be needed.

In cut areas, the designer should:

- Indicate where benching may be necessary,
- Delineate approximate ditch locations and/or inlets and storm drains; and
- Evaluate potential erosion problems and indicate what measures should be taken to minimize them.

In flat sections, the designer should:

- Indicate where the runoff can be disposed; and

- Evaluate the impact of ditch or storm drain slopes.
- At intersecting roads and superelevation transitions the designer should:
- Establish and verify inlet locations; and
- Note changes in cross slopes, warping and crowning of roadways, and the means of effective interception of surface runoff.
- Field reviews should include gathering information required by the sediment and stormwater management checklist.

In all cases, check the boundaries of drainage areas, describe the topography, determine the ground cover, and verify soil types against the published soil survey maps from the county conservation district, to aid in determining the coefficient of runoff, runoff curve number, and time of concentration.

6.5.4 DRAINAGE REPORT

Drainage design and therefore the report is divided into several topic areas including, hydrology; roadside ditches; open channels; pavement and storm drains; cross road culverts; erosion and sedimentation control; stormwater quantity management; and stormwater quality management.

Each of these topic areas has multiple resource publications, Internet sites, and software for design as well as design aides such as charts, nomographs, tables, and figures. In addition, some topic areas have several acceptable methods for performing the design calculations. In order for a reviewer to check the report, it is necessary that all the resources or applicable portions of each be included or identified.

The analysis performed during design is documented in a comprehensive report of the hydrological characteristics of the drainage areas and hydraulic calculations relating to drainage and stormwater management. The basics of the drainage report are developed after defining federal, state and local requirements, data collection, a preliminary design and a comprehensive field review. A preliminary H&H Report is necessary for producing an acceptable preliminary plan package. Obtaining written approval from the Project Manager of this report and proposed methodology is required.

For the semi-final plan submittal, a final H&H Report is prepared which also includes the final analysis for drainage and stormwater management with verification that the approved criteria have been met or exceeded along with construction detail. The final drainage report describes the drainage characteristics within the project area, the various design criteria, data collected, data sources and references, anticipated computer software for preliminary analysis, a proposed analysis format, etc. This report may need minor revisions after its final review. Refer to DelDOT's *ES_2M Design Guide* for the required contents of a stormwater management report. Additional guidance and samples are available from the Quality Section.

6.6 HYDROLOGY

Hydrology studies the effect of rainfall intensity, duration, and runoff at prescribed frequencies on a watershed. The design of most drainage facilities requires the peak rate of flow at a specified frequency; others, such as drainage systems involving detention storage, depend on runoff hydrographs that estimate the pre- and post-development peak discharge.

6.6.1 REFERENCES

Publications

The following references are needed in order to prepare a hydrological study. Because of the availability of computer software to perform the calculations, updated versions of these publications frequently delete the basic reasoning and understanding of principles used to develop the software. Therefore, some references also include the original publications.

- *Urban Hydrology for Small Watersheds*, Technical Release No. 55 (TR-55), NRCS, Revised 2003

- *Hydraulic Design of Highway Culverts*, Highway Design Series 2 (HDS-2), FHWA, 2002
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4 (HDS-4), FHWA, 1983 and 2001
- *Hydrology*, Hydraulic Engineering Circular 19 (HEC-19), FHWA, 1984
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular 22 (HEC-22), FHWA, 2001
- *Magnitude and Frequency of Floods on Nontidal Streams in Delaware*, U.S. Geological Survey Scientific Investigations Report 2006-5146, (http://pubs.usgs.gov/sir/2006/5146/) - regression equations

Tables and Figures

The following tables and figures are required to perform a hydrological study of a watershed:

- DelDOT's Criteria for Design Frequency
  - Figure 6-1
- Frequency Factor, C_f
  - Section 3.2.2.1, HEC-22
- Intercept Coefficient (k) for Velocity vs. Slope Relationship
  - Figure 6-4
  - Table 3-3, HEC-22
- Delaware's Rainfall Intensity Estimate
  - Figures 6-5 to 6-7
- Runoff Coefficients for Rational Method
  - Figure 6-8
  - Table 5.7, HDS-2
  - Appendix B, Tables 11 and 12, HDS-4
  - Table 3-1, HEC-22
- Nomograph for Determining Velocities Using the Upland Method of Estimating T_{t1} and T_{t2}
  - Figures 37 and 52, HEC-19
  - Figure 3-1, TR-55
- Soil Group Descriptions for TR-55 Method
  - Figure 6-9
  - Appendix A, TR-55
- Hydrologic Soil Groups for Delaware
  - Figure 6-10
  - Exhibit A, TR-55
- Delaware's 24-Hour Rainfall Depths
  - Figure 6-11
- Runoff Curve Numbers
  - Table 5.4, HDS-2
  - Table 2-2a thru d, TR-55
- Manning's Roughness Coefficients (n)
  - Table 2.1, HDS-2
  - Table 3-4, HEC-22
  - Table 3-1, TR-55
- Unit Peak Discharge (q_u) for NRCS Type II Rainfall Distribution
  - Exhibit 4-II, TR-55
- Adjustment Factor (F_p) for Pond & Swamp Areas Spread Throughout the Watershed
  - Table 5.6, HDS-2
  - Table 3-9, HEC-22
  - Table 4-2, TR-55

Computer Software

- TR-20
- TR-55
- WINTR-55
- PondPack
- WMS (Watershed Modeling System)
- HEC-HMS

6.6.2 DESIGN FREQUENCY

Since it is not economically feasible to design drainage facilities for the maximum potential runoff, a limit on the runoff for the respective design frequency must be specified. The frequency at which the corresponding flood can be expected to occur is the reciprocal of the probability that the expected flood will be equaled or exceeded in a given year. The frequency period is also known as the recurrence interval or return period. The risk of flooding and economy in construction are the two factors that govern the design storm frequency to be used for designing any drainage facility. See Figure 6-1 for design frequency criteria.

6.6.3 PEAK DISCHARGE

Peak discharge is the maximum rate of flow that results from a storm corresponding to the selected design frequency for a particular drainage location being analyzed or designed. There are numerous methods available for determining peak discharge. For roadway drainage DelDOT uses the Rational Method for homogenous drainage areas such as pavement and stormwater drain system design. This method is most accurate for watersheds smaller than 50 acres, but can be used for watersheds up to 200 acres. The NRCS TR-55 Method is used for complex watersheds up to 300 acres having several subareas that may or may not be homogenous. This method also can be used for stormwater management analysis such as detention, retention and control structures. Areas larger than this normally require drainage facilities that fall within the responsibility of the Bridge Design section. Both of these methods are briefly discussed in following sections. Example 1, Attachment B, illustrates the use of the rational method; Example 2 uses TR-55.

6.6.3.1 THE RATIONAL METHOD

The rational method employs an empirical equation relating runoff to rainfall intensity. It estimates the peak rate of runoff using: the drainage area; a weighted runoff coefficient that considers existing and proposed land use and types of soils and surfaces; and the rainfall intensity.

The Rational Method's equation is expressed as:

$$Q = C_f CIA \qquad (6.1)$$

where:

Q = Peak flow (ft^3/s)

C_f = Dimensionless frequency factor for design frequencies greater than 10-yr

C = Dimensionless runoff coefficient, a function of the watershed's ground cover

I = Design storm rainfall intensity (in/hr)

A = Drainage area (ac)

This equation is most reliable for surfaces that are smooth, uniform and impervious such as pavements. The equation's reliability is subject to three basic assumptions: (1) the peak runoff at any point is a direct function of the average rainfall intensity for the time of concentration to that point; (2) the recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity; and (3) the time of concentration is the time required for the runoff to become established and flow from the most distant point of the drainage area to the point of discharge.

For a drainage area consisting of different subareas with varying soil characteristics and cover, different runoff coefficients are assigned and a weighted average is developed using the equation:

$$C = \frac{\sum C_x A_x}{A_{total}} \qquad (6.2)$$

For more detailed information about the rational formula refer to HDS-2 and HEC-22.

PROCEDURE

Step 1. Obtain the following information for each design segment:

a. Drainage area;

b. Land use (percentage of impermeable area such as pavement, sidewalks, roofs, etc.);

c. Soil types (highly impermeable or impermeable soils);

d. The hydraulically most distant point of the drainage area to the point of discharge with significant contribution;

e. Difference in elevation from the farthest point of the drainage area to the point of discharge.

Step 2. Determine time of concentration (t_c)

The time of concentration (t_c) is the maximum time required for the runoff to flow from the most remote point in a drainage area

or subarea to the proposed point of discharge. Among a number of alternative paths that runoff could take from far distant points of a drainage area, the time of concentration represents the longest possible travel time (which is not necessarily the longest distance). The path of runoff that governs the time of concentration is the longest hydraulic length of the drainage area. It may be necessary to check several watercourses to determine the longest flow path with significant contribution.

The type and length of flow vary over the hydraulic length. The characteristics of these flow types divide them into three flow patterns: overland (sheet) flow (T_{t1}), shallow concentrated flow (T_{t2}), and open channel flow (T_{t3}). The time of concentration is the summation of the travel times for each type of flow over each of their hydraulic lengths. The minimum value of t_c is 6 minutes (0.1 hr) and the maximum is 600 minutes (10 hr).

The factors affecting the time of concentration and the time of flow are:

1. **Surface roughness:** Development in the watershed changes the flow velocity retardance from very slow shallow overland flow through vegetation, redirecting the flow to impervious areas such streets and gutters, to storm drains that transport the runoff more rapidly. Therefore, the travel time is usually decreased.
2. **Channel shape and flow pattern:** Travel time in small watersheds is influenced by the resulting upstream overland flow. In developing or developed areas, the runoff is directed to defined channels as soon as possible. This results in increased flow velocity and decreased travel time.
3. **Slope:** Drainage designs have a variety of site grading solutions to manage water flow. If there is extensive use of swales, ditching or storm drains, the slopes within the watershed can be modified dramatically. Generally, swales and ditches increase slopes, and storm drains decrease slopes.

Overland (Sheet Flow) takes place on plane surfaces, nonconverging irregular surfaces, usually at the uppermost areas of watersheds, and is less than 1.5 inches deep. The hydraulic length for sheet flow is normally assumed to be a maximum 100 feet for unpaved surfaces and 150 feet for paved surfaces. The equation for sheet flow is a version of the kinematic wave equation and is:

$$T_{t1} = \frac{0.93}{I^{0.4}}\left(\frac{nL_1}{\sqrt{s}}\right)^{0.6} \qquad (6.3)$$

where:

T_{t1} = Travel time (min)

n = Roughness coefficient from Table 3-1, TR-55 or Table 3-2, HEC-22

L_1 = Length of flow, (ft)

I = Average rainfall excess intensity for a storm duration = t_c (in/hr)

s = Surface slope (ft/ft)

I depends on t_c and is initially assumed from Figures 6-5, 6-6 or 6-7. t_c is calculated from equation 6.4 and checked against the initial assumed value. This is repeated until the two successive values are the same.

NRCS (TR-20), has also developed a method for determining T_{t1} for overland flow based on a 24-hr Type II rainfall distribution, eliminating the need for trial and error calculations until T_t equals storm duration. The equation is:

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}^{\ 0.5}s^{0.4}} \qquad (6.4)$$

where:

T_{t1} = Travel time (min)

n = Roughness coefficient from Table 3-1, TR-55, Table 3-2, HEC-22 or Table 2-1, HDS-2.

L_1 = Length of flow, (ft)

P_{24} = 2-yr Type II 24-hr rainfall (in)

s = Surface slope (ft/ft

The **Upland (Velocity) Method** is used to determine the travel time for shallow concentrated and open channel flow segments by dividing their respective hydraulic lengths by each segment's average flow velocity:

$$T_t = \frac{L}{60V} \qquad (6.5)$$

where:

T_t = Segment travel time (min)

L = Segment length (ft)

V = Segment velocity (ft/s)

Each segment's velocity depends upon the watercourse's average slope and surface roughness.

Shallow Concentrated Flow takes over from sheet (overland) flow as very shallow channels or gutters along the hydraulic length, 1.5 to 6.0 inches in depth. Limit sheet flow to 100 feet for unpaved areas and 150 feet for paved areas. The equation for determining the average velocity for shallow concentrated flow is:

$$V = 32.8ks^{0.5} \qquad (6.6)$$

where:

V = Average velocity (ft/s)

k = velocity intercept coefficient based on slope and surface type, Figure 6-4

s = average slope of the watercourse (ft/ft)

V can be obtained graphically from Figures 37 or 52, HEC-19; Figure 4b, HDS-4; or Figure 3-1, TR-55. The travel time (T_{t2}) is determined by using equation 6.5.

Figure 6-4
Intercept Coefficient (k) for Velocity vs. Slope Relationship for Rational Method

Land Cover and (Type of Flow)	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation, contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow)	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

Open Channel Flow includes natural streams, ditches, pipes and gutters in the lower stretch of the watershed along the hydraulic length to the point of termination at an outfall. Determining when channel flow begins is based on several indicators, including survey data defining the channel, visible signs on aerial photography of the watershed, on area topographic maps, and USGS quadrangle sheets.

Determine the velocities by using Manning's equation:

$$V = \frac{1.49(R)^{2/3}(s)^{1/2}}{n} \qquad (6.7)$$

where:

V = Average velocity (ft/s)

R = Hydraulic radius (ft) = A/P_w (6.7a)

A = Cross sectional flow area (ft^2)

P_w = Wetted perimeter (ft)

s = Energy grade line (channel or pipe flow line) (ft/ft), and

n = Manning's roughness coefficient for open channel flow

The travel time for open channel flow, T_{t3}, is obtained from equation 6.5. In developed areas there may be a variety of open channel facilities including storm drains, swales, ditches, gutters. The travel time for each one of these is computed by using Manning's formula and then combined to provide T_{t3} for the complete open channel flow in the watershed.

The time of concentration, t_c, is equal to the sum of the travel times derived for all three types of flow:

$$t_c = T_{t1} + T_{t2} + T_{t3} \qquad (6.8)$$

Step 3. Determine the rainfall intensity (I)

Rainfall intensity is the amount of rainfall in inches per hour based on a duration that equals the time of concentration and the design storm frequency. The rainfall intensity can be obtained from Figures 6-5 to 6-7 provided that the time of concentration (duration) is known.

Step 4. Select the appropriate runoff coefficient (C)

The rational method uses a runoff coefficient that is representative of the drainage area being studied. This coefficient averages the area ambient moisture content, infiltration/ evaporation potential, average slope, existing and future ground cover, and developed surfaces.

The drainage area is divided into its various pre- and post-developed subareas. Using Figure 6-8, a runoff coefficient is assigned and a weighted coefficient is calculated. Higher values of C are used for steeply sloped areas and longer return periods because infiltration and other losses have a reduced effect on the runoff.

Step 5. Compute the design peak flow (Q)

Determine the peak discharge using the Rational Method equation, $Q = C_f CIA$.

Figure 6-5
Rainfall Intensity Estimates (in/hr) for Rational Method - New Castle County

Frequency (yr)	Duration (min)									
	5	10	15	30	60 (1 hr)	120 (2 hr)	180 (3 hr)	360 (6 hr)	720 (12 hr)	1440 (24 hr)
2	4.97	3.97	3.33	2.30	1.44	0.87	0.62	0.38	0.23	0.13
5	5.83	4.67	3.94	2.80	1.79	1.08	0.78	0.48	0.29	0.17
10	6.42	5.13	4.33	3.13	2.04	1.24	0.89	0.55	0.34	0.20
25	7.13	5.68	4.80	3.55	2.37	1.45	1.05	0.66	0.41	0.25
50	7.60	6.05	5.10	3.84	2.60	1.61	1.18	0.74	0.47	0.29
100	8.06	6.40	5.40	4.13	2.85	1.77	1.30	0.83	0.53	0.34
200	8.44	6.69	5.63	4.38	3.07	1.93	1.43	0.92	0.60	0.39
500	8.88	7.02	5.89	4.69	3.36	2.14	1.60	1.05	0.70	0.46

Figure 6-6
Rainfall Intensity Estimates (in/hr) for Rational Method - Kent County

Frequency (yr)	Duration (min)									
	5	10	15	30	60 (1 hr)	120 (2 hr)	180 (3 hr)	360 (6 hr)	720 (12 hr)	1440 (24 hr)
2	5.06	4.05	3.40	2.34	1.47	0.90	0.65	0.40	0.24	0.14
5	6.01	4.81	4.06	2.88	1.85	1.13	0.82	0.50	0.30	0.18
10	6.68	5.35	4.51	3.27	2.13	1.31	0.95	0.59	0.36	0.21
25	7.54	6.01	5.08	3.76	2.50	1.56	1.14	0.71	0.44	0.27
50	8.15	6.49	5.48	4.13	2.79	1.76	1.29	0.81	0.51	0.32
100	8.76	6.96	5.86	4.49	3.09	1.96	1.45	0.92	0.59	0.37
200	9.92	7.39	6.22	4.84	3.39	2.17	1.62	1.04	0.67	0.43
500	10.02	7.93	6.65	5.29	3.80	2.45	1.85	1.21	0.80	0.52

Figure 6-7
Rainfall Intensity Estimates (in/hr) for Rational Method - Sussex County

Frequency (yr)	Duration (min)									
	5	10	15	30	60 (1 hr)	120 (2 hr)	180 (3 hr)	360 (6 hr)	720 (12 hr)	1440 (24 hr)
2	5.06	4.04	3.39	2.34	1.47	0.91	0.66	0.40	0.24	0.14
5	6.02	4.83	4.07	2.89	1.85	1.16	0.84	0.52	0.30	0.19
10	6.76	5.40	4.56	3.30	2.15	1.35	0.99	0.61	0.36	0.22
25	7.67	6.11	5.15	3.82	2.54	1.61	1.19	0.74	0.45	0.28
50	8.32	6.62	5.59	4.21	2.85	1.83	1.35	0.85	0.52	0.33
100	8.96	7.12	6.00	4.59	3.16	2.05	1.53	0.97	0.61	0.38
200	9.60	7.61	6.40	4.98	3.49	2.28	1.71	1.10	0.70	0.45
500	10.38	8.21	6.88	5.48	3.93	2.59	1.97	1.28	0.84	0.54

Note: Interpolation shall be used for rainfall for intermediate durations.

Figure 6-8
Recommended Runoff Coefficients (C) for Rational Method

Type of Drainage Area Surface			Runoff Coefficient, C
Earth Surfaces	Sand, from uniform grain size with no fines to well graded with clay or silt	Bare	0.15
		Light Vegetation	0.10
		Heavy Vegetation	0.05
	Loam, from sandy or gravelly to clayey	Bare	0.20
		Light Vegetation	0.10
		Dense Vegetation	0.05
	Gravel, from clean gravel & gravel sand mixtures with no silt or clay to high clay or silt contents	Bare	0.25
		Light Vegetation	0.15
		Dense Vegetation	0.10
	Clay, from coarse sandy to pure colloidal clays	Bare	0.40
		Light Vegetation	0.30
		Dense Vegetation	0.25
Lawns	Sandy soil	Slope, Flat to 2%	0.05
		Average Slope 2% to 7%	0.10
		Steep Slope Over 7%	0.15
	Clayey soil	Slope, Flat to 2%	0.13
		Average Slope 2% to 7%	0.18
		Steep Slope Over 7%	0.25
Pavements	Asphalt		0.95
	Concrete		0.95
	Compacted graded aggregate or gravel		0.70
Railroad yard areas			0.30
Parks, golf courses and cemeteries			0.15
Playgrounds			0.25
Unimproved Areas			0.15
Cultivated areas			0.30
Swamp or marsh areas			0.08
Roofs			0.90
Drives and walkways			0.90
City business areas			0.85
City with dense residential areas and varying soil and vegetation conditions			0.70
Residential areas, single family units			0.40
Residential areas, duplexes and twins			0.50
Residential areas, multi-units			0.70
Suburban residential areas (lots 1/2 ac or more)			0.35
Apartment complexes			0.60
Light industrial areas			0.70
Heavy industrial areas			0.80

6.6.3.2 THE NRCS METHOD

NRCS Technical Release 55 (TR-55, 1986; WINTR-55, 2003), *Urban Hydrology for Small Watersheds* is one of the methods commonly used to determine a watershed's peak discharge. TR-55 is a computer application based on data and routines developed using TR-20. This method is particularly useful for determining pre- and post-development runoff rates and the subsequent design of any required stormwater control structures.

The two most commonly used design applications for roadway drainage in TR-55 are: (1) the Graphical Peak Discharge method and (2) the Tabular Hydrograph method. The Graphical method is limited to a single homogenous watershed where land use, soil, and cover are uniformly distributed throughout the watershed. The Tabular method uses hydrographs for analyzing a larger more complex watershed having: multiple homogenous subareas with several converging reaches; changing land uses within any of the existing homogenous watershed; and storage and control structures. Worksheets for each method are provided in TR-55.

The two methods use the same approach to determine the basic design data necessary to for calculating the watershed('s) peak discharge: the runoff depth, time of concentration and travel time within the watershed(s) and an initial abstraction factor (I_a) based on the CN to determine the unit peak discharge per square mile per inch of runoff. Equations convert the runoff depth, the unit peak discharge, the drainage area, and a storage adjustment factor to calculate the peak watershed discharge or generate a pre- and post-hydrograph of the peak discharge.

The study begins by defining the watershed and dividing it into homogenous subareas using existing and proposed land uses. Land use determines the percentage of pervious and impervious surfaces. The drainage runoff characteristics are also based upon the hydrological soil classification. A runoff curve number (CN) is assigned to these subareas. CN represents the hydrologic characteristics of the drainage area including the average slope, soil group type, plant cover, amount of impervious surfaces, interception rates, and surface storage. Using the runoff curve number and the NRCS 24-hour rainfall data, the design storm(s) runoff depth for the watershed(s) is determined as follows:

Runoff Depth (q_d)

The runoff depth equation is:

$$q_d = \frac{(P_{24} - 0.2S))^2}{P_{24} + 0.8S} \qquad (6.9)$$

where

q_d = Runoff depth for a specified return period (in)

P_{24} = 24-hour rainfall using Figure 6-11 for the corresponding return period (in)

S = Potential maximum retention after runoff begins, inches

S is related to the soil and cover conditions through the runoff curve number for the various subareas and defined by the equation:

$$S = \frac{1000}{CN} - 10 \qquad (6.10)$$

where

CN = Weighted average runoff curve number using a ratio of assigned subarea CN's to their area as related to the entire watershed area. CN must 40 or greater.

The hydrologic soil group(s) in each subarea is based on the soil classification (denoted as A, B, C and D) based on their rainfall infiltration rates. A represents soils with the maximum rate of rainfall infiltration and D the soils with the lowest rate of infiltration.

Refer to Table 2-2a-d, TR-55, for the runoff curve numbers for urban and rural areas. These tables were developed assuming an Initial abstraction (I_a) of 20% of the potential maximum retention after runoff begins (S) in the drainage area. I_a represents

losses prior to runoff and includes initial infiltration storage in surface depressions, evapotranspiration, leaf and vegetation retention, and other factors. Cover types represented by vegetation, bare soil, and impervious surfaces are usually determined from field reconnaissance, aerial photographs, land use maps, and soil survey maps. Hydrologic condition indicates infiltration and runoff; it is estimated from the density of plant and residue cover in rural areas.

A soil survey for each county is available from the United States Department of Agriculture. These publications contain aerial photographic maps of Delaware with detailed soils information. Figure 6-10 lists the hydrologic soil groupings commonly found in Delaware. Soil survey maps for Kent and Sussex counties are available online at the web site http://www.udel.edu/FREC/spatlab for the University of Delaware Spatial Analysis Lab.

After determining CN and the 24-hour rainfall for the watershed, the runoff depth (q_d) is calculated using Worksheet 2, figure 2-1 and table 2-1 in TR-55 or equations 6.9 and 6.10.

Next, the travel time for each type of flow and the ultimate time of concentration to the point of interest must be determined.

Time of Concentration and Travel Time (t_c)

The flow times for the three types of flow within each watershed (as described in Section 6.4) are calculated as follows.

Sheet (Overland) Flow

The travel time when using TR-55 is obtained from the modified Manning's-kinematic solution and is determined in hours, modified from equation 6.4:

$$T_{t1} = \frac{0.007(nL_1)^{0.8}}{P_2^{\,0.5} s^{0.4}} \qquad (6.11)$$

where

T_{t1} = Travel time for overland flow (hr)

P_2 = 2-year, 24-hour rainfall depth (in)

n = Manning's roughness coefficient for sheet flow (Table 3-1, TR-55)

L_1 = Portion of hydraulic length on which sheet flow takes place (ft)

s= Average slope of the hydraulic length L_1 (ft/ft)

The maximum for L_1 is 100 feet for unpaved surfaces and 150 feet for paved surfaces.

Shallow Concentrated Flow

For shallow concentrated flow, the time of concentration (T_{t2}) is computed using equation 6.5. V_2 is obtained from Table 3-1, TR-55, or the equations:

$$V = 16.10(s)^{0.5} \quad \text{Unpaved surfaces} \quad (6.12)$$

$$V = 14.99(s)^{0.5} \quad \text{Grassed waterway} \quad (6.12a)$$

$$V = 20.30(s)^{0.5} \quad \text{Paved surfaces} \quad (6.13)$$

where

V = Average velocity (ft/s), and

s = Slope of the hydraulic gradeline (watercourse slope) (ft/ft)

Open Channel Flow

The travel time for open channel flow (T_{t3}) is determined by using Manning's equation, and equations 6.3 and 6.5 When using equation 6.5 use 3600 instead of 60 to calculate hours instead of minutes.

The time of concentration for the drainage basin (t_c) is the sum of the travel time for each flow segment. The minimum value for this method is 6 minutes (0.1 hr) and a maximum of 10 hours. See Worksheet 3, TR-55, for the procedure.

At this point the designer must decide on the scope of the watershed analysis. If the watershed is hydrologically homogenous with land use, soils, and cover uniformly distributed with one main watercourse, then the Graphical Peak Discharge Method can be used. If the watershed has nonhomogeneous areas that can be divided into homogenous areas, land use changes are proposed for a

portion of the watershed and stormwater management facilities requiring hydrographs are necessary, then the Tabular Hydrograph Method is used.

There are other regulatory controls that influence the selected method. State sediment and stormwater policy is that all hydrologic computations must use the NRCS Type II-24 hour rainfall event and for all projects south of the Chesapeake and Delaware Canal, the Delmarva Unit Hydrograph will be incorporated into the design process.

GRAPHICAL PEAK DISCHARGE METHOD

The peak discharge at the site under consideration is obtained from the equation:

$$Q_i = q_u A_m q_d F_p \qquad (6.14)$$

where,

Q_i = Peak discharge for the *i*'th year frequency (ft^3/s)

q_u = Unit peak discharge (ft^3/s /sq mi/in)

A_m = Drainage area (sq mi)

q_d = Runoff depth (in) (equation 6.9)

F_p = Pond and adjustment factor (Table 4-2, TR-55)

The only unknown variable in this equation is q_u. The steps to find the value of q_u are:

1. Using the weighted CN determine the initial abstraction (I_a) from Table 4-1, TR-55.
2. For the design frequency, select the 24-hour rainfall (P) from Figure 6-11.
3. Calculate the ratio of I_a/P.
4. Peak discharge per square mile per inch of runoff (q_u) is obtained from exhibit 4-II for Type II Rainfall Distribution, the previously defined t_c and the I_a/P ratio.

Worksheet 4, Appendix D, TR-55, provides an easy guide for calculating Q_i. Also, the equations used to generate the figures and tables in TR-55 can be found in Appendix F, TR-55.

TABULAR HYDROGRAPH METHOD

First, use worksheet 5a to develop a summary of basic watershed data by subarea. The total t_c for each subarea is the total travel time for the subarea reach. Find the hydrograph coordinates for selected t_c's using exhibit 5. The flow time is found using the equation:

$$q = q_t A_m Q \qquad (6.15)$$

where:

q = Hydrograph coordinate (cfs) at hydrograph time (t)

q_t = Tabular hydrograph unit discharge from exhibit 5 (csm/in)

A_m = Individual subarea drainage area (mi^2)

Q = Runoff (in)

The data needed to model the present and future conditions of the watershed using hydrographs for each subarea and routing these hydrographs to the watershed control structures and outfall are:

1. Drainage area for each homogenous subarea. (mi^2)
2. Time of concentration for each subarea in hours. See the previous discussion and Chapter 3, Worksheet 3, TR-55 for the procedure.
3. Time of travel for each routing reach in hours. See the previous discussion and Chapter 3, Worksheet 3, TR-55 for the procedure
4. The weighted CN for each subarea. See Table 2-2 and Worksheet 2, TR-55 for the procedure.
5. 24-hour rainfall for the design frequency using the Delmarva Unit Hydrograph
6. Runoff, in inches, for each subarea. See the previous discussion and Chapter 2, Worksheet 2, TR-55 for the procedure.
7. Initial abstraction (I_a) from Table 5-1, TR-55 .

8. Calculate the ratio of I_a/P for each subarea.
9. Develop a composite hydrograph by summing prerouted individual subarea hydrographs using Worksheet 5b, TR-55.

Chapter 5 in TR-55 describes the procedure for developing a watershed's composite flood hydrograph with the limitations that may apply.

Figure 6-9
Soil Group Descriptions for TR-55 Method

Soil Group	Hydrologic Description	Soil Type
A	High infiltration and low runoff, very well drained sands or gravels	Sand, loamy sand or sandy loam
B	Moderately well drained soils with fine to moderately coarse textures	Silt loam or loam
C	Low infiltration rate, moderately fine to fine texture	Sandy clay loam
D	Very low infiltration rate, high runoff potential, predominately clayey soil	Clay loam, silty clay loam, sandy clay, silty clay or clay

Figure 6-10
Hydrologic Soils Descriptions for TR-55 Method

Aldino	C	Elsinboro	B	Keyport	C	Pepperbox	B
Assawoman	D	Evesboro	A	Kinkora	D	Plummer	D
				Klej	B	Pocomoke	D
Bayboro	D	Fort Mott	A			Portsmouth	D
Berryland	D	Fallsington	D	Mulica	D	Pone	D
Butlertown	C			Manor	B		
		Greenwich	B	Metapeake	B	Rumford	B
Calvert	D	Gleneig	B	Matawan	C	Rutlege	D
Chester	B			Mattapex	C	Rosedale	A
Codorus	C	Hammonton	C	Montalto	C		
Collington	B	Hatboro	D			Sassafras	B
Comus	B	Hurlock	D	Nanticoke	D		
				Neshaminy	B	Talleyville	B
Delanco	C	Johnston	D				
				Osier	D	Watchung	D
Elioak	C	Kalmia	B	Othello	D	Woodstown	C
Elkton	D	Kenansville	A				

Other common hydrologic soils

Borrow Pits	Extremely variable	Ingleside	A
Coastal Beaches	A	Swamp	D
Made land	C	Broadkill, Westbrook	D
Mixed alluvial	C-D	Urban Land	C-D
Mixed silts and clay	C		

Figure 6-11
24-Hour Rainfall Depths for Delaware for TR-55 Graphical Method

NRCS Type II, 24-Hour Duration			
Storm Event	County		
	New Castle	Kent	Sussex
1-yr	2.7	2.7	2.8
2-yr	3.2	3.3	3.4
5-yr	4.1	4.3	4.4
10-yr	4.8	5.2	5.3
25-yr	6.0	6.5	6.7
50-yr	6.9	7.6	7.9
100-yr	8.0	8.9	9.2
500-yr	10.9	12.6	13.0

6.7 OPEN CHANNEL FLOW

Open channels are the most commonly used component of a drainage system. They may occur naturally within the drainage basin or be artificially introduced in the design process. Open channels include natural/manmade ditches, streams, median swales and gutters. They may be unlined or lined with artificial or natural materials to protect against erosion.

In drainage design, the term "open channel" is frequently used to describe the design and flow characteristics of all drainage systems (ditches, storm drains or culverts) that have their flow controlled by atmospheric pressure and gravity. The primary design control is the difference in elevation between the various open channel sections. However, the open channel's shape, surface type, coefficient of friction, and alignment are considered in determining its flow.

In open channel design, the final geometric parameters are selected to ensure that the flow does not exceed critical depth or critical velocity. This is an iterative process to reach the appropriate design.

Open channels are used:

- Longitudinally as intercepting channels at the top of a cut section;
- In cut sections of roadways to remove stormwater;
- In medians to covey roadway runoff to inlets:
- At the bottom and toe of embankment slopes to convey stormwater to a discharge point.

6.7.1 REFERENCES

Publications:

The primary references for understanding the principles and analysis of open channel flow are:

- *Design Charts For Open-Channel Flow*, Hydraulic Design Series No. 3, (HDS-3) FHWA, 1973
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4, (HDS-4) FHWA, 1983
- *Design of Riprap Revetment*, Hydraulic Engineering Circular 11 (HEC-11), FHWA, 1989
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, Hydraulic

Engineering Circular 14 (HEC-14), FHWA, 1975

- *Design of Roadside Channels with Flexible Lining*, Hydraulic Engineering Circular 15 (HEC-15), FHWA, 1986 and 2005
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular 22 (HEC-22), FHWA, 2001
- *Open-Channel Hydraulics*, Ven Te Chow, 1959

Tables and Charts:

Access to the following tables and charts is needed for analyzing open channel flow:

- Open Channel Flow Schematic
  - Figures 6 and 7, HDS-4
- Manning's Roughness Coefficients *n* for Open Channels
  - Figure 6-16
  - Table 1, HDS-3
  - Tables 12 and 14, HDS-4
  - Tables 2.1 and 2.2, HEC-15
- Channel Geometry Equations
  - Appendix B, HEC-15
  - Table 2-1, Chow
- Nomograph for Solution of Manning's Equation
  - Chart 83, HDS-3
  - Appendix C, Chow
- Solution of Manning's Equation for Channels of Various Side Slopes
  - Chart 16, HEC-12
- Capacity of Trapezoidal Channels
  - HDS-3 and HDS-4
- Geometric Design Chart for Trapezoidal Channels
  - HDS-3
  - Appendix B, Chow
- Uniform Flow in Trapezoidal Channels by Manning's Equation
  - Table B.1, HEC-14
- Open Channel Design Procedure
  - Figure 6-12
- Maximum Non-Scouring Ditch Grades with Grass Lining
- Permissible Velocities for Channels Lined with Grass
  - Figure 6-14
- Maximum Allowable Velocities for Open Channels
  - Figure 6-15
- Channel Lining Design Procedure
  - Figure 6-17
- Channel Lining Design Computation Chart
  - Figure 6-18
- DelDOT's Roadside Ditch Design Form
  - Example 4, Attachment B
- K_b Factor for Maximum Stress on Channel Bends
  - Chart 21, HEC-22
- Stability Factor Selection Criteria
  Chart 3, HEC-11
- Correction Factor for Riprap Size
  - Chart 2, HEC-11
- Riprap Size Relationship
  - Chart 1, HEC-11

Computer Software

- HYDRAIN
- HY7 WSPRO
- HEC-RAS

6.7.2 OVERVIEW

Most roadside drainage design is assumed to be steady, uniform flow. The designer assumes that the channel cross section, slope, alignment and surface roughness are reasonably constant over a sufficient length. For steady, uniform flow in open channels, Manning's equation (Equation 6.7) is used to determine the velocity.

The capacity of the channel is determined by:

$$Q = VA \qquad (6.16)$$

Combining the two equations results in:

$$Q = A\frac{1.49}{n}R^{2/3}s^{1/2} \qquad (6.17)$$

In the above equations:

V = Mean velocity (ft/s)

n = Manning's roughness coefficient

R = Hydraulic Radius (ft), the ratio of flow area of cross section (A) to the wetted perimeter (P), A/P_w

s = Slope of the energy grade line (ft/ft). Usually it is parallel to the water surface or the profile of the channel bottom.

Q = Rate of flow (ft^3/s)

A = cross sectional area of the flow (ft^2)

Manning's roughness coefficient n may vary with the depth of flow. A range of n values for different types of open channels is provided in the referenced resources.

To simplify solving the equations a conveyance factor (K) can be developed in a tabular or curve form for the more commonly used channel cross sections based on the following equation:

$$K = \frac{1.49}{n} AR^{2/3} \qquad (6.18)$$

As a result, substituting K into Manning's equation the rate of flow, Q, would be:

$$Q = Ks^{1/2} \qquad (6.19)$$

6.7.3 ROADSIDE DITCHES

Roadside ditches are trapezoidal or V-shaped open channels that usually parallel the roadway and are lined with grass or non-erodible materials required for erosion protection. Even in curb sections the designer may need to include a roadside ditch in the drainage design. Median ditches and other small-excavated channels are designed using the same principles as for roadside ditches.

Roadside ditches provide an opportunity to reduce runoff pollutants with infiltration through permeable (vegetative) surfaces that allow the pollutants to be absorbed into the underlying soils and reduce the peak flow velocity. The swales and ditches include stone check dams, vegetated beds, and banks with shallow longitudinal slopes and relatively shallow side slopes. One of the controlling factors is the design flow.

Roadside ditches serve several purposes:

- To collect runoff from the highway and adjacent areas which may sometimes extend outside the right-of-way, and dispose of the accumulated runoff at suitable outlet locations,
- To drain the pavement structure and the top layer of supporting subgrade to prevent saturation and loss of support for the pavement, and to protect it from damage from freeze-thaw cycles, and
- To control the quantity and quality of roadway runoff while minimizing any impact to adjacent properties. Draining runoff onto adjacent properties should only be considered when absolutely necessary; in this case, temporary and/or permanent easements may be needed.

6.7.3.1 DESIGN CRITERIA

In additional to any controls set in other chapters of this manual, the following criteria is to be used in the design of roadside ditches:

1. The rational method is used to compute the design runoff, and Manning's equation for capacity. Figure 6-1 has design frequencies.
2. Roadside ditches shall conform to clear zone criteria and legal requirements. Safety of motorists, minimum maintenance requirements, aesthetic values, and avoiding potential damage to abutting properties as well as adverse environmental impacts are essential considerations in design. Refer to AASHTO's *Roadside Design Guide*.
3. Maintain a minimum freeboard of 0.5 ft minimum, 1 ft preferred, below the edge of shoulder.
4. For adequate subbase and pavement drainage, the bottom of the roadside ditch should be at least 2.5 feet below the shoulder edge of the traveled way.
5. Side slopes for grass-lined ditches shall be 3:1 or flatter to facilitate mowing operations. The designer should review the current roadside vegetation to ensure the proper selection of a roughness coefficient.

6. At the top of cut slopes, an intercepting ditch should be investigated. Every effort shall be made to prevent sediment and debris from filling in the ditch.
7. All roadside ditches are to be lined with grass or special materials for scour protection. The ditch linings are to be designed to withstand potential erosion.
8. The minimum ditch grade is 0.3%, but 0.5% is preferred.
9. Gradual transitions should be used for cross section changes. Normally, transition rates are 25 ft for every foot change in the ditch bottom width and 100 ft for increasing or decreasing the value of *z* (horizontal component of the side slope) by 1.
10. After completing the design, the designer should ensure that the proposed right-of-way and easements are adequate to construct the selected ditch sections.

6.7.3.2 DESIGN PROCEDURE

Since the channel's *n* value changes with time, typically (but not always) becoming higher as vegetation is established, the design section shall be checked for velocity constraints based on the lowest *n* value for the newly constructed channel that will yield the highest velocity. The channel shall also be checked with a higher *n* factor for the mature growth stage to establish the minimum capacity of the channel. The vegetative retardance factor is directly related to the selected *n* value. Curves for these relationships can be found in Chow's *Open Channel Hydraulics*, HEC-15, and the NRCS *Engineering Field Handbook*.

The design procedure consists of three phases:

1. Design the ditch for stability. Determine the dimensions using a low retardance factor, since the permanent stand of grass has not been established.
2. Design for the maximum capacity by determining the increase in depth of flow necessary to provide the maximum capacity under the assumed final ditch lining conditions.
3. After checking these two conditions and deciding on the final dimensions, the design criteria freeboard is added.

Refer to Sections 6.10, 11 and 12 for a discussion on designing roadside ditches as a part of stormwater management. These sections also more fully describe the phased approach to ditch design.

The following design procedure is used for roadside ditches.

1. A preliminary layout of the roadside ditch system with flow patterns shall be drawn on the construction plans along with other drainage facilities (existing and proposed), utilities, right-of-way and other specific details that may influence or restrict the ditch design.
2. In general, the grade of the roadside ditch should closely follow the grade line of the highway. Deep ditches and frequent breaks in grade should be avoided. The distance between break points should preferably be at least 100 ft. Use Figure 6-13 as a guide for limiting the maximum grade of roadside ditches. Working plans showing the ditch layout should also include the ditch profile and all features above and below ground that may be potential obstructions to the proposed ditch.
3. Highway cross sections, usually at 50-ft intervals, showing the natural ground and the proposed highway section including all roadside ditches used in establishing the construction limits within the proposed right-of-way.
4. Schematic diagrams of the ditch plan, including geometric elements of the ditch, topographic features, and basin divides shall be drawn to facilitate the computation of peak discharges at the design frequency.
5. An approved spreadsheet (such as the one in Example 4, Attachment B, should be used for hydraulic computations. Manning's roughness coefficient can be

assumed to be 0.05 for the grass surface of the channel.

6. If the computed velocity is lower than the permissible velocity for soil with grass linings as stipulated in Figure 6-14, then permanent grass seeding and mulching per DelDOT's *Standard Specifications* should suffice.
7. If the computed velocity exceeds the permissible velocity on grass surface, then the designer must increase the ditch cross section and flatten the profile to lower the velocity of flow below the permissible limit or use a special channel lining for permanent erosion control as described in Section 6.7.4.

6.7.4 DITCH EROSION CONTROL

6.7.4.1 OVERVIEW

When the flow velocity of a roadside ditch exceeds the allowable velocity on bare earth of the channel (see Figure 6-15) during construction, then a temporary ditch lining, such as an erosion control blanket, is needed to prevent erosion of the ditch. For surface erosion due to overland flow, the permissible velocities of Figure 6-14 should also apply.

See DelDOT's *ES_2M Design Guide*, *Standard Construction Details*, and the *Delaware Erosion & Sediment Control Handbook* for the requirements and construction details for erosion and sediment control items to be used during construction.

In preparing an H&H report, the post-construction ditch section is analyzed. Usually most surfaces will have grass or some other type of vegetation. As a result, the permissible velocity increases on all types of soil used in construction. In spite of the growth of vegetation, the permissible velocity may be exceeded in certain cases that would necessitate the use of a permanent lining in lieu of seeding and mulching. This permanent ditch lining, which is capable of withstanding erosion, may be either rigid or flexible. The rigid lining usually consists of cast-in-place concrete paving, fabric formed concrete revetment mattress, bituminous (asphaltic) concrete paving or grouted riprap. For flexible lining, alternatives include riprap with or without geotextile fabric and wire enclosed riprap (gabions).

Rigid linings can sustain high velocities up to 20 ft/s, and should be considered when the channel bed is not expected to undergo appreciable settlement. The cost of construction is a very important factor in the selection of this type of lining. Soil reinforcing mats are a cost-effective alternative to rigid linings.

Section 6.7.4.2 briefly discusses the procedure for design a flexible lining. HEC-11 and HEC-15 contain detailed information on the design, example problems and specifications for flexible lined channels. The following design procedures are based on those publications.

Figure 6-12
Open Channel Design Procedure

Open Channel Design using $Q = A\frac{1.49}{n}R^{2/3}s^{1/2}$			
Step 1	Select channel cross section (typically trapezoidal)	Step 5	Determine required depth[1]
Step 2	Determine slope (normally the centerline profile grade line ≤ 3%)	Step 6	Determine subcritical or supercritical flow[2]
Step 3	Select lining (initial trial assumed to be grass; slopes greater than 3% may need different lining)	Step 7	Calculate design velocities for erodible lining sections-check criteria[3] (reevaluate section by section as necessary)
Step 4	Determine roughness coefficient *n* (use an average value from Figure 6-16)	Step 8	Add minimum free board

[1] Use charts from HDS-3, nomograph solution for Manning's equation, and/or charts for hydraulic elements of channel sections, partial area of flow in a circular channel and depth of flow in a trapezoidal channel (also available on DelDOT's web site).

[2] Avoid supercritical flow by adjusting design parameters.

[3] Use a conservative retardance, i.e., C for channel capacity and D for flow velocity.

Figure 6-13
Maximum Non-Scouring Ditch Grades with Grass Lining[1]

Soil Type	Flow Depth (in)	Grade (%)
Erosive	6	3.0
	12	1.5
	18	0.8
Average	6	6.0
	12	3.0
	18	1.6
Non-Erosive	6	10.0
	12	5.0
	18	3.0

[1] Slopes greater than 5% are not recommended.

Figure 6-14
Permissible Velocities for Open Channels Lined with Grass

Cover Type	Slope Range (%)	Permissible Velocity (ft/s)	
		Erosion-resistant soils	Easily eroded soils
Bermuda grass	0-5 5-10 >10	8 7 6	6 5 4
Buffalo grass, Kentucky bluegrass, Smooth brome and Blue grama	0-5 5-10 >10	7 6 5	5 4 3
Grass mixture	0-5 5-10 (max)	5 4	4 3
Lespedeza sericea, Weeping lovegrass, Ischaemum (Yellow bluestem), Kudzu, Alfalfa and Crabgrass	0-5 (max)	3.5	2.5
Annuals-used for temporary erosion control such as Common Lespedeza or Sudangrass	0-5 (max)	3.5	2.5

Figure 6-15
Maximum Allowable Velocities for Open Channels

Soil type	AASHTO Classification	Maximum velocity (ft/s)	
		Bare Earth	Grass Lining
Fine sand	A-3, Beach Sand	2.0	—
Sandy loam	A-2-4 Non-plastic	2.0	4.0
Silt loam	A-4 Non-plastic	2.5	5.0
Ordinary firm loam	A-7-5 Plastic, silt clay sandy	3.0	5.0
Fine gravel	Granular	4.0	—
Alluvial silt (colloidal)	Plastic topsoil	4.0	6.0
Stiff clay	A-7-6 Clay	4.5	7.0
Graded loam to cobbles	Non-plastic soil and rock	4.5	7.0
Graded silt to cobbles	Plastic soil and rock	5.0	8.0
Coarse gravel	Creek Gravel	6.0	—
Cobbles and shingles	Soft rock	7.0	—
Shale and hardpan	Medium rock	8.0	—
Bedrock	Rock	20.0	—
Paved lining			
Concrete		18.0	
Asphalt		15.0	
Grouted riprap		18.0	

Figure 6-16
Manning's Roughness Coefficients (*n*) for Open Channels

Type of Channel and Surface Description	Range of *n* values	
	Normal	Maximum
Natural Streams		
1. Fairly regular section		
a. Some grass and weeds, little or no brush	0.030	0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035	0.050
c. Some weeds, light brush on banks	0.035	0.050
d. Some weeds, heavy brush on banks	0.050	0.070
e. Some weeds, dense willows on banks	0.060	0.080
f. For trees within channel, with branches submerged at high stage, increase all the above values by	0.010	0.020
2. Irregular sections, with pools, slight channel meander Increase respective values above by	0.010	0.020
3. Mountain streams, with pools, no vegetation in channel, bank usually steep, trees and brush along banks submerged at high flood stage		
a. Bottom gravel, cobbles, and few boulders	0.040	0.050
b. Bottom cobbles with large boulders	0.050	0.070

Excavated of Dredged Channels					
	Normal	Maximum			
1. Earth, straight and uniform			2. Earth, winding and sluggish		
a. Clean, recently completed	0.018	0.020	a. No vegetation	0.025	0.030
b. Clean, after weathering	0.022	0.025	b. Grass, some weeds	0.030	0.033
c. Gravel, uniform section, clean	0.025	0.030	c. Dense weeds or aquatic plants in deep channels	0.035	0.040
d. With short grass, few weeds	0.027	0.033	d. Earth bottom and rubble sides	0.030	0.035
			e. Stony bottom and weedy sides	0.035	0.050
			f. Cobble bottom and clean sides	0.040	0.050
3. Dragline – excavated or dredged			4. Rock cuts		
a. No vegetation	0.028	0.033	a. Smooth and uniform	0.035	0.040
b. Light brush on banks	0.050	0.060	b. Jagged and uniform	0.040	0.050
5. Channels not maintained, weeds and brush uncut					
a. Dense weeds, high as flow depth				0.080	0.120
b. Clean bottom, brush on sides				0.050	0.080
c. Dense brush, high flood stage				0.100	0.140

Lined Open Channels					
1. Temporary Erosion Protection			2. Permanent Erosion Protection		
a. Soil reinforcing mat	0.025	0.036	a. Concrete	0.013	0.015
b. Excelsior blanket	0.035	0.066	b. Asphalt	0.016	0.018
c. Jute net	0.022	0.028	c. Compacted gravel	0.033	0.044
			d. Plain riprap	0.035	0.045
			e. Grouted riprap	0.030	0.040

Highway channels & Swales with maintained Vegetation (Values are for velocities of 6 and 2ft/s)						
	Bermuda, Kentucky bluegrass and Buffalograss		Good stand, any grass		Fair stand, any grass	
			Normal	Maximum		
	Normal	Maximum				
1. Depth of flow up to 0.7 foot						
i. Mowed to 2 in	0.045	0.070				
ii. Length 4 to 6 in	0.050	0.090				
iii. Length about 12 in			0.090	0.180	0.080	0.140
iv. Length about 24 in			0.150	0.300	0.130	0.250
2. Depth of flow 0.7 –1.5 feet						
i. Mowed to 2 in	0.035	0.050				
ii. Length 4 to 6 in	0.040	0.060				
iii. Length about 12 in			0.070	0.120	0.060	0.100
iv. Length about 24 in			0.100	0.200	0.090	0.170
3. Grass linings (general)					0.030	0.060

6.7.4.2 DESIGN PROCEDURE FOR FLEXIBLE LININGS

The normal treatment for reducing the chance of future erosion is to use stone lining (riprap) in the erodible ditch section. The general design procedure involves the following steps:

1. The depth of flow (d) and velocity (V) in the channel corresponding to the known peak discharge are computed with the Manning's roughness coefficients for riprap from Figure 6-16 as illustrated in Example 4, Attachment B.

2. The required D_{50} riprap size is obtained from the equation:

$$D_{50} = C\frac{0.001V_a^{\ 3}}{\left(d_{avg}^{\ 0.5}\ K_1^{\ 1.5}\right)} \qquad (6.20)$$

where

D_{50} = the size of stone (ft) such that 50% of all stones in the riprap are smaller than D_{50}

C = a correction factor for the specific gravity of stone and the stability factor to be applied

V_a = Average velocity (ft/s)

d_{avg} = Average depth of flow (ft)

K_1 = Bank angle correction factor

C can be calculated by using the equation:

$$C = 1.61\left[\frac{SF}{S_s - 1}\right]^{1.5} \qquad (6.21)$$

where

SF = Stability factor to be applied, Table 1, HEC-11 (Default = 1.2)

S_s = Specific gravity of stones in the riprap (Default = 2.65)

K_1 can be calculated form the equation:

$$K_1 = (1 - 2.27\sin^2\theta)^{1/2} \qquad (6.22)$$

where

θ = Angle of the bank with the horizontal axis

The value of C may alternatively be obtained from Chart 2, HEC-11. The remaining expression of equation 6.21 can be obtained from Chart 1, HEC-11.

The corresponding weight W_{50} (lbs.) for D_{50} (ft) stone is given by:

$$W_{50} = 32.67D_{50}^3 S_s \qquad (6.23)$$

However, for rock riprap, Manning's roughness coefficient varies with the mean size D_{50} as given in the equation:

$$n = 0.0395D_{50}^{1/6} \qquad (6.24)$$

For a more accurate solution, the value of n corresponding to D_{50} should be obtained from equation 6.25 and used in Steps 1 and 2 for a refined value of D_{50}.

3. With the required mean stone size (D_{50}), the type of riprap can be specified from the gradation data in DelDOT's *Standard Specifications*. Also refer to the *Standard Construction Details*.

4. In addition, the following criteria shall be taken into considered when specifying the layer thickness:

   a. The layer thickness should not be less than the spherical diameter of D_{100} stone or less than 1.5 times the spherical diameter of D_{50} stone whichever is the greater thickness. D_{100} denotes that 100% of the riprap stones are smaller than this size.

   b. The layer thickness should not be less than 12 inches for practical placement.

   c. The thickness determined by the above criteria shall be increased by 50% when the riprap is placed under water due to uncertainties associated with this type of placement.

   d. An increase in thickness of 6 to 12 inches accompanied by an appropriate increase in stone sizes should be provided where riprap revetment will be subjected to floating debris, ice or waves.

To prevent the riprap from settling, a geotextile filter fabric is placed between the riprap and the underlying soil. This prevents migration of fine particles through voids in the riprap, distributes the weight of the riprap units to provide a more uniform settlement, and permits relief of hydrostatic pressures within the soil.

Please refer to the *Standard Construction Details* for more information on the layers of the ditch section.

Geotextile Filter Fabrics. The design of filter fabric is based on specifications conforming to its functional requirements. DelDOT's *Standard Specifications* specify filter fabrics for use under riprap for erosion protection. For good performance, a properly selected filter fabric should be installed as recommended by the manufacturer, the *Standard Specifications* and the *Standard Construction Details*.

Figure 6-17
Channel Lining Design Procedure

Step 1	Hydrologic computations: includes determining the drainage area, rainfall design year, rainfall intensity and runoff coefficient	Step 7	Determine hydraulic resistance using HEC-11 charts with known R, A and d_{max} or calculate
Step 2	Design flow - temporary and/or permanent	Step 8	Determine velocity using HEC-11 charts with known R and S_o
Step 3	Soil erodibilty; retardance	Step 9	Determine the allowable flow rate, Q
Step 4	Channel description including geometry; includes bottom width, side slopes, flow line slope, top width and area	Step 10	If $Q_{allowable} >> Q_{design}$, the channel is over designed If $Q_{allowable} << Q_{design}$, the channel is under designed See Figure 6-18 for a useful computation chart.
Step 5	Trial lining type	Step 11	Based on above results, if necessary, perform Steps 3 thru 10 with new parameters
Step 6	Determine permissible maximum depth of flow, d_{max}, (using HEC-11 Charts)	Step 12	If a temporary lining is to be used until establishment of permanent cover, it may be prudent to repeat steps 6 thru 9 using temporary lining characteristics

Figure 6-18
Channel Lining Design Computation Chart

Lining Type	d_{max}	B	d_{max}/B	A/Bd	A	R/d	V	$Q = AV$	T	Remarks

6.8 PAVEMENT DRAINAGE AND STORM DRAINS

In certain situations, especially for urban highways, it may not be practical to construct roadside ditches (open channels) to collect pavement runoff and convey it to an outfall point. The options are to provide curbs or curb and gutter combinations along the shoulder edges for containing the runoff and channelizing the flow into inlets through which the pavement runoff is removed.

Gutter flow along the pavement is confined to a width and depth that will not obstruct nor cause a hazard to traffic. The ponding width will have a corresponding depth. The gutter flow capacity depends upon the contributing drainage area, gutter width, cross slope, longitudinal slope, and roughness.

Usually the stormwater flowing along median swales or in open areas of parking lots is also removed through inlet structures. A storm drain receives the runoff from inlets, conveys it through a closed conduit, and discharges at a suitable outfall. This is referred to as a closed drainage system as opposed to open channels/roadside ditches. The design of storm drains involving curbed drainage and inlets will be generally described in this section.

6.8.1 REFERENCES

PUBLICATIONS

- *Design Charts For Open-Channel Flow*, Hydraulic Design Series No. 3 (HDS-3), FHWA 1973 (Reprint 1980)
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4 (HDS-4), FHWA, 1983 and 2001
- *Drainage of Highway Pavements*, Hydraulic Engineering Circular 12, (HEC-12), FHWA, 1984
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular 22, (HEC-22), FHWA, 2001
- DelDOT *Design Guidance Memorandum Number1-20R.*

TABLES, CHARTS AND FIGURES

Access to the following references will be needed to perform this analysis, most of which are in HEC-12 or HEC-22. HEC-12 has been superseded by HEC-22 but still may be useful in design.

- Manning's Roughness Coefficients *n* for Pavements and Gutters
  - Table 4-3, HEC-22
- Allowable Water Spread
  - Figure 6-2
- Inlet and Gutter Sections
  - Figure 6-19
- Nomograph for Velocity in Triangular Gutter Sections
  - Chart 4B, HEC-22
- Flow in Triangular Gutter Sections
  - Chart 29, HDS-3
  - Chart 1B, HEC-22
- Frontal Flow to Total Gutter Flow Ratio
  - Chart 2B, HEC-22
- Flow in Composite Gutter Sections
  - Chart 5, HEC-12
- Curb-Opening and Slotted Drain Inlet Length for Total Interception
  - Figure 35b, HDS-4
  - Chart 9, HEC-12
- Curb-Opening and Slotted Drain Inlet Interception Efficiency
  - Figure 36, HDS-4
  - Chart 10, HEC-12
  - Chart 8B, HEC-22
- Grate Inlet Capacity in Sump Conditions
  - Chart 11, HEC-12
  - Chart 9B, HEC-22
- Depressed Curb Opening Inlet Capacity in Sump Conditions
  - Chart 12, HEC-12
  - Chart 10B, HEC-22
- Orifice Flow in Depressed Curb-Opening Inlet
  - Chart 14, HEC-12
  - Chart 12B, HEC-22
- Slotted Drain Inlet Capacity in Sump Locations
  - Chart 15, HEC-12

  - Chart 13B, HEC-22
- Ratio of Frontal Flow to Total Flow in Trapezoidal Channel
  - Chart 17, HEC-12
  - Chart 15B, HEC-22
- Inlet Clogging Factor of Safety
  - Figure 6-3
- Gutter and Inlet Design using HEC-22
  - Figure 6-20
- Frontal Flow Interception factor, R_f
  - Figure 6-21
- Side Flow Interception Factor, R_s
  - Figure 6-22
- Min. Pipe Slope for Full Flow at 3 ft/s
  - Figure 6-24
  - Table 7-7, HEC-22
- Circular Pipe Conveyance Factor (*K*)
  - Figure 6-25
- Wall Thickness and Approximate Weight of Circular Concrete Pipe
  - Figure 6-26
- Manning's Roughness Coefficients (*n*) for Pipe
  - Figure 6-27
- Inlet Spacing Computation Form
  - Example 5, Attachment B

COMPUTER SOFTWARE

- HYDRAIN using the HYDRA module
- InRoads Storm & Sanitary
- HY22 Urban Drainage Design Programs

6.8.2 PAVEMENT DRAINAGE

The primary source for this section is HEC-22. The flow capacity of the curb and gutter section of a roadway pavement depends upon:

1. Manning's roughness coefficient of the surface,
2. Longitudinal slope of the roadway,
3. Cross slope,
4. Allowable spread, and
5. Inlet spacing.

6.8.2.1 MANNING'S ROUGHNESS COEFFICIENT

Manning's roughness coefficient varies slightly for different types of pavement and gutter surfaces. For simplicity, use n = 0.016 for all surfaces of pavements and gutters.

6.8.2.2 LONGITUDINAL SLOPE

The longitudinal roadway slope S_L (ft/ft) is initially established by following the ground profile. The roadway profile is adjusted to balance the cut and fill sections to maximize use of existing soil. Excavation required for drainage also enters into this decision. Flat slopes create flooding problems in a curbed drainage system. Therefore, a minimum gradient of 0.30% within 50 ft of the PI station in the sag or crest of vertical curves should be used in design. Warping the shoulder by gradually changing its cross slope shall be considered when the profile grade falls below 0.3% on vertical curves. Also for a curbed drainage system, the roadway profile shall have a minimum gradient of 0.3% on tangent sections.

In the design of roadways, equal tangent parabolic vertical curves are commonly used for setting the profile. Several basic equations apply when designing drainage for this type of profile. To determine an elevation at a desired point on the vertical curve use the equation:

$$E_x = E_a + \frac{g_2 - g_1}{2L} x^2 + g_1 x \qquad (6.26)$$

where

E_x = Elevation of a point at a distance x from the PVC (ft)

E_a = Elevation PVC (ft)

g_1 = Entering grade (%)

g_2 = Exiting grade (%)

x = Distance measured from PVC (ft) divided by 100

L = Length of vertical curve (ft) divided by 100

Note: g_1 and g_2 are positive for ascending grade and negative for descending grade

To determine the turning point on a vertical curve (highest point for a crest curve or lowest point for a sag curve) use the equation:

$$x_t = \frac{g_1 L}{g_{1-} g_2} \qquad (6.27)$$

where

x_t = Location of the turning point from PVC (ft)

Other notations are as previously defined. The elevation of the turning point can be obtained from equation 6.30 by setting x equal to x_t.

The slope of a point (S) on a vertical curve is determined using the equation:

$$S = \frac{g_2 - g_1}{L} x + g_1 \qquad (6.28)$$

S is the slope in (%) at a point on the curve that is a distance x from the PVC. The other notations are as previously defined.

6.8.2.3 CROSS SLOPE

Adequate cross slope on roadways is essential for rapid movement of surface water to reduce sheet flow and potential hydroplaning. On a normal section, the cross slope on a traffic lane is 2% and the shoulder cross slope is 4%. Where these minimum cross slopes are not provided (as in transitions between superelevated and normal sections, or on cross streets and near curb ramps) all efforts shall be made to minimize the sheet flow by providing additional drainage appurtenances and/or with suitable geometric adjustments.

6.8.2.4 ALLOWABLE WATER SPREAD

It is imperative that the computed width of water spread does not exceed the allowable water spread as stipulated in Figure 6-2. The drainage report shall include a comparison of the two values to ensure compliance with the criteria.

6.8.2.5 CURB AND GUTTER FLOW

There are two types of gutter sections used to control drainage along curbed roadways. One section has a uniform cross slope and the other has a composite cross slope. Their design is considered a shallow open channel. Refer to Figure 6-19, DelDOT's *Standard Construction Details*, and Figure 4-1, HEC-22, for a description of the shape, details, dimensions and terminology used in designing gutter and inlet drainage.

6.8.2.5.1 GUTTER WITH UNIFORM CROSS SLOPE

As illustrated in Figure 6-19, triangular open channels with uniform cross slope are characterized by:

- T = Width of water spread (ft)
- d = Depth of flow at curb (ft)
- S_x = Cross slope (ft/ft)
- S_L = Longitudinal roadway slope (ft/ft)
- n = Manning's roughness coefficient
- A = Cross-sectional area of flow (ft^2)

Manning's equation for flow rate in an open channel has to be modified for use with gutter sections because the depth of flow is very shallow and wide. The governing equations for average velocity (V, ft/s) and flow (Q, ft^3/s) in a triangular gutter section are:

$$V = \frac{1.12}{n} S_L^{0.5} S_x^{0.67} T^{0.67} \qquad (6.29)$$

$$Q = \frac{0.56}{n} S_L^{0.5} S_x^{1.67} T^{2.67} \qquad (6.30)$$

Also for a uniform triangular gutter section,

$$d = TS_x \qquad (6.31)$$

$$A = 0.5T^2 S_x = 0.5 \, d^2 / S_x \qquad (6.32)$$

Therefore, the flow rate is given by:

$$Q = \frac{0.56}{n} S_L^{0.5} \frac{d^{2.67}}{S_x} \qquad (6.33)$$

See Charts 1B and 4B, HEC-22, for the nomographs to solve these equations. From Chart 4B, HEC-22, the flow velocity (V) can be determined when the width of water spread (T) is given (and vice versa) with other known parameters.

The width of water spread (T) varies between inlets in a curbed drainage system and so does the velocity of flow (V). If T_1 is the spread at the trailing end of an inlet, T_2 is the spread at the approach end of the downstream inlet, and T_a represents the average width of spread between the two inlets on the triangular gutter section, then T_a can be obtained from Table 4-4, HEC-22.

T_a is useful in estimating the travel time of surface water flow on a triangular gutter section with drainage inlets. The velocity V_a corresponding to T_a is the average velocity that can be obtained from Chart 4B, HEC-22. Once V_a is known, the average travel time is determined by dividing the length of travel (distance between the two inlets) by V_a.

Chart 1B, HEC-22, provides the solution for the rate of flow Q in a triangular gutter section. When Q is known, T can be found and vice versa, provided n, S, and S_x are also known. This chart can also be used for solving the flow rate of V-shaped gutter sections by combining the two cross slopes of the V-sections into a single cross slope as illustrated on the chart.

6.8.2.5.2 GUTTER WITH COMPOSITE CROSS SLOPE

Composite cross slope results when a break exists in the cross slope of the gutter section. See Figure 6-19 for typical gutter sections and nomenclature. A composite cross slope is normally associated with the use of Integral P.C.C. Curb & Gutter, Type 1. The hydraulic efficiency of a gutter section with composite cross slope is significantly higher than one with uniform cross slope. Therefore, the use of Integral P.C.C. Curb & Gutter, Type 1, is preferred in the design of a curbed drainage system.

Composite curb and gutter sections are characterized by the following terms that are also used in the gutter flow equations:

- T = Width of water spread (ft)
- T_s = Width of section with normal cross slope, equal to the total spread minus the gutter pan width
- S_x = Cross slope (ft/ft) of adjacent traveled way or shoulder
- d' = Depth of flow at cross section break
- W = Gutter pan width of greater slope
- S_w = Slope of gutter section
- d = Depth of flow at curb (ft)
- S_L = Longitudinal roadway slope (ft/ft)
- n = Manning's roughness coefficient
- A = Cross-sectional area of flow (ft^2)

Figure 6-19
Inlet and Gutter Sections

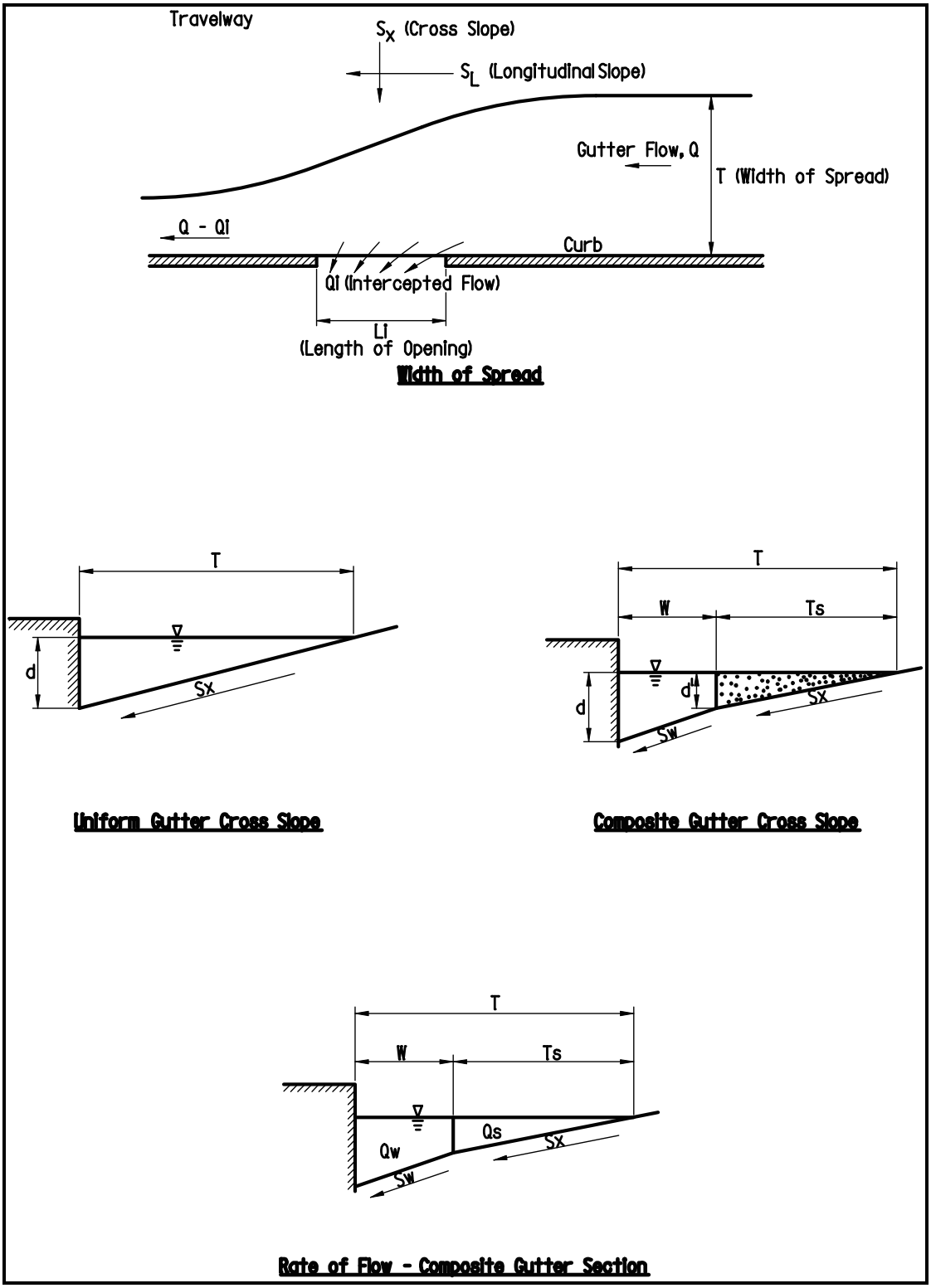


Figure 6-20
Gutter and Inlet Design using HEC-22

Capacity (Gutter and Inlet)			
Step 1	Determine design parameters		
L = Length of design strip A = Area of contribution C = Runoff coefficient of contributing area t_c = Time of concentration I = Design year rainfall intensity Q = Peak flow using $Q = CIA$[1]		S_x = Slope of adjacent shoulder or pavement S_w = Slope of gutter section n = Roughness coefficient (0.016 for concrete) T = Allowable spread width of flow W = Proposed gutter width L_{E1} = Length of grate	
Step 2	Calculate the following ratios needed for using charts: $Z = 1/S_x$, W/T and S_w/S_x	Step 9	Determine E (the total efficiency of the inlet) using: $E = (R_f E_o) + R_s(1-E_o)$
Step 3	Find Qn, the flow in the shoulder or pavement (area outside of W) from Chart 1B	Step 10	Find Q_i (the inlet interception flow) using: $Qi = EQ$
Step 4	Find gutter velocity V from Chart 4B using S_w/S_x ratio	Step 11	Determine Q_b (the inlet flow bypass) from $Q_b = Q - Q_i$
Step 5	Find E_o from Chart 2B using factors S_w/S_x and W/T	Step 12	Using the design flow values, determine the depth of water at the curb line for any curb overflow[2] and actual spread.
Step 6	Find Q_w (the gutter flow rate over the grate) using: $E_o = Q_w/Q$	Step 13	If Step 12 results do not meet the criteria, than repeat the procedure until the correct parameters and results are met.
Step 7	Find R_f (frontal flow interception rate) from Figure 6-21 or Chart 5B.	Step 14	Add the inlet flow bypass to the next Q for the next inlet downstream.
Step 8	Find R_s (side flow interception factor) from Figure 6-22 or Chart 6B.	Step 15	Determine the location of the first inlet by iteratively finding the area generating a flow equal to Q_i.

1. The flow to the all inlets is the sum of flow from all contributing areas that will reach the inlet being analyzed.
2. Don't overflow the curb.
3. See Example 5 for complete form and instructions, also HEC-22 Figure 4-19.3.

In the hydraulic analysis of a composite section, the area is broken into three triangular sections. The modified Manning's equation is applied to each section. The rate of flow in a gutter section with a composite cross slope is given by the equation:

$$Q = \frac{0.56}{n} S_L^{0.5} \left(\frac{d^{2.67}}{S_w} + \left[\frac{d'^{2.67}}{S_x} - \frac{d'^{2.67}}{S_w} \right] \right) \quad (6.33)$$

where

$$d = T_s S_x + W S_w \quad (6.35)$$

$$d' = T_s S_x \quad (6.36)$$

$$A = 0.5[T_s^2 S_x + 2 T_s W S_x + W^2 S_w] \quad (6.37)$$

Charts 1B and 2B, HEC-22, can be used in conjunction with each other to obtain Q for a composite gutter section. The notations are defined in the respective charts.

Chart 5, HEC-12, provides the hydraulic solution of composite gutter sections. When Q, n, S_L, W, S_x and S_w are known, T can be found from this chart. Likewise, Q can be determined when T is given.

6.8.2.5.3 GUTTER FLOW DESIGN TABLES

The hydraulic analysis of gutter sections for both uniform and composite cross slopes can be performed in accordance with the equations and charts referred to in this section as well as available software. For performing gutter flow design by hand, drainage tables for pavements of varying geometric characteristics that are usually incorporated in road design are provided on DelDOT's web site; the variables in these tables are the width of water spread (T), the flow rate (Q), and the average velocity (V).

6.8.2.6 DRAINAGE INLETS

Drainage inlets for intercepting surface runoff should be properly designed with respect to their location and capacity so that the drainage system functions efficiently. They are used on roadways, median swales, roadside ditches and parking areas.

6.8.2.6.1 INLET TYPES

There are a variety of inlets available for use in drainage. Many have a unique shape and application. The Department's basic inlets and grate types are shown in the *Standard Construction Details*. These details have a series of inlet top unit designs (drainage inlet assembly) to fit the available types of curbs and for swale locations.

Inlet capacity is improved by combining a grate inlet with a curb opening, particularly at sag points, and is relatively free from clogging by debris. This type of inlet is commonly used on projects.

Curb-inlets have no grate opening and are used particularly on narrow paved medians and curbed sections were the driver might shy away from a grate. They work best where grades are flatter than 3 percent. The inlet is depressed 2 inches below the gutter flow line with transitions at both ends. It is best to use Integral P.C.C. Curb and Gutter, Type 3, with curb-inlets since it only has a one-foot gutter pan. The minimum length of these inlets is 5 ft; the total length shall be specified in 5 ft multiples.

6.8.2.6.2 INLET GRATES

The types of inlet grates used on projects are shown in DelDOT's *Standard Construction Details*. All the grates are 20 in by 36 in. A description of the grates follows.

Type 1 grate has an opening area of 320 in^2, approximately 44% of the total area. The rounded bars intercept flow more efficiently. It is used adjacent to curb with or without integral gutter where bicycle traffic can be anticipated.

Type 2 grate has an opening area of 370 in^2, approximately 51% of the total area. This grate is used adjacent to a curb in controlled access highways or in median swales where bicycle traffic is restricted.

Type 3 grate has an opening area of 295 in^2, approximately 41% of the total area. This type of grate is used in open parking areas, median swales, and along roadsides where bicycle traffic can be expected. These grates are intended to intercept the surface runoff in sump conditions and shall not be used beside curbs.

Type 4 vane grate has an opening area of 215 in^2, approximately 30% of the total area. This type of grate has a higher hydraulic capacity and lower weight than the other types. It may be used where bicycle traffic can be expected. It is not recommended for use in sump locations.

6.8.2.6.3 HYDRAULIC CHARACTERISITICS OF INLETS

In addition to size and shape, the interception capacity and efficiency of the inlet depend upon its location. The intended hydraulic function is also a selection and design consideration.

The hydraulic analysis of inlets is described for their three most common uses. These are to intercept flow:

- Along a curb on a continuous grade,
- Along a curb in a sag or surface runoff in a sump location, and
- In a swale (ditch).

6.8.2.6.4 INLET INTERCEPTION ON CONTINUOUS GRADE

HEC-22 is the primary resource to be used for designing inlet interception on a continuous grade. The following is a brief discussion of this procedure. The chart references throughout Sections 6.8.2.6.2 and 6.8.2.6.3 are found in HEC-22. For design purposes, the capacity of a combination inlet is analyzed based on the grate efficiency and the curb opening is considered a relief for clogging conditions.

The three elements of runoff (Figure 6-19) associated with the inlet capacity on a continuous grade are:

- The rate of flow of stormwater approaching the inlet (the total gutter flow, Q).
- The rate of flow intercepted by the inlet (Q_i).
- The carryover or bypass rate of flow (Q_b).

Therefore,

$$Q_b = Q - Q_i \qquad (6.38)$$

The efficiency (E) of the inlet is:

$$E = \frac{Q_i}{Q} \qquad (6.39)$$

An inlet on continuous grade can be a grate inlet, a curb inlet or a slotted drain inlet.

Grate Inlet The total gutter flow Q consists of two components:

$$Q = Q_w + Q_s \qquad (6.40)$$

where

Q_w = Frontal flow over width W of the grate, and

Q_s = Side flow.

The ratio of frontal flow to total gutter flow (E_o) is defined by the equation:

$$E_o = \frac{Q_w}{Q} = 1 - (1 - W/T)^{2.67} \qquad (6.41)$$

Where:

Q = Total gutter flow (ft^3/s)

Q_w = Flow over width W of grate (ft^3/s)

W = Width of depressed gutter or grate (ft)

T = Total spread of water in the gutter (ft)

Solutions for E_o are presented in Chart 2B for various $\frac{W}{T}$ and $\frac{S_w}{S_x}$ ratios.

Let: $R_f = \dfrac{Q_{wi}}{Q_w}$ (6.42)

where

R_f = Frontal flow Interception factor,

Q_{wi} = Frontal flow Intercepted by the inlet.

Also,

$$R_s = \frac{Q_{si}}{Q_s} \quad (6.43)$$

where

R_s = Side flow interception factor,

Q_s = Side flow intercepted by the inlet.

The efficiency E is given by,

$$E = R_f E_o + R_s (1 - E_o) \quad (6.44)$$

Therefore,

$$Q_i = EQ = Q\,[\,R_f\,E_o + R_s\,(1 - E_o)] \quad (6.45)$$

The value of E_o from Chart 2B is used in the above equations. R_f and R_s depend upon the approach velocity V that corresponds to Q for the total gutter flow. Whether the flow takes place on either a uniform cross slope or a composite cross slope, V can be obtained by solving the equations or from the nomographs. Having known V, the values of R_f and R_s can be obtained from Figures 6-21 (for the Type 1 grate only) and 6-22, or Charts 5B and 6B, respectively.

Figure 6-21
Type 1 Grate Frontal Flow Interception Factor, R_f*

V	< 6	7	8	9	10	11	12	13	14	15
R_f	1.0	0.92	0.83	0.73	0.63	0.56	0.47	0.38	0.28	0.19

*Adapted from Chart 5B, HEC-22

Figure 6-22
Side Flow Interception Factor*, R_s

V	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
S_x	R_s														
0.02	0.63	0.32	0.19	0.12	0.08	0.06	0.05	0.04	0.03	0.03	0.02	0.20	0.02	0.01	0.01
0.03	0.71	0.42	0.26	0.17	0.12	0.09	0.07	0.06	0.05	0.04	0.03	0.03	0.02	0.02	0.02
0.04	0.77	0.49	0.32	0.22	0.16	0.12	0.09	0.07	0.06	0.05	0.04	0.04	0.03	0.03	0.02
0.05	0.81	0.55	0.37	0.26	0.19	0.14	0.11	0.09	0.07	0.06	0.05	0.05	0.04	0.03	0.03
0.06	0.83	0.59	0.41	0.29	0.22	0.17	0.13	0.11	0.09	0.07	0.06	0.05	0.05	0.04	0.04

*Using equation $R_s = \dfrac{1}{\left[1 + (0.15)(V^{1.8})/S_x L^{2.3}\right]}$

Curb-Opening Inlet. For uniform cross slope, the length of curb opening for total interception of gutter flow is given by the equation:

$$L_T = 0.6\, Q^{0.42}\, S_L^{\;0.3} \left(\frac{1}{n\, S_x} \right)^{0.6} \qquad (6.46)$$

where

L_T = Length of curb opening in feet for total interception of gutter flow (100% efficiency of interception).

Other notations are as defined previously.

The efficiency E for inlets shorter than L_T is expressed by the equation:

$$E = 1 - \left(1 - L/\, L_T\right)^{1.8} \qquad (6.47)$$

This equation is also valid for depressed curb-opening inlets.

Charts 7B and 8B provide graphical solutions of these equations.

However, curb-opening inlets must be depressed to significantly increase their interception of gutter flow. This results in a composite cross slope as illustrated on Chart 7B. The equation is:

$$S_e = S_x + S'_W\, E_o \qquad (6.48)$$

Where,

S_e = Equivalent cross slope for depressed opening (ft/ft)

S_x = Pavement cross slope (ft/ft)

S'_w = Cross slope of the gutter measured from the edge of pavement (ft/ft)

E_o = Ratio of flow over the depressed width to the flow over the total gutter section (Q_w/Q)

As shown in Chart 7B, W is the depressed width or gutter width (ft) and a equals the gutter depression (in).

Therefore,

$$S'_W = \frac{a}{(12W)} \qquad (6.49)$$

Thus for depressed curb-opening inlets:

$$L_T = 0.6\, Q^{0.42}\, S_L^{\;0.3} \left(\frac{1}{n\, S_e} \right)^{0.6} \qquad (6.50)$$

Slotted Drain Inlet The equations for solving for L_T and E and Chart 7B for curb-opening inlet along with the nomograph in Chart 5B for interception efficiency are applicable to slotted drain inlets on a continuous grade.

6.8.2.6.5 INLET INTERCEPTION IN SAG LOCATIONS

Inlets on sag vertical curves of roadway pavement, in open parking areas, and on median swales, operate in a sump condition. As the surface runoff approaches from some or all sides of the inlet, the interception takes place as if the flow occurs over a weir or an orifice depending on the depth of flow. The inlet operates as a weir with low head and as an orifice with relatively high head.

Note that when using Chart 9B the following applies:

- Inlet Type 1─Opening ratio = 0.44
- Inlet Type 2─Opening ratio = 0.51
- Inlet Type 3─Opening ratio = 0.41
- Inlet Type 4─Opening ratio = 0.30

Inlet grates in sag locations are analyzed for two conditions:

Condition 1 - Inlet Grate Operating as a Weir

The flow rate intercepted by a grate inlet is

$$Q_i = C_w\, P\, d^{1.5} \qquad (6.51)$$

where

Q_i = Flow intercepted by the inlet (ft^3/s)

C_w = Weir coefficient = 3

P = Perimeter of the sides of inlet approached by runoff (ft)

d = Average depth of water adjacent to the inlet (ft) (see Figure 4-17, HEC-22)

Chart 9B has the graphical solution to this equation. As indicated on the chart, if the flow is hindered on one side of the inlet due to the curb, then that side has to be omitted in calculating *P*. For practical purposes *W* and *L* shall be taken as the outside dimensions of the grate inlet.

Generally, the Type 1 grate operates as a weir in a curbed drainage design. The permissible depth of flow at the curb is limited to the height of curb and the allowable spread. However, when used in parking areas and swales, the Type 1 grate may operate as an orifice because a greater depth of flow is likely to be accommodated.

Condition 2 - Inlet Grate Operating as an Orifice

This condition exists when the depth of water exceeds 8 in. The flow rate intercepted by the grate inlet is given by:

$$Q_i = C_o A (2gd)^{0.5} \qquad (6.52)$$

Where,

C_o = Orifice coefficient = 0.67

A = Clear area of grate opening (ft^2)

g = Acceleration due to gravity (32.2 ft/s^2),

d = Average depth of water adjacent to the inlet (ft) (see Figure 4-17, HEC-22)

Chart 9B presents the solution for equation 6.52 using the given inlet opening ratios.

When the depth of water is in transition between weir and orifice flow, the perimeter line on the lower half of the figure should be connected by a smooth curve with the corresponding clear area line of the upper half for Q_i in transition.

Depressed Curb-Opening Inlet in Sag Location

Case 1 - Depressed Curb-Opening Inlet Operating as a Weir

The inlet interception Q_i (ft^3/s) is given by:

$$Q_i = 2.3 (L + 1.8W) d^{1.5} \qquad (6.53)$$

where

L = Length of depressed curb opening (ft)

W = Lateral width of depression (ft)

d = Depth of water (ft) at curb measured from normal cross slope

In this case,

$$d = T S_x \qquad (6.54)$$

where

T = Water spread

S_x = Slope of adjacent pavement

For the depressed inlet to operate as a weir:

$$d \le [\, h + \frac{a}{12} \,] \qquad (6.55)$$

where

h = Height of curb-opening inlet (ft)

a = Depth of depression (in)

Case 2 - Depressed Curb-Opening Inlet Operating as an Orifice

The inlet interception Q_i (ft^3/s) is given by:

$$Q_i = 0.67\, h\, L\, (2\, g d_o)^{0.5}$$

$$= 0.67\, A \left[2g \left(d_i - \frac{h}{2} \right) \right]^{0.5} \qquad (6.56)$$

where

d_o = Effective head on the center of orifice throat (ft)

A = Clear area of the opening (ft^2)

h = Height of depressed curb-opening inlet (ft)

d_i = Height of depressed curb-opening orifice (ft), expressed as:

$$d_i = TS_x + \frac{a}{12} \qquad (6.57)$$

a = Depth of depression (in)

For the depressed curb-opening inlet to operate as an orifice, the depth of flow should be greater than 1.4 *h*.

Chart 10B gives the graphical solution of equations for depressed curb-opening inlets. Chart 12B gives the solution for inclined and vertical orifice throats.

Slotted Drain Inlet

The slotted drain inlet in sump condition operates as a weir when the depth of flow is up to 0.2 ft and as an orifice when the depth of water exceeds 0.4 ft. Chart 13B provides the solution both for weir and orifice conditions with dashed lines for transition. Note the multiplying factor involving the width of slot at the top of the figure.

6.8.2.6.6 INLET IN OPEN CHANNEL

Grate inlets are installed in median swales or roadside ditches to intercept the flow of these open channels. They can be either on a continuous grade or in a sump depending on the profile of the channel. For the sump condition, the methodology outlined in Section 6.8.2.6.5 applies. For installations on a continuous grade, the analysis for interception by grate inlets in Section 6.8.2.6.4 is valid, except for E_o that was defined by equation 6.41.

E_o for trapezoidal channels can be obtained from Chart 15B. The intercepted flow Q_i is determined by using equation 6.45.

6.8.2.6.7 FACTOR OF SAFETY FOR CLOGGING OF GRATE INLETS

Because grate inlets may become clogged with debris, the theoretical interception capacity should be properly reduced in drainage design.

Generally a grate inlet on continuous grade is placed beside a curb that has an opening. This opening, disregarded in the interception analysis, should alleviate the effect of clogging that seldom occurs on a continuous grade. Therefore, a factor of safety to reduce the theoretical interception capacity of grates on continuous grade may be deemed unnecessary.

However, in sump conditions where clogging is anticipated, the factors of safety to be applied are:

- for curb on one side use 1.5 (S_{f1})
- for no curb, i.e., the interception takes place from all four sides, use 2.0 (S_{f2}).

For weir flow, the clogging shall be assumed to occur lengthwise, so that a reduced width of grating is taken into consideration to account for the factor of safety. For example, consider a standard grate that operates as a weir. If it is placed alongside a curb, then the perimeter to be considered for computing the interception with factor of safety is given by:

$$P = 2\left(\frac{W}{S_{f1}} + L\right)/12 \qquad (6.58)$$

$$P = 2\left(\frac{20}{1.5} + 36\right)/12 = 5.22\ ft$$

If the curb does not exist, then the interception would occur from all four sides. Therefore,

$$P = 2\left(\frac{W}{S_{f2}} + L\right)/12 \qquad (6.59)$$

$$P = 2\left(\frac{20}{2} + 36\right)/12 = 7.67\ ft$$

When a grate inlet in a sump condition operates as an orifice, then its clear open area (A) should be divided by the factor of safety specified in the foregoing to compute the actual interception capacity.

6.8.2.6.8 INLET LOCATIONS

6.8.2.6.8.1 DESIGN CRITERIA

1. Drainage inlets shall be designed for the runoff frequency and spread as defined in DelDOT's design criteria. The design involves determining the spacing of inlets based on their interception capacity.

2. Aside from inlet locations as required by their hydraulic design, additional inlets shall be provided in strategic areas to avoid the

concentration of sheet flow by intercepting the runoff in advance. Some of those strategic locations are:

- Upstream of median breaks, entrance and exit ramp gores, curb ramps, crosswalks and street intersections.
- Upstream and downstream of bridge approaches.
- Upstream of superelevation tangent runout and cross slope reversal.
- End of channels in cut sections.
- Behind curb and sidewalk in low areas.
- At the low point of a sag vertical curve.

The inlet(s) at the low point should be designed to keep the water spread within the allowed limit. The preferred method is to design flanking inlets to act in relief of the low point inlet if it gets clogged or the water spread exceeds the limit due to a less infrequent storm. The flanking inlets shall be located where the gutter elevation is approximately 0.2 feet higher than the low point or where allowed spread would be exceeded if a sump drain clogged. The method described in HEC–22 is used to design flanking inlets.

Equations for elevation and low point on a sag vertical curve are included in Section 6.8.2.2. The distance of each flanking inlet from the low point x (feet) is given by:

$$x = (4000L/A)^{0.5} \qquad (6.60)$$

Where:

L = Length of vertical curve

A = Algebraic difference in approach grades

3. The maximum spacing of inlets shall be 300 ft.

4. The efficiency of inlets on continuous grade shall be at least 70%. However, inlets provided at strategic locations should intercept 100% of the gutter flow.

5. The factor of safety for potential clogging of inlet grates in sump conditions are specified in Section 6.8.2.6.7. In approved locations, parallel bar grates are preferred in sump conditions.

6. The criteria for using curb-opening inlets are in Section 6.8.2.6. When the roadway profile is steeper than 3%, curb-opening inlets should not be used.

7. Gutter inlets should not be used to intercept runoff from areas adjacent to the roadway, neither within nor outside the right-of-way. The surface runoff from those areas may be more efficiently intercepted in advance, and sometimes disposed of by other means such as roadside ditches.

6.8.2.6.8.2 SPACING OF DRAINAGE INLETS

Inlet spacing begins at a required distance from the crest of roadway subject to previously defined location criteria. The flow rate of stormwater intercepted by the first inlet and the bypass flow rate are computed and its efficiency is checked. If the inlet efficiency is found to be lower than the prescribed minimum, then the inlet location is moved upstream to achieve the required 70% efficiency. Then the computation proceeds to the next inlet where the discharge from its watershed (downstream of the first inlet) is added with the bypass from the first inlet. This total gutter flow rate is used to compute the water spread, which cannot exceed the design criteria, and the efficiency of this inlet. The procedure is repeated as the roadway profile moves into a sag curve where the storm water is ultimately removed from the inlet at the low point, which operates in a sump condition at 100% efficiency. The following equations are used for spacing inlets on grade. It should be noted that these equations are based on flow data obtained using the procedure outlined in Figure 6-20.

Spacing of First Inlet from Crest.

$$L_1 = \frac{Q_{1T} \; x \; 43{,}560}{C_1 \, I_1 \, \overline{W}_1} \qquad (6.61)$$

where

L_1 = Distance of the first inlet from crest (ft)

Q_{1T} = Total gutter flow capacity upstream of the first inlet within the width of permissible water spread that conforms to the required efficiency

C_1 = Weighted average runoff coefficient of the drainage area of the first inlet

I_1 = Rainfall intensity corresponding to the time of concentration for the first inlet (in/hr)

$\overline{W_1}$ = Mean width of the first inlet's drainage area (ft)

Spacing of Subsequent Inlets

$$L_n = \frac{\left[Q_{nT} - Q_{B,n-1}\right] x\, 43{,}560}{C_n\, I_n\, \overline{W}_n} \quad (6.62)$$

where

L_n = Distance of *n*'th inlet from the *(n-1)*'th inlet (ft)

Q_{nT} = The total gutter flow capacity upstream of n'th inlet within the width of permissible water spread that conforms to the required inlet efficiency (ft^3/s)

$Q_{B,n-1}$ = Bypass flow rate from the *(n-1)*'th inlet (ft^3/s)

C_n= Weighted average runoff coefficient of the drainage area of the *n*'th inlet

I_n= Rainfall intensity corresponding to the time of concentration for the *n*'th inlet (in/hr)

$\overline{W_n}$ = Mean width of the *n*'th inlet's drainage area (ft)

For non-uniform drainage areas, the solutions to these equations are obtained by trial and error. The spacing distances are assumed and the mean widths of drainage areas are computed along with other parameters until the equations are satisfied with the assumed spacing of the drainage inlets.

All inlets of the drainage system shall be numbered and the computed results shall be presented in a tabular format. Use a procedure such as described in Example 5, Attachment B.

6.8.3 STORM DRAINS

The hydraulic design of a system is performed after the preliminary layout locating inlets, storm drains, and outfalls. The Rational Formula is used to determine the peak discharge for sizing storm drains. The hydraulic design is a two step process: (1) select a preliminary pipe size based on hydrology and simplified hydraulic computations and (2) compute the hydraulic gradeline (HGL) for the system. The second step refines the preliminary pipe size based on the hydraulic losses in the system.

HEC-22 is the primary resource for the design of storm drains**.** Since this is a trial and error process, the use of approved computer software is recommended. The designer should check that the output is in an approved format for inclusion in the H&H Report. It is particularly important that the results are compared to DelDOT's drainage criteria in the output data where applicable.

Each pipe segment of the storm drain system is designed individually for size and slope from inlet to inlet or junction to junction. The Rational Formula is used to compute the peak discharge at the inlet in the beginning of each segment. From the uppermost end of the storm drain system the design begins and continues downstream for each segment to the outfall point. When all pipe segments are designed, it may be necessary to determine the hydraulic gradeline elevation to check the operation of the system under the design storm frequency by computing head losses in the system starting from the design high water elevation at the outfall and proceeding upstream (backwards) considering every pipe segment up to the beginning inlet. This may be a trial and error procedure that is simplified by using computer software.

The storm drain design process begins with the collection of the following data:

1. Hydrologic data of the watershed including the outfall data.
2. Locations of underground utilities, existing or proposed.
3. Topographic data (including runoff basin divides, streets, developments, and lot lines) of the entire watershed draining into the storm drain system with arrows on the maps denoting all directions of flow from each lot and street. This information should be superimposed on the working drawings of preliminary plans.
4. Preliminary roadway plans for the project including inlet locations, roadway typical sections, profiles, and cross sections.

Choosing inlet locations is nearly complete before the actual storm drain design. The next step is to develop a layout plan for the conveyance of the intercepted runoff through main storm drains and laterals that connect all outlets to a suitable outfall point. This system layout plan shall indicate locations of inlets, manholes and junction boxes; all pipe segments with directions of flow; and existing or proposed underground utilities such as water, gas, cable and sanitary sewer. Then a rough profile gradeline from inlet to inlet of the storm drain system leading to the outfall is established, the pipe segments are designed, potential utility conflicts (if any) are resolved, the hydraulic gradeline is checked, and final adjustments are made for approval of the design.

6.8.3.1 DESIGN CRITERIA

Storm drains — The following general criteria and those in Figure 6-3 shall apply to storm drain design:

1. The storm drain trunk line should not be under a traveled way but in the median, a shoulder area or behind the curb.
2. The minimum velocity should be 3 ft/s flowing full to prevent the deposition of sediments and debris inside the storm drain system. See Figure 6-24.
3. In order to prevent damage to the pipe surface, the maximum velocity at full flow should not exceed 15 ft/s. Slopes flatter than 10% are normally used.
4. To prevent water bubbling out of inlets and manholes, particularly at low points, the hydraulic gradeline shall remain at least 1 ft below the gutter elevation at any inlet or top of manhole.
5. For concrete pipe, Class IV is most commonly used along with Class V for greater load bearing locations, such as where there is minimal cover. Other pipe materials (such as corrugated metal and High Density Polyethylene (HDPE)) must also be considered as per federal regulations. The designer has to ensure that the pipe material can bear site-specific loads, perform well under the site conditions and comply with the project's expected service life criteria.
6. The preferred minimum cover over a storm drain is 3.0 ft. Refer to DelDOT's *Standard Construction Details, Standard Specifications, AASHTO LRFD Bridge Design Specifications,* and manufacturer's recommendations for proper bedding and cover requirements under roadway pavements.
7. A simple initial approach for preliminary design and evaluation to ensure the system will operate as open channel flow is to lay out each pipe segment at its friction slope under full flow condition and use equations 6.64 and 6.65.
8. The layout of pipe junctions at inlets, manholes and junction boxes shall be carefully planned for smooth flow with minimum head loss. Pipes entering or exiting a junction cannot exceed an angle of 45°. The designer shall ensure that the pipes fit into the proposed structure, horizontally and vertically. Refer to the *Standard Construction Details* for minimum clearances to the drainage structure walls.
9. At changes in pipe size, the soffits (top inside surfaces of the two pipes) should be

maintained at the same level by dropping the invert of the larger pipe.

10. The size of any pipe segment shall not decrease in size in the downstream direction even if a change in design such as increasing the slope would permit the use of a smaller sized pipe.
11. For cleanout and maintenance inspection, access to storm drains shall be provided through drainage inlets or manholes within the 300 ft maximum distance.
12. If a storm drain or ditch carrying highway runoff extends beyond the right-of-way, then a permanent easement or additional right-of-way to the outfall shall be acquired for future maintenance.
13. Curved alignment of storm drains following the general alignment of the roadway may be permitted. The manufacturer's recommendations for pipe layout should be followed.
14. The design criteria for strength and hydraulic analysis of pipes shall conform to the provisions of Section 6.9.
15. The designer's initial site evaluation should include the prevailing ground water elevation level for estimating dewatering costs.
16. Pressure flow design should not be considered.

Outfalls — The following general criteria should be considered when determining storm drain outfalls.

1. Compliance with the NPDES.
2. Compliance with the Delaware Erosion and Sediment Control regulations.
3. Coordinated with the locations of any required stormwater quantity and quality management systems.
4. Preferably discharges into a natural drainage course.
5. Ensuring that overland flow occurs before entering any wetlands.
6. The storm drain outfall flow should not scour the downstream reach.
7. The outfall should desirably operate at the design storm flow level under inlet control and as a free (non- submerged) outfall.
8. The design of the system should be refined to avoid adverse hydraulic conditions such as excessive velocity, erosion, or excessive scouring requiring the need for expensive energy dissipaters, or bank overflow protection.
9. Embankment slopes and headwall designs for the outfall shall be in conformance with the *Roadside Design Guide.*

Figure 6-23
General Guidelines for Culvert Outfall Treatment*

Pipe outfall design velocity	Channel treatment required
Max. allowable velocity above those in Figures 6-14 and 6-15 up to 10 ft/s	Sod or stabilization
10 ft/s to 14 ft/s	Dumped riprap for a distance of 15 to 25 feet
Greater than 14 ft/s	Special treatment, such as a plunge pool, stilling basin or energy dissipater

*Note: The outfall design is determined by the stormwater control regulations and design procedure.

Figure 6-24
Minimum Pipe Slope to Ensure a 3.0 ft/s Velocity in a Storm Drain Flowing Full

Inner Pipe Diameter (in)	Full Pipe Flow (ft^3/s)	Minimum Slope (%)				
		n = 0.011	n = 0.012	n = 0.013	n = 0.020	n = 0.024
12	2.4	0.32	0.38	0.44	1.04	1.49
15	3.7	0.24	0.28	0.32	0.78	1.11
18	5.3	0.18	0.22	0.26	0.60	0.87
21	7.2	0.15	0.18	0.21	0.49	0.71
24	9.4	0.12	0.15	0.17	0.41	0.59
27	11.9	0.11	0.13	0.15	0.35	0.51
30	14.7	0.09	0.11	0.13	0.30	0.44
36	21.2	0.07	0.09	0.10	0.24	0.34
42	28.9	0.06	0.07	0.08	0.20	0.28
48	37.7	0.05	0.06	0.07	0.16	0.23
54	47.7	0.04	0.05	0.06	0.14	0.20
60	58.9	0.04	0.04	0.05	0.12	0.17
66	71.3	0.03	0.04	0.05	0.11	0.15
72	84.8	0.03	0.03	0.04	0.09	0.14

6.8.3.2 DESIGNING STORM DRAINS

Open channel design shall be used for the design of a storm drain system. For the rare occasions when pressure flow design is used due to the topographic constraints of the site, the engineering justification and calculations for pressure flow design shall be included in the project's drainage report and must be approved by DelDOT's Assistant Director - Design.

6.8.3.2.1 OPEN CHANNEL DESIGN

Under most conditions a roadway's storm drain system flows under atmospheric pressure and gravity. The various pipe segments are sized assuming that they will flow practically full under design discharge such that the surface of the design flow in the pipe is open to atmospheric pressure. One of the controlling design criteria for pipe design is the minimum slope necessary to maintain 3 ft/s flowing full to reduce silting.

The design depth of flow should be lower than the full depth (diameter) of the pipe. A circular pipe flowing at a depth of 93% of its diameter is able to carry the flow equal to its full depth capacity.

The design method consists of computing the design discharge to be carried by each pipe segment and determining the size and slope of pipe necessary for carrying this discharge. This is accomplished by proceeding step by step from the beginning of a storm drain line downstream to its terminus, which may be a junction point where one trunk line discharges into another line or the outfall point. It is essential that the soffit elevation of the outlet pipe at the point of discharge is at or above the design tailwater elevation at the outfall.

The outfall point is a discharge point into a natural watercourse, excavated channel, retention pond or another storm drain system for disposing of the stormwater collected from the highway. Estimating the design tailwater elevation is based on hydrologic considerations relating to the downstream channel characteristics as well as the ratio of watershed area of the storm drain system (tributary) to the total drainage area of the outfall (main stream). The tailwater elevation can be affected by differences in the design frequencies and peak discharge rates of the watersheds, including downstream structures.

One simple initial approach for preliminary design and evaluation to ensure the system will operate as open channel flow is to lay out

each pipe segment at its friction slope under full flow condition and use equations 6.64 and 6.65; also see Section 6.8.3.2.4. Placing the pipe at its friction slope results in determining the normal depth of uniform flow under constant discharge. Essentially this is the slope at which the friction and gravity forces in the direction of flow are equal but acting in different directions. Hydraulically this means that at normal depth the slope of the pipe, the slopes of the HGL and EGL are numerically equal and parallel to each other. Normal depth is a function of the discharge flow rate, the channel geometry, the channel/pipe slope, and the resistance due to friction. The equations in Section 6.8.3.2.4 use these factors to find the correct pipe size and slope. Figure 6-28 shows pipe flow capacity for various pipe sizes at its friction slope using Manning's equations for *n* equals 0.012. Available software maximizes the hydraulic efficiency of the system based upon the input parameters selected for the best and most economical final design.

The open channel design of a storm drain system can be performed following the procedure and filling out a form similar to that in Example 6, Attachment B.

6.8.3.2.2 HYDRAULIC GRADE LINE PROCEDURE

Since the design criteria specifies that the HGL elevation at any structure be one foot below the structure top elevation, it is necessary to compute the elevations of the HGL in a storm drain system whether under open channel or pressure flow conditions. The computations are based on the design frequency discharges.

Normally the HGL is assumed to be outlet control. The HGL is determined by first establishing the design tailwater elevation at the outfall and comparing it to the average critical depth (d_c) and the height of the pipe (d_c+D)/2. The larger of these values is selected as the beginning elevation. Depending upon the outfall channel, establishing the tailwater elevation could involve analysis of joint or coincidental occurrence of storm events. Normally, the ratio of the upstream and downstream watersheds is large enough that the tailwater depth would be the normal flow depth of the receiving channel.

Then working upstream along the entire pipe system, compute friction loss for each individual pipe segment, losses due to curvature or bends, and junction losses at the various structures to determine the HGL.

For pressure flow design, refer to Example 6, Attachment B, for a form and procedure for tabulating computed results. Computer programs such as HYDRA make the computation of a system's EGL and HGL simple. However, it is important that the designer understand the process to judge if the input and output are valid.

6.8.3.2.3 SAG POINTS

DelDOT's design criteria require that at sag points where the only drainage available is the storm drain, the pipe draining the sag point and inlet(s) should be sized for a 50-yr frequency rainfall. Although software is available for most of these calculations, they can be done by computing the bypass flow occurring at each inlet during a 50-yr frequency rainfall and accumulating it at the sag point. For sizing the downstream storm drain, one method of determining the additional bypass flow is to convert the flow to an equivalent using the product of the weighted runoff coefficient and contributing area, *CA*, that can be added to the design *CA*; see discussion on the rational formula and the inlet design procedure in Example 5, Attachment B. This equivalent *CA* is found by dividing the 50-yr bypass by the I_{10} in the pipe at the sag point.

Sag point drainage is an important design consideration due to flooding of the roadway and adjacent properties as well as the potential of creating a hydroplaning area. In order to prevent this condition from occurring, the use of flanking inlets is recommended.

6.8.3.2.4 HYDRAULIC PROCEDURES

The hydraulic capacity of a storm drain system is determined with equations 6.16

through 6.19.For circular storm drains flowing full and R = D/4, and the equations become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad (6.63)$$

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2} \qquad (6.64)$$

where

V = Velocity (ft/s)

D = Pipe diameter (ft)

S = Storm drain slope (ft/ft)

n = Roughness coefficient

Q = Pipe flow (ft^3/s)

The references listed in Sections 6.8 and 6.9 contain nomographs and charts that can be used to solve these equations during design.

Another approach to finding pipe flow is to develop a conveyance factor, K, which represents the physical characteristics of the channel/pipe; see equations 6.18 and 6.19.

Using a combination of these equations, the friction slope, S_f, of the conveyance surface can be used to determine the values for the velocity and flow capacity of the pipe. The equations developed are:

$$S_f = \left(\frac{Vn}{1.49R^{2/3}}\right)^2 = 29\frac{n^2}{R^{4/3}}\left(\frac{V^2}{2g}\right) \qquad (6.65)$$

For full flow conditions:

$$S_f = \frac{184n^2}{D^{4/3}}\left(\frac{V^2}{2g}\right) \qquad (6.66)$$

Or,

$$S_f = \left(\frac{Q}{K}\right)^2 \qquad (6.67)$$

where K is defined as

$$K = \frac{1.49}{n} AR^{2/3} \qquad (6.68)$$

Or,

$$K = \frac{0.463}{n} D^{2.67} \qquad (6.69)$$

For concrete pipes (n = 0.012) flowing flow:

$$S_f = \frac{Q^2}{1490\left(D^{8/3}\right)^2} \qquad (6.70)$$

The head loss due to friction H_f is determined by the equation:

$$H_f = S_f L \qquad (6.71)$$

where

L = Length of pipe

However, most design will be done using software. In the references there are tables that show of the results of solving these equations for different pipe materials, slopes, and roughness coefficients. These tables allow for a preliminary evaluation of pipe size and slope as well as checking software output. The full flow capacity is calculated by multiplying the square root of the slope times the K value obtained from Figure 6-25.

6.8.4 FLARED END SECTIONS

Flared end sections (FES's) are to be used outside of the clear zone at intakes requiring personnel safety grates and outfalls. (Personnel safety grates are only placed on the intake end of a pipe run that is not linear; refer to DelDOT Design Guidance Memorandum No. 1-15.) **They are not recommended for roadside drainage installations or on the ends of driveway pipes where crashworthy treatments are needed**; please refer to AASHTO's *Roadside Design Guide*. During installation, the FES shall be placed at the flow line grade of the pipe as with any pipe section. FES's are not manufactured to fit skewed slopes. For a skewed pipe culvert, the FES is placed in line with the pipe, and the fill slope is warped to fit the FES. When calculating the quantity of pipe, the FES is not included in the length of pipe, and is a separate pay item.

Figure 6-25
Circular Pipe Conveyance Factor (*K*)

D Pipe Diameter (in)	*A* Area (ft^2)	*R* Hydraulic Radius (ft)	$K = \frac{1.49}{n} AR^{2/3}$		
			n =0.011	*n* =0.012	*n* = 0.013
12	0.785	0.250	42.2	38.7	35.7
15	1.227	0.313	76.6	70.2	64.8
18	1.767	0.375	125	114	105
21	2.405	0.438	188	172	159
24	3.142	0.500	268	246	227
27	3.976	0.563	367	337	311
30	4.909	0.625	486	446	411
33	5.940	0.688	627	575	531
36	7.069	0.750	790	725	669
42	9.621	0.875	1192	1093	1009
48	12.566	1.000	1702	1560	1440
54	15.904	1.125	2330	2136	1972
60	19.635	1.250	3086	2829	2611

Figure 6-26
Wall Thickness and Approximate Weight of
Circular Concrete Pipe Class IV with Type B Wall Thickness

Size (in)	Wall Thickness (in)	Approximate Weight (lbs. per ft)	Size (in)	Wall Thickness (in)	Approximate Weight (lbs. per ft)*
12	2.0	93	36	4	524
15	2.25	127	42	4.5	686
18	2.5	168	48	5	867
21	2.75	214	54	5.5	1068
24	3	264	60	6	1295
30	3.5	384	66	6.5	1542
33	3.75	451	72	7	1811

* Pipe sections are 8 ft in length.

Figure 6-27
Manning's Roughness Coefficients (*n*) for Pipe

Type of Pipe	Recommended *n*
Concrete - Round and elliptical	0.012
Corrugated Metal - Annular (2 2/3 x ½ in) Helical (2 2/3 x ½ in) Spiral Rib	 0.024 0.020 0.012
Plastic - Polyvinyl High Density Polyethylene	 0.011 0.012
Polyethylene-Double Walled	0.012

Figure 6-28
Friction Slope (ft/ft) for *n* = 0.012, Full Flow

Q (cfs)	Pipe Diameter (in)											
	12	15	18	21	24	27	30	36	42	48	54	60
2	0.003											
4	0.011	0.003										
6	0.024	0.007	0.003									
8	0.043	0.013	0.005									
10	0.067	0.020	0.008	0.003								
12	0.097	0.029	0.011	0.005								
14	0.132	0.040	0.015	0.007	0.003							
16		0.052	0.020	0.009	0.004							
18		0.066	0.025	0.011	0.005	0.003						
20		0.082	0.031	0.014	0.007	0.004						
24		0.118	0.044	0.020	0.010	0.005						
26		0.138	0.052	0.023	0.011	0.006						
25			0.048	0.021	0.010	0.006	0.003					
28			0.061	0.027	0.013	0.007	0.004					
30			0.069	0.031	0.015	0.008	0.005					
32			0.079	0.035	0.017	0.009	0.005					
34			0.089	0.039	0.019	0.010	0.006					
35			0.095	0.042	0.020	0.011	0.006					
36			0.100	0.044	0.022	0.012	0.007					
38			0.111	0.049	0.024	0.013	0.007					
40				0.054	0.027	0.014	0.008	0.003				
42				0.060	0.029	0.016	0.009	0.003				
44				0.066	0.032	0.017	0.010	0.004				
45				0.069	0.034	0.018	0.010	0.004				
46				0.072	0.035	0.019	0.011	0.004				
48				0.078	0.038	0.020	0.012	0.004				
50				0.085	0.042	0.022	0.013	0.005				
55				0.103	0.050	0.027	0.015	0.006	0.003			
60				0.122	0.060	0.032	0.018	0.007	0.003			
65					0.070	0.038	0.021	0.008	0.004			
70					0.082	0.044	0.025	0.009	0.004			
75							0.028	0.011	0.005			
80							0.032	0.012	0.005	0.003		
85							0.037	0.014	0.006	0.003		
90							0.041	0.016	0.007	0.003		
95							0.046	0.017	0.008	0.004		
100							0.051	0.019	0.008	0.004		
105							0.056	0.021	0.009	0.005		
110							0.061	0.023	0.010	0.005	0.003	
115							0.067	0.025	0.011	0.005	0.003	
120							0.073	0.028	0.012	0.006	0.003	
125							0.079	0.030	0.013	0.006	0.003	
130							0.086	0.032	0.014	0.007	0.004	
135							0.092	0.035	0.015	0.008	0.004	
140							0.099	0.038	0.016	0.008	0.004	
145							0.106	0.040	0.018	0.009	0.005	0.003
150								0.043	0.019	0.009	0.005	0.003
160								0.049	0.022	0.011	0.006	0.003
170								0.055	0.024	0.012	0.006	0.004

6.9 PIPES AND PIPE CULVERTS

6.9.1 REFERENCES

Publications

The primary publications for understanding the principles for the analysis of pipes and pipe culverts are:

- *Design Charts For Open-Channel Flow*, Hydraulic Design Series No. 3 (HDS-3), FHWA 1973 (Reprint 1980)
- *Introduction to Highway Hydr*aulics, Hydraulic Design Series No. 4 (HDS-4), FHWA 1983 and 2001
- *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (HDS-5), FHWA, 2005
- *Hydraulic Charts for the Design of Highway Culverts*, Hydraulic Engineering Circular No. 5 (HEC-5), FHWA, 1980
- *Capacity Charts for the Hydraulic Design of Highway Culverts*, Hydraulic Engineering Circular No. 10 (HEC-10), FHWA, 1972
- *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12, (HEC-12), FHWA, 1984
- *Hydraulic Design of Improved Inlets for Culverts*, Hydraulic Engineering Circular No. 13 (HEC-13), FHWA, 1972
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, Hydraulic
- Engineering Circular No. 14 (HEC-14), FHWA, 1975
- *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22, (HEC-22), FHWA, 2001

In addition to these references there are manuals available from the different pipe manufacturers, including the *Concrete Pipe Design Manual* (viewable online at http://www.concrete-pipe.org/techdata.htm) and the *Handbook of Concrete Culvert Pipe Hydraulics* by the American Concrete Pipe Association, *Modern Storm Sewer Design* by the American Iron and Steel Institute, and the design manual of the Plastics Pipe Institute. All of these give pipe size data, general hydrology and hydraulic design, structural design, durability and recommended installation methodology. Much of this information is not available in other publications.

Tables and Charts

Access to the following tables and charts are needed. Most of these can be found in the referenced publications.

- Defining Energy Concept in Pipe Flow
  - Figure III-8 and III-9, HDS-5
  - Figure 7-8, HEC-22
- Manning's Roughness Coefficient *n* for Pipes
  - Figure 6-27
  - Table 14, HDS-4
  - Table 4, HDS-5
  - Tables 3-4 and 7-1, HEC-22
- Friction Slope for Circular Pipe, Full Flow
  - Figure 6-28
  - Charts 53 and 54, HDS-3
- Hydraulic Elements of Circular Sections for Various Flow Depths
  - Figure 40, HDS-4
  - Charts 26 and 27, HEC-22
- Uniform Flow in Circular Sections Flowing Partly Full
  - Table B.2, HEC-14
- Velocity in Pipe Conduits
  - Charts 35 thru 51, HDS-3
- Velocity in Elliptical Pipes
  - Chart 74, HDS-3
- Critical Depth of Flow - Circular Pipes
  - Chart 4B, HDS-5
  - Appendix B, HEC-14
- Critical Depth for Elliptical Pipes
  - Chart 31B, HDS-5
  - Figure B.2, HEC-14
- Entrance Loss Coefficients
  - Table 12, HDS-5
  - Table 7-5b, HEC-22

- Headwater Depth for Concrete Pipe Culverts with Inlet Control
  - Chart 1B, HDS-5
  - Chart 28B, HEC-22
- Head for Concrete Pipe Culverts Flowing Full
  - Chart 5B, HDS-5
- Headwater Depth for C.M. Pipe Culverts with Inlet Control
  - Chart 2B, HDS-5
  - Chart 29B, HEC-22
- Head for Standard C.M. Pipe Culverts Flowing Full
  - Chart 6B, HDS-5
- Headwater Depth for C.M. Pipe-Arch Culverts with Inlet Control
  - Chart 34B, HDS-5
- Head for Standard C.M. Pipe-Arch Culverts Flowing Full
  - Chart 39B, HDS-5
- Headwater Depth for Oval Concrete Pipe Culverts with Inlet Control
  - Chart 29B, HDS-5
- Head for Elliptical Concrete Pipe Culverts Flowing Full
  - Chart 33B, HDS-5
- Allowable Fill Heights for RCP Pipe
  - DelDOT's web site
- Allowable Fill Heights for Round Corrugated Steel Pipe
  - DelDOT's web site
- Allowable Fill Heights for Pipe Arch Corrugated Steel Pipe
  - DelDOT's web site
- Allowable Fill Heights for Round Corrugated Aluminum Pipe
  - DelDOT's web site
- Allowable Fill Heights for Pipe Arch Corrugated Aluminum Pipe
  - DelDOT's web site
- Typical Cross-Sections Elliptical Concrete Pipe
  - Concrete Pipe Design Manual, Illustrations 5.1 and 5.3
- Minimum Slopes for Full Flow in Pipes at 3 ft/s
  - Figure 6-24
  - Table 7-7, HEC-22
- Determining Culvert Size with HEC-5
  - Figure 6-29
- Culvert Design Form
  - Figure 7-7, HEC-22

Computer Software

- WinHY-8 Hydraulic Design of Highway Culverts
- HEC-RAS
- NFF

6.9.2 HYDRAULIC DESIGN OF PIPE CULVERTS

Culverts addressed in this chapter are pipe culverts and are defined as a separate category in drainage design. Their primary function is to carry a watershed's peak discharge flow from one side of a roadway embankment to an acceptable downstream outfall. The important considerations in locating and designing a culvert are:

- Topography
- Debris potential
- Design storm frequency
- Allowable upstream water depth
- Allowable downstream water depth
- Allowable outfall velocity
- Minimum culvert flow velocity
- Risk of overtopping the roadway
- Geometric and safety criteria
- Erosion and sediment criteria

For a project, pipe culverts may be one or a combination of several designated materials depending upon the service life criteria for the project's highway functional classification, the project scope, location within the roadway section, environmental factors, manufacturer's installation requirements, and economics.

Culverts may be single or multiple barrels with the following shapes:

1. Box
2. Circular

3. Elliptical (for concrete pipes only, also called oval concrete pipes), and
4. Pipe arch

Factors that determine the performance, capacity and required culvert size include the following:

1. D = Height of inside barrel opening (ft)
2. HW = Headwater depth at the culvert entrance
3. AHW = Allowable headwater depth at the inlet permitted by design criteria or other safety considerations.
4. d_c = Critical depth (ft) is the depth of flow with minimum specific energy. The specify energy is the sum of the depth plus velocity head for a given discharge.
5. k_e = Entrance loss coefficient for full flow
6. L = Length of culvert
7. S_f = Head loss per length of culvert due to friction (ft)
8. n = Coefficient for surface roughness of the pipe wall (see Figure 6-27)
9. S_o = Slope of pipe
10. TW= Tailwater depth at the culvert outfall
11. V = Mean velocity at any depth (ft/s)

The design method for sizing pipe culverts is based on procedures in HEC-5, HEC-10 and other publications. The following discussion describes the basic concept for culvert design. Computer programs such as HEC-RAS, NFF and HY-8 automate the design methods. However, the designer should have an understanding of the basis used to develop the software.

When using any approved procedure, the results must be summarized to include the projected flow capacity, the pipe size, the required slope, discharge velocity and a comparison of the calculated values with the design criteria.

6.9.3 DESIGN CRITERIA

The design frequency is selected from Figure 6-1. The allowable headwater shall remain at least 6 inches below the edge of roadway embankment, preferably one foot. However, in flood plain areas, the headwater shall conform to the applicable regulations. Since the outlet velocity is invariably greater than the velocity in the natural channel, erosion control provisions for the outlet velocity should be used. Altering the hydraulic characteristics of the culvert can reduce the outlet velocity.

The allowable headwater and tailwater depths are significant factors in culvert design. Some the concerns inaccurately determining these heights are:

- Damage to upstream and downstream properties.
- Flooding of travel lanes.
- Relationship to low point(s) in the roadway profile.
- $HW / D \leq 1.5$
- Proposed/probable future land use affects on the drainage watershed.
- Possible damage to the roadway support structure.
- FEMA 100-yr flood plain regulations.
- Potential for debris clogging and possible bypassing of culvert to the sag point.
- Effect on wetlands and other environmentally sensitive areas.
- Stability of the downstream discharge area beyond the point of outfall.

Figure 6-29
Culvert Size Determination Using HEC-5

Step 1	Enter known data, i.e., Q, L, S_o, and AHW, in an acceptable culvert design form. For the initial trial, use the friction slope for S_o.	Step 6	If $TW \geq D$, $h_o = TW$. If $D > TW$, then $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger. Obtain d_c from the charts.
Step 2	Using the principles of open channel flow; determine the design storm depth of flow; assume this is the TW unless downstream conditions control this depth.	Step 7.	Compare the two values of HW. The higher value indicates what the flow condition is, i.e., outlet or inlet control
Step 3	Select a trial size assuming inlet control, $HW/D = 1.5$; use appropriate HW nomograph. If $HW > AHW$, try larger size pipe until $HW < AHW$.	Step 8.	If inlet control prevails, the type of culvert, the size and entrance type being evaluated are correct. Other combinations of pipe materials and entrance types could be checked if appropriate
Step 4	Assume that outlet control prevails.	Step 9	The values of HW are compared with AHW. The culvert size is adjusted and reevaluated until $HW = AHW$
Step 5	Find HW using: $HW = h_o - LS_o$. See HEC-5 for procedure and chart references.		

6.9.4 DESIGN PROCEDURE SUMMARY

Flow Controls

Flow through the culvert is determined by the available energy differential between the inlet and outlet. Depending upon this relationship, the culvert may flow under two types of flow. The design must consider each type of flow to select a culvert type and size.

- **Inlet Control:** The culvert size, inlet geometry and headwater depth determine the flow capacity. Inlet control exists as long as water can flow through the pipe at a greater rate than it enters. This type of flow is not affected by downstream conditions nor the roughness, length and slope of the culvert. Primarily the barrel shape, cross-sectional area, inlet edge and headwater depth affect inlet control. Culverts flowing under inlet control will always flow partially full.
- **Outlet Control:** Outlet control exists as long as water can enter the culvert at a greater rate than the water can flow through it. Capacity is affected by energy losses beginning at the outlet with the backwater affect of the tailwater depth, proceeding upstream with the losses due to culvert barrel characteristics including the size, shape, type of material, roughness, slope and length. At the inlet, losses due to the inlet geometry and headwater depth affect the capacity. The culvert can flow either with partial or full flow.

The following is a brief overview of the two most commonly used methods for determining culvert sizes. One method is based on a series of culvert capacity charts and the other is based on a series of nomographs using inlet and outlet control as the primary parameters. These methods involve manually determining the required culvert size by trial and error. The designer may choose to use the HY8 Culvert Analysis Microcomputer

Program, HEC-RAS or other approved software.

The steps for design are easy and provide a reliable design and/or a check on computer generated designs. The methods are limited and do not address hydrograph routing or velocity concerns involving energy dissipators or outlet scour potential.

Critical depth is an important consideration in the design of culverts. For the available energy head, the maximum discharge occurs at critical flow and the depth of flow at this point is defined as the critical depth. A culvert under inlet control does not have the hydraulic losses associated with outlet control and has its maximum capacity at critical flow.

For inlet control the critical depth occurs in the barrel near the culvert entrance and normal flow and depth is reached before the outfall. Conversely under outlet control the critical depth occurs near the outfall and the tailwater depth becomes an important factor.

Capacity Chart Method - HEC-10

Define Design Data

1. Establish the design *Q*.

2. Approximate the length *L* of the culvert.

3. Select a roughness factor *n* and preliminary slope.

4. Determine the allowable headwater depth.

5. Determine the allowable outfall velocity based on the proposed erosion characteristics of the discharge point.

6. Select a trial culvert, including a barrel cross sectional shape and entrance type. Approximate the initial trial pipe size by dividing the allowable headwater depth by a factor of two.

Determine culvert size

Select the appropriate capacity chart for the trial culvert size. Following the chart directions, find the headwater depth using inlet control. If this exceeds the allowable headwater depth, then try a larger pipe. Conversely, if it is less than the allowable, then try a smaller pipe. Next compare the headwater depths for outlet and inlet control. The control with the higher headwater governs and is used to continue the design.

Determine outlet velocity

A. Outlet Control—Outlet velocity equals the flow divided by the flow cross sectional area at the outlet. If the outlet is not submerged, the flow area is usually based on a depth of flow equal to the average of the critical depth and the pipe's vertical height.

B. Inlet Control—The outlet velocity is calculated using Manning's formula. Capacity charts for solving this equation for full flow and partial flow values are found in the references.

Selection Report

An important step is to summarize the results showing the selected pipe size, pipe material, required headwater and outlet velocity. The design criteria shall be compared with the design results to ensure compliance.

Nomograph Method HEC-5

Define Design Data

1. Establish the design *Q*.

2. Approximate the length, *L*, of the culvert.

3. Select a roughness factor *n* and preliminary culvert slope.

4. Determine the allowable headwater depth.

5. Determine the allowable outfall velocity based on the proposed outfall conditions, i.e., erodible or non-erodible discharge point.

6. Select a trial culvert, including a barrel cross sectional shape and entrance type. Divide the allowable headwater depth by a factor of two for the initial trial size using the friction slope for S_o.

Find headwater for trial culvert

A. Inlet Control.

(1) Given the design *Q*, size and type of culvert, follow the use directions and the appropriate inlet control nomograph to find the *HW*.

(2) If the *HW* is greater than the allowable, then try another trial size until HW is acceptable for inlet control.

B. Outlet Control

(1) Given Q, size and type of culvert, and estimated tailwater depth (TW, ft), above the invert at the outlet for the design frequency flood condition in the outlet channel:

(a) Locate the appropriate outlet control nomograph for the type of trial culvert. Find the entrance loss coefficient, k_e.

(b) Find the head (H) following the instructions for using the nomograph.

(2) For tailwater (TW) elevation equal to or greater than the top of the culvert at the outlet set h_o equal to TW and find HW by the equation:

$$HW = H + h_o - S_oL \qquad (6.72)$$

(3) For tailwater elevations less than the top of the culvert at the outlet, use $h_o = d_c + D/2$ or TW, whichever is the greater, where d_c (the critical depth in feet) is determined from the appropriate critical depth chart.

C. Compare the headwaters previously found for inlet and outlet control. The higher headwater governs in determining the flow control for the given conditions and the selected trial culvert.

D. If HW is higher than acceptable, select a larger trial culvert size and find HW as previously described.

Determine outlet velocity

A. Inlet control — Determine the outlet velocity using Manning's equation.

B. Outlet control —– The outlet velocity is equal to the discharge flow divided by the flow cross sectional area at the outlet.

Selection Report

Record the results including the selected culvert size, type, headwater depth, and outlet velocity. Where necessary, compare with the design criteria and indicate whether or not the criteria have been met.

6.10 STORMWATER MANAGEMENT

The goal of stormwater management is to restrict the peak rate or volumetric rate of stormwater runoff after the project area is developed to the same as or less than it was originally, and to preserve or improve the quality of the runoff.

Restricting the post-development peak flow rate reduces the potential for increased downstream erosion and flooding. Typically this quantity control is accomplished using wet ponds or infiltration. These ponds are designed to provide sufficient storage volume so that a restrictive outlet will release no more than the pre-development rate. Stormwater ponds are also used for stormwater quality management with sediment forebays and permanent non-draining pools that encourage the deposition of runoff-borne pollutants, most of which are either sand sized or are bonded to sand sized particles. This allows the removal of an estimated 80% of generated pollutants by controlling the flow rate and letting them naturally settle to the bottom.

If a higher degree of treatment is required, specialized designs are available but they are expensive and generally require extensive, regular maintenance.

DelDOT's Stormwater Engineer is the responsible authority for ensuring a project's stormwater management control and erosion control features are adequate and in compliance with the latest policies, rules and regulations. This section is a general discussion of these subjects to inform and supplement, but not replace any approved design guides, manuals, memorandums of agreement, or other requirements.

Sections 6.10, 6.11 and 6.12 are an overview of DelDOT's *ES_2M Design Guide*, DNREC's *Green Technology: The Delaware Urban Runoff Management Approach*, *DURMM: The Delaware Urban Runoff Management Manual*, and NRCS's *Pond Code 378*. In addition, there are Internet web sites with extensive information on stormwater management concepts and design methodologies.

In stormwater management, a commonly used term is Best Management Practices (BMP's). BMP's are policies, practices, procedures or structures used to mitigate the adverse impacts on surface water quality resulting from changes in the watershed. BMP's are subject to the available area and site conditions for implementation.

One stormwater management strategy is Low Impact Development (LID). LID emphasizes conservation and the use of on-site natural features integrated with engineered, small-scale hydrologic controls in order to more closely mimic the pre-development hydrologic function of the site. LID strategies and techniques can be used to reduce the impact of runoff on the receiving waters. Some LID principles include:

- Maximize retention of native forest cover and restore disturbed vegetation to intercept, evaporate and transpire precipitation.
- Retain and incorporate topographic features and patterns.
- Create a hydrologically rough landscape that slows storm flows and increases the time of concentration.
- Provide multiple or redundant LID measures in order to increase reliability.

6.10.1 REFERENCES

PUBLICATIONS

- *ES_2M Design Guide*, DelDOT
- *Green Technology: The Delaware Urban Runoff Management Approach*, DNREC
- *Delaware Erosion & Sediment Control Handbook*, DNREC
- *Hydraulic Design of Highway Culverts*, Highway Design Series 2 (HDS-2), FHWA
- *Design of Riprap Revetment*, Hydraulic Engineering Circular 11 (HEC-11), FHWA
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, Hydraulic Engineering Circular 14 (HEC-14), FHWA
- *Design of Roadside Channels with Flexible Lining*, Hydraulic Engineering Circular 15 (HEC-15), FHWA
- *Hydrology*, Highway Engineering Circular 19 (HEC-19), FHWA
- HEC-22, *Urban Drainage Design Manual Second Edition*
- *Open-Channel Hydraulics*, Ven Te Chow, 1959
- *National Engineering Handbook*, NRCS
- Various State DOT sites i.e. VDOT, MDOT, WSDOT and IDOT

COMPTER SOFTWARE

- Delaware Urban Runoff Management Model (DURMM)
- HEC-2: Floodplain Determination
- TR-20: Large Drainage basin hydrology
- TR-55: Hydrology, including hydrograph generation
- FLOWMASTER: Hydraulic design for various cross-sections such as trapezoidal, v-shape, rectangular, and closed pipes.
- PondPack Hydraulics including pond routings
- HydroCad
- Other approved software on DNREC's Drainage and Stormwater web site

6.10.2 EROSION CONTROL AND STORMWATER MANAGEMENT

The regulations involving Stormwater Management, Erosion, and Sedimentation Control are intended to minimize the adverse impact of projects that change the existing natural environment as well as manmade features due to increased flow and degraded water quality.

Erosion control measures are used during construction to manage surface scour and washouts created by uncontrolled runoff through the construction site and the subsequent deposit of sediment and other pollutants on natural areas and adjacent properties. Permanent stormwater measures

are frequently incorporated into the temporary erosion control plans.

Typically, DelDOT projects affect the time of concentration, rate and volume of stormwater peak runoff from the site due to an increase in the amount of impervious surfaces such as a larger roadway pavement section, sidewalks, and concrete gutters.

Erosion control practices include diversion of water entering the site by the construction of diversion berms and swales on the uphill side to route water around or away from the work area; and/or piping clean water all the way through the project and discharging below the work area. Limitations on total work area allowed to be stripped at any time, and stabilized construction entrances are also a form of erosion control, as is frequent interim mulching, and permanent post construction vegetative stabilization.

6.10.3 SEDIMENT AND STORMWATER MANAGEMENT

Sediment control involves the controlled deposition and removal of materials eroded during various construction activities. Sediment control practices include placing silt fencing at designated locations on the construction site; sediment traps; sediment basins; stone check dams in the flow lines of swales and ditches; and filters around storm drain inlets (catch basins). Early construction phasing of permanent features such as wet and dry stormwater ponds with sedimentation forebays, riprap-stilling basins at culvert outlets, biofiltration swales, and vegetative filtration strips are also used. All permanent facilities are cleared, cleaned and reconstructed as necessary at the end of construction activities.

Stormwater management, erosion, and sediment control, are highly interrelated. DNREC and DelDOT combine these elements into one set of plans termed a Sediment and Stormwater Management Plan that accompanies the Stormwater Management Report.

6.10.4 GENERAL CRITERIA

The following is based on the Delaware Sediment & Stormwater regulations.

- Stone check dams are used in swales, ditches and channels to control velocities and erosion until sufficient cover has been established. .
- Outlet protection is required at all discharge points of pipes, channels and spillways.
- Erosion control matting is required on slopes of 3:1 or greater and all concentrated flow locations.
- Sediment traps and basins shall be utilized and sized to accommodate 3600 ft^3 of storage per acre of contributing drainage area until project stabilization is complete. These structures shall be located at the base of the drainage area. The required minimum length to width ratio is 2:1.
- Drainage calculations provide pre- and post-development velocities, peak rates of discharge, and inflow and outflow hydrographs of stormwater runoff at all existing and proposed points of discharge from the site for the 2-yr and 10-yr and 100-yr frequency storm.
- Hydrological data is to be developed using TR-20 or TR-55 with a storm duration based on the 24-hour rainfall event. For projects south of the C&D Canal, the Delmarva Unit Hydrograph shall be used.
- All ponds are to be designed and constructed in accordance with approved version of NRCS Small Pond Code 378.
- For water quality, ponds are sized to store the runoff from a 2-yr storm with a maximum of 1 inch of runoff released over a 24-hour period.
- Post-development peak rates of discharge for the 2-yr and 10-yr (and the 100-yr frequency storm events for projects north of the C&D Canal) and in some cases post-development volumetric rate shall not exceed the pre-development peak rates of discharge based on the same storm frequency criteria.

- Set aside areas for disposal of sediment removed from the stormwater management facilities must be provided. These areas are to be large enough to accommodate at least 2% of the stormwater management facility volume to the elevation of the 2-yr storage volume elevation, have a maximum depth of one foot, and have a slope not to exceed 5%.
- Wet ponds are to have a forebay to act as a sediment trap and two ten foot (10 ft) wide safety benches, one at one foot above and the other at one foot below the normal pool elevation.

6.10.5 PROJECT DESIGN AND REVIEW CHECKLIST

DelDOT's *ES_2M Design Guide* and the *Delaware Erosion & Sediment Control Handbook*, are the primary references for preparing the Erosion, Sediment and Stormwater plans. An approved checklist is submitted to the Stormwater Engineer at each plan review stage for any project disturbing over 5,000 square feet of land. A completed checklist is required for each plan submission. The items that do not apply to the project shall be marked "N/A".

6.11 STORMWATER QUANTITY MANAGEMENT

6.11.1 REFERENCES

For publications and computer software, see Section 6.10.

6.11.2 GENERAL

Stormwater quantity control is based on the requirement that the peak rate of discharge from an area after development cannot exceed the peak rate of discharge from that site before development. Both infiltration techniques and stormwater detention pond with a restrictive outlet structure achieves this goal. The measures to control stormwater quantity also can be valuable in maintaining the quality of project runoff and are usually designed concurrently. One such measure is a wet pond that serves both purposes. The permanent pool of water allows for filtration of contaminants, groundwater recharging, creation of wildlife habitat, and other environmental benefits.

Since much of stormwater design is generated through computer analysis, there are several practical considerations, including the following:

1. Field review of the site to ensure the stormwater can physically and economically be directed to the selected location. Just designating an area on the plans is not adequate without a field check.
2. Locate the water table to ensure the pond will have stormwater capacity. If the pond will be normally filled to capacity with groundwater then it will provide little detention value.
3. Take soil borings to ensure the site can be economically excavated or at least have the proper excavation bid items, e.g., rock excavation.
4. Ensure there are provisions for periodic maintenance, e.g., access.

Another important consideration is that many stormwater management facilities are in or adjacent to developed areas and may not be considered a community asset. Therefore, the design should consider aesthetics and safety. Ponds become an attraction particularly when they are flowing full. It is important that the outlet structure be designed for the safety of those who may use it for recreation. The most economical or hydraulically efficient design may not be the safest alternative for the site conditions.

A landscape plan to accompany each pond design is desirable. Woody vegetation, shrubs, or trees planted on the embankment are not allowed. Fencing is discouraged because of maintenance and perceived safety issues. Methods of vegetating ponds vary.

6.11.3 POND DESIGN

Stormwater ponds fill with water when the rate of inflow exceeds the rate of outflow. The volume of water that is temporarily stored during a runoff event can be expressed graphically as the area between the inflow and

outflow hydrographs. Figure 6-30 outlines the steps to design a wet pond.

6.11.3.1 POND SIZING

The hydraulic analysis of detention ponds generally involves the following steps:

Step 1. Define design criteria. Determine the required storm frequency events and other criteria as mandated by DNREC. In addition, follow the design criteria based on the functional highway classification as defined in Figure 6-1.

Step 2. Select sites. Based on the topography of the project area select feasible preliminary pond locations. Select these locations based on: proximity to the project; an acceptable outfall location to drain the water (such as a natural stream); adequate space to construct a structure; moderate to flat slopes so grading steep slopes doesn't consume all the space; suitable geotechnical conditions; outside floodplains and wetlands; etc. Other considerations include setting tentative elevations for the bottom, permanent pool level, principal spillway including the various storm frequency structure(s), and the emergency spillway level.

Step 3. Determine peak flows. Using TR-55, TR-20 or another approved version of the NRCS Soil Cover Complex Method, determine the peak rates of runoff for existing conditions (pre-development) from the drainage areas contributing to each selected pond location. These rates cannot be exceeded by post-construction flows.

Step 4. Determine trial pond size. As a design tool, TR-55 has a manual method of determining an initial trial pond size for the designer to assess possible pond location. The method estimates the storage volume required to allow for the control of the peak outflow discharge. The pre- and post-development flows as developed by TR-55 and using TR-55 Figure 6-1 allows the designer to quickly estimate the storage volume. This value is then geometrically related to the available area and depth at a selected site.

Step 5. Fit pond to available location. Sketch the trial pond on the plans to determine its limits and if it is viable. Establishing the bottom elevation of the pond is the starting point. Selecting this elevation is based on hydraulics and geotechnical considerations. Hydraulic limitations involve locating and setting the invert of a positive discharge outfall such as an existing stream or ditch, figuring back to the pond location at a nominal 2% slope. The pond has to be able to drain, so the outlet elevation has to be above the discharge point. Geotechnical considerations include an analysis of the types of soil or rock that may be encountered. Excavating unsuitable material or rock is expensive. The determination of the normal groundwater level at the proposed pond location is extremely important. First, the storage capacity will be decreased by the continual inflow of groundwater. Second there could be a need to protect the existing aquifer from contamination. This also is not inexpensive.

Depending upon the design, additional space may be required for sedimentation forebays or safety benches. For the design of small to moderate ponds (most serve only a few acres in area), allowing an extra 30 to 50% of surface area would be appropriate for initial sizing. Current regulations may also require buffer areas around the pond for access or wetlands protection.

The grading for the pond involves embankment slopes typically 4:1, but no steeper than 3:1. The pond shape is an important part of its function. Irregular shapes with ratios of length to width of at least 2 to 1 are preferred. For safety purposes, there are two 10 ft wide safety benches located one foot above the normal pool and one foot below the normal pool level. The upper bench can be eliminated if the side slopes are 4:1 or flatter. The recommended depth for a wet pond is 3 to 6 ft. The top of the embankment should be 10 ft wide. A 10 ft buffer for access around the toe is preferred. The design top elevation should allow for future embankment settlement.

Obviously ponds consume considerable land area and early site selection is very important.

Step 6. Verify the trial pond size. The size approximated in Steps 4 and 5 must be checked through the use of inflow and outflow hydrographs, analyzing the discharge capacity of alternative outflow structures and routing the various storm flows through the pond. Because several design storms and types of outfall structures must be routed through the pond, approved stormwater management software is used to generate the data. The input data for the software must comply with the current regulations.

The process is to develop stage-storage relationships using the estimated available storage at various pond elevations, then determine what type of spillway is the best outlet structure. Two typical types are a rectangular concrete box or concrete wall with assorted weirs and orifices to control any water quality flow as well as several storm frequency flows. Computer software quickly determines the discharge characteristics of most of the combinations. The discharge characteristics of the outlet structure constitute the stage discharge curve. Size the outlet structure for water quality in accordance with the current DNREC regulations.

Computer software combines the stage-storage curve and the stage-discharge curve based on their common factor stage. The inflow hydrograph is then routed through the pond for each of the required frequencies.

The routed outflow peak rate is compared with the pre-development peak rate or other mandated criteria. Currently, it must be equal to or less than existing for all frequencies. If the routed outflow is greater or considerably less then allowed, then a change in the pond size and/or outlet structure is evaluated. Since it is recognized that there is a rather large error (25% for TR-55) in developing stormwater management volumes, downsizing ponds should not be done without careful study of both the input and generated data.

Step 7. Check emergency spillway. The maximum stage (elevation) reached in the pond during the storm routing (particularly the 100-yr) is also an important part of the analysis. Regulations require one foot (or more) of freeboard (height remaining) at peak stage. If the freeboard is less than one foot, regrading the pond is needed either by raising the embankment or regrading the pool to increase the storage, lowering the detention elevation. Sometimes the pond is so small that the water overtops the dam; this is unacceptable and must be remedied by regrading to make the pond considerably larger, by about a third as a first approximation. If the maximum routed elevation is significantly under the freeboard requirement, a reduction in pond size might be appropriate.

Step 8. Structural review and hazard classification. After the final embankment details are determined, a structural review and hazard classification as to the potential damage that might occur due to a major breach or failure of the structure is performed. The analysis is done using national dam safety standards based on the ultimate land use that may occur during the life of the structure.

6.11.3.2 TYPES OF OUTLET STRUCTURES

The effectiveness of a storage basin is very dependent upon the outlet structure. An outlet structure consists of the principal spillway with provisions for emergency overflow. The principal spillway should have the capacity to convey the design discharge to predevelopment levels as mandated by DNREC from the storage basin over a 24-hr duration without allowing the flow to pass through the emergency spillway.

Principal spillways include a combination of weirs, orifices, drop inlets, and pipes. The typical principal spillway consists of either a weir that controls the flow over the embankment or a multistage riser structure functioning as a drop inlet with one or more weirs and outlet conduits that are designed to control one or more selected storm flows. The multi-stage riser system uses separate

openings of various sizes placed at different elevations to control the rate of discharge from the watershed for the specified design storms.

The type of spillway depends upon topography, the size of the storage basin, and the drainage characteristics of the downstream conveyance system. The principal spillway is sized to carry the design storm without allowing flow to enter the emergency spillway using extended detention devices.

If site conditions do not allow for an emergency spillway, then it would be necessary to design the principal spillway to carry the 100-yr storm flow without overtopping the facility. Sizing an outlet structure is based on hydrologic routing calculations as well as the potential threat to downstream life and property. The crest elevation of the principal spillway must be at least 1.0 ft below the crest elevation of the emergency spillway.

The placement of emergency spillways should be on original ground and avoid downstream features such as residences and other facilities.

6.11.3.3 OUTLET HYDRAULICS

A stormwater management facility has an outlet system that controls water flow and quality. The hydraulic design depends on whether the storage basin is intended to be dry or permanently wet. Frequently it is intended that the facility function as an extended detention system that provides control over both water quantity and water quality. This means it allows the pre-development flow to slowly be released over a 24-hr period into the outfall channel while a permanent pool of water is retained in the pond. Preferred design alternatives and criteria per current regulations should be verified before beginning the design to determine the method(s) of stormwater management. The three most commonly used outlets are discussed briefly in the following sections.

6.11.3.3.1 ORIFICES

Orifices are small openings in an outlet structure. For stormwater quality, a circular opening is typically provided to extend detention flow.

When the water rises above the top of the opening, the discharge through an orifice is determined using:

$$Q_o = C_o A_o \left(2gH_o\right)^{0.5} \qquad (6.73)$$

where

Q_o = Discharge through the orifice (ft^3/s)

C_o = Orifice entrance coefficient

A_o = Cross-sectional area of orifice (ft^2)

g = Acceleration due to gravity (32.2 ft/s^2)

H_o = Total head of water (ft), measured from water surface to the center of the orifice. If the orifice outfall is submerged, then H_o is the elevation difference between the headwater and tailwater surfaces.

C_o is a variable coefficient that depends upon the orifice diameter, edge shape and its wall thickness as follows:

- Sharp-edged orifice with walls thinner than the diameter: $C_o = 0.60$
- Sharp-edged orifice with walls thicker than the diameter: $C_o = 0.80$
- Rounded edge orifice: $C_o = 0.92$

6.11.3.3.2 CONCRETE WALL SPILLWAY WITH WEIR

Weirs are used in lieu of a riser/barrel system for shallow basin spillways. The weir outlet is a channel, not a pipe, which operates more efficiently with less maintenance and allows for greater opportunity for improving water quality. The top of the spillway weir wall is used to control the storage volume and the outlet flow. It is important that weir structures constructed on an embankment be protected from undermining and erosion.

Weirs are also commonly used in a concrete box riser system. (See Section 6.11.3.3.3.) The shape, size and number of the weirs may vary to control the runoff for various storm conditions. In addition, concrete box risers allow for the easy installation of stormwater quality orifices and, if necessary, an opening at the bottom to drain the pond.

Two common types of weirs are the broad crested weir and the V-notch weir.

The discharge over a broad crested weir is:

$$Q_w = C_w L_w H_w^{1.5} \qquad (6.74)$$

where

Q_w = Discharge over the weir (ft^3/s)

C_w = Broad-crested weir coefficient

L_w = Length of the weir (ft)

H_w = Head of water over the weir crest (ft)

C_w varies based upon a relationship between the width of the weir and the head measured at a designated point upstream from the weir. An average value is 3.1 or 3.3.

The discharge over a V-notch weir is given by the equation:

$$Q_{vw} = C_{vw} \tan\left(\frac{\theta}{2}\right) H_{vw}^{2.5} \qquad (6.75)$$

where

Q_{vw} = discharge (ft^3/s)

C_{vw} = weir coefficient, usually 2.5

θ = angle of the notch at the apex (degrees)

H_{vw} = total energy head (ft)

In the design of multi-stage risers, notched or rectangular weirs are frequently used to control the flow from one or more design years. When the depth of flow rises above the top elevation of the weir, the weir may be subject to orifice flow conditions. See Section 6.11.3.3.3.

6.11.3.3.3 DROP INLETS

Drop inlets are used in spillway design and may be square, rectangular or circular. The vertical portion of a drop inlet is referred to as a riser. The inlet is a horizontally positioned opening through which water enters, drops vertically through a shaft, and then flows through a horizontal pipe culvert (referred to as a barrel) and then discharged at the downstream outlet.

A drop inlet spillway may operate under three conditions depending upon the height of flow over the inlet. When the water elevation is just slightly above the top elevation, the inlet operates as a weir. When the water increases and begins flowing into the inlet from all directions, the flow actually becomes unstable and the inlet operates as an orifice. For a circular pipe riser, this flow condition usually occurs when the height of flow (*H*) is 1.2 to 1.5 times the diameter (*D*). Eventually, the head can be high enough that the riser flows full and operates as culvert flow. Normally the spillway design prevents this level of flow from occurring.

There is a point in orifice flow where vortex flow occurs, resulting in severe turbulence and reduced inlet capacity. This condition is to be avoided by ensuring the riser is large enough to operate under the weir flow condition, the barrel is at full flow with as low a head over the crest as practical, and before orifice flow develops.

Hydraulically the ideal flow condition occurs when the outlet conduit and riser are operating at full flow. The riser should operate at weir flow with the lowest head over the top as practical. In the design of the riser, determine the elevation over the riser when the flow transitions from weir flow to orifice flow and then make sure that the barrel controls the flow at that elevation. Since this transition point is difficult to accurately determine, it is usually sufficient to assume that the transition occurs where the weir flow equation value is equal to the orifice flow equation value.

If the H/D ratio is less than 1.5, conduits smaller than 12 inches in diameter are usually analyzed as a submerged orifice. The barrel is usually designed as a pipe culvert flowing full with inlet control using the procedure outlined in HDS-5.

The drop inlet spillway is treated as a combination of weir, orifice and pipe flow. Various levels of flow are analyzed using the weir and orifice equations. If the head is high enough (normally avoided), the drop inlet acts as a pipe and the flow is determined by the equation:

$$Q = A\left[\frac{2gH}{1+k_b+k_e+k_f L}\right]^{0.5} \qquad (6.76)$$

where

H = difference between headwater and tailwater elevations (ft)

k_b = bend loss coefficient, use 0.6

k_e = entrance loss coefficient, use 0.5

k_f = barrel friction loss coefficient

L = length of pipe (ft)

The other variables are as previously defined.

Since multi-stage risers have to be analyzed based on several design storms, various spillway combinations and flow conditions, computer software is used. A stage-discharge curve is developed plotting the discharge versus head for each of the flow conditions. The minimum flow for any given head is the discharge used in continuing the evaluation of the selected spillway, pond size etc.

Example 7, Attachment B, contains a simplified pond design example.

6.12 STORMWATER QUALITY MANAGEMENT

6.12.1 GENERAL

The preferred methods of protecting and improving water quality are referred to as Green Technology BMP's. DNREC's *Green Technology: The Delaware Urban Runoff Management Approach* provides detailed discussion, design guidance and construction details for designing nonstructural BMP's. These include the biofiltration, bioretention and infiltration design concepts. BMP's are most effective and require the least maintenance if designed as a system involving several types of BMP's working together.

6.12.2 BIOFILTRATION

Biofiltration techniques are intended for improving post-construction water quality. There are two basic types of biofiltration devices: swales and filter strips. Swales are shallow ditches that carry flow, whereas filter strips are vegetative patches that intercept sheet flow. The design should emphasize biofiltration, rather than transporting flow with the greatest possible hydraulic efficiency. The design is therefore based on criteria that promote sedimentation, filtration, and other pollutant removal mechanisms.

Biofiltration Swale — Grass

This BMP is designed to convey stormwater at a non-erosive velocity while improving water quality through infiltration, sedimentation and filtration. Check dams can be used to slow the flow and create small temporary ponding areas to promote infiltration. This option is one of the least expensive as it involves no engineered filter fabric, imported pervious materials or outlet structures. The design considerations are channel capacity; erosion; a runoff velocity of 1 fps for the design storm; and a total length providing the desirable residence time. These swales usually involve a typical section that has a wider bottom width, flatter side slopes and denser vegetation than a normal open channel.

Filter Strips

Filter strips are relatively flat graded areas that are heavily vegetated. They are designed to treat runoff and remove pollutants primarily through vegetative filtration. They are not intended as a stand-alone solution but as part of a treatment system and/or pre-treatment for another BMP. Filter strips are effective in providing a buffer between incompatible land-uses, can be landscaped and made aesthetically pleasing, and provide groundwater recharge in highly pervious soil areas. Filter strips may be constructed to allow treatment of sheet flow directly from a site or natural areas that are left around drainage channels. The flow characteristics for natural areas are similar to shallow concentrated flow.

Constructed filter strips are more effective if the entering flow is spread out maintaining sheet flow onto the filter and a shallow ponding area is created through the use of berms.

6.12.3 BIORETENTION

Bioretention BMP's are structural stormwater controls that accept and temporarily store the water quality volume using special soils and vegetation in shallow basins or landscaped areas. Their function is to remove pollutions, not control runoff. Some advantages are they can be easily integrated into a site's landscape plan, are aesthetically pleasing and need low maintenance.

Bioretention Areas

Bioretention areas are engineered, landscaped, shallow stormwater basins. Stormwater collected in the upper layer of the basin is filtered through surface vegetation, a mulch layer, and a pervious soil layer, and then temporarily stored in a stone aggregate base layer. The water quality volume is drained from the aggregate base by infiltration into the underlying soil or/or removed to an outlet through a perforated pipe subdrain. These systems work best if they are combined with a perimeter grass filter strip or grass swale to reduce the sediment and velocity prior to entering the biorentention area.

6.12.4 INFILTRATION

Most BMP's partially rely on infiltration for their effectiveness. Stand-alone infiltration BMP's are designed specifically to control a selected volume of the runoff, retain it, and infiltrate all or part of it into the ground.

Infiltration depends upon the underlying soils to provide significant capacity to absorb the proposed flow. In addition, there must not be a possibility of adversely affecting the existing groundwater or aquifer. This technique is the most site-sensitive, most maintenance-intensive and most likely to fail of the BMP's. Because of the potential of clogging infiltration, BMP's are usually constructed after a site has been stabilized and has an established vegetative cover with an upstream BMP for removal of sediment. The two types of infiltration techniques are infiltration trenches and basins.

Infiltration Trenches

Since they occupy the smallest area infiltration trenches are the BMP most commonly used to treat the water quality volume and recharge the groundwater. These trenches are long, narrow and filled with a freely draining stone aggregate. The trapped water is filtered into the underlying soil. Normally there is no engineered outlet. In order to prolong their effectiveness and reduce maintenance there must be a positive sediment trap system above the site. Although the trenches can process fine sediment, coarse sediment can quickly clog the trench.

Infiltration Basin

If more area is available, an alternative is to provide an infiltration basin. Essentially this is a dry pond to trap water and discharge it into groundwater flow through infiltration. This concept uses heavy vegetation to control velocities and increase the percolation.

Infiltration techniques are more maintenance intensive and site specific. There must be highly pervious underlying soils, no rock and no potential for contaminating the existing groundwater. In addition, construction scheduling must be adjusted to allow for project completion prior to their installation.

Figure 6-30
Design Steps for a Wet-Extended Detention Stormwater Pond

Step 1	Determine hydrologic data for the site and confirm pre-developed flow rates.
Step 2	Compute runoff water quality volumes and compute release rates.
Step 3	Estimate the permanent pool volume and extended detention volume.
Step 4	Determine forebay requirements and preliminary locations and size(s).
Step 5	Prepare a preliminary grading plan for the stage-storage curve.
Step 6	Field review the project. Locate a site area and confirm a stormwater pond is practical using: the estimated pond size; configuration; forebay locations; maintenance access; set aside areas; and safety features.
Step 7	Design the water quality orifice or weir.
Step 8	Set the permanent pool volume and elevation.
Step 9	Size the 2-yr control orifice or weir.
Step 10	Check for performance of the 2-yr opening.
Step 11	Size the 10-yr control opening.
Step 12	Check the performance of the 10-yr opening.
Step 13	Perform a hydraulic analysis of the facility including the forebay, the pond, the riser flow control, the barrel inlet flow and the barrel outlet.
Step 14	Size the 100-yr release opening or emergency spillway.
Step 15	Check total discharge and performance of the 100-yr control opening.
Step 16	Design any required outlet protection as per HEC-14.
Step 17	Perform any required buoyancy calculations.
Step 18	Determine the required seepage control.
Step 19	Check final design of inlets, sediment forebays, outlet structures, maintenance access, and safety features making sure all criteria has been met.
Step 20	Based on the final grading and design, determine the pond hazard classification.
Step 21	Develop a landscape plan.

Steps 7 to 12 are used to determine the appropriate outlet structure and stage-discharge curve for the storm events as mandated by DNREC.

Chapter Six

ATTACHMENT A
DRAINAGE DESIGN AIDES

GENERAL

DELDOT DESIGN CRITERIA

Design Frequency ..... DelDOT Figure 6-1
Allowable Spread on Pavement Cross Section ..... DelDOT Figure 6-2
Miscellaneous Design Criteria ..... DelDOT Figure 6-3

RECOMMENDED PUBLICATIONS

- AASHTO - *Model Drainage Manual*
- DelDOT - *ES_2M Design Guide*
- DNREC
  - *Delaware Erosion & Sediment Control Handbook*
  - *Green Technology: The Delaware Urban Runoff Management Approach*
- FHWA
  - HDS-2 *Highway Hydrology* (FHWA-NHI-02-001)
  - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
  - HDS-5 *Hydraulic Design of Highway Culverts* (FHWA-NHI-01-020)
  - HEC-2 *Water Surface Profiles Software and User's Manual*
  - HEC-11 *Design of Riprap Revetment* (FHWA-IP-89-016)
  - HEC-10 *Capacity Charts for the Hydraulic Design of Highway Culverts*, 1972
  - HEC-12 *Drainage of Highway Pavements*, 1984 (replaced by HEC-22)
  - HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
  - HEC-15 *Design of Roadside Channels with Flexible Lining* (FHWA-IF-05-114)
  - HEC-19 *Hydrology*, 1985 (replaced by HDS-2)
  - HEC-22 *Urban Drainage Manual*, 2001 (FHWA-NHI-01-021)
- NRCS
  - *Urban Hydrology for Small Watersheds, Technical Release No. 55* (TR-55), 2003
  - *National Engineering Handbook*, 1997

RECOMMENDED INTERNET SITES

- DelDOT's Web Site (http://www.deldot.gov/) - Design Resource Center
- FHWA Hydraulics Engineering (http://www.fhwa.dot.gov/engineering/hydraulics)
- New Jersey Department of Transportation (http://www.state.nj.us/transportation/eng/)
- University of Wisconsin Cooperative Extension (http://learningstore.uwex.edu/) - Wisconsin Storm Water Manual: Grassed Swales (G3691-7)
- US Army Corps of Engineers Hydrologic Engineering Center (HEC) (http://www.hec.usace.army.mil)

- Virginia Department of Transportation (http://www.virginiadot.org) - Drainage Manual

RECOMMENDED SOFTWARE

- HY7 WSPRO (2001) - Gradually varied flow in open channels
- HY8 Culvert Analysis - Methodology and data found in HDS-5, HEC-14 and HEC-15
- HY22 Urban Drainage Design Program - Highway pavement drainage, open channel flow, critical depth solutions, storm stage-storage computations and reservoir routing
- TR-55 Urban Hydrology for Small Watersheds
- TR-20
- WSP-2

HYDROLOGY

REFERENCES

- FHWA
  - HDS-2 *Highway Hydrology* (FHWA-NHI-02-001)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
  - HEC-19 *Hydrology*, 1985 (replaced by HDS-2)
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)
- NRCS - TR-55 *Urban Hydrology for Small Watersheds, Technical Release No. 55*, 2003

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

DelDOT's Criteria for Design Frequency ..... DelDOT Figure 6-1
Frequency Factor, C_f ..... Section 3.2.2.1, HEC-22
Delaware's Rainfall Intensity Estimates ..... DelDOT Figures 6-5 thru 6-7
Runoff Coefficients ..... DelDOT Figure 6-8; Table 5.7, HDS-2
Appendix B, Tables 11 and 12, HDS-4; Table 3-1, HEC-22
Velocities for Upland Method of Estimating T_{t1} and T_{t2} ..... Figures 37 and 52, HEC-19
Figure 3-1, TR-55; USDA Hydrology Technical Note N4, Time of Concentration (1986)
Soil Group Descriptions for TR-55 Method ..... DelDOT Figure 6-9
Hydrologic Soils Descriptions for TR-55 Method Common to Delaware ..... DelDOT Figure 6-10
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Adjustment Factor (F_p) - Pond & Swamp Areas Spread In Watershed ..... Table 5.6, HDS-2
Table 3-9, HEC-22; Table 4-2, TR-55

OPEN CHANNEL FLOW AND ROADSIDE DITCHES

REFERENCES

- FHWA
  - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)

- HEC-11 *Design of Riprap Revetment* (FHWA-IP-89-016)
- HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
- HEC-15 *Design of Roadside Channels with Flexible Lining* (FHWA-IF-05-114)
- HEC-22 *Urban Drainage Manual*, 2001 (FHWA-NHI-01-021)

- *Open-Channel Hydraulics*, Ven Te Chow, 1959

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Capacity of Trapezoidal Channels ..... HDS-3 and HDS-4
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Maximum Nonscouring Ditch Grades with Grass Lining ..... DelDOT Figure 6-13; Tables 2-3, HDS-3
DelDOT's Roadside Ditch Design Form ..... DelDOT Attachment B
Permissible Velocities for Open Channels ..... DelDOT Figures 6-14 and 6-15
K_b Factor for Maximum Stress on Channel Bends ..... Chart 21, HEC-22
Correction Factor for Riprap Size ..... Chart 2, HEC-11
Riprap Size Relationship ..... Chart 1, HEC-11

PAVEMENT DRAINAGE AND STORM DRAINS

REFERENCES

- FHWA
  - HDS-3 *Design Charts for Open-Channel Flow* (FHWA-EPD-86-102)
  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
  - HEC-12 *Drainage of Highway Pavements*, 1984 (replaced by HEC-22)
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Manning's Roughness Coefficients (n) for Pavements and Gutters HEC-22
Allowable Water Spread ..... DelDOT Figure 6-2
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Flow in Composite Gutter Sections ..... Chart 5, HEC-12
Gutter Flow Rate *Q* on Uniform Cross Slope (ft^3/s) ..... DelDOT Web Site

Gutter Flow Q on Composite Section (ft^3/s) ..... DelDOT Web Site
Average Gutter Flow Velocity for Uniform and Composite Sections ..... DelDOT Web Site
Spread at Average Velocity in a Reach of Triangular Gutter ..... Table 4-4, HEC-22
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Depressed Curb Opening Inlet Capacity in Sump Conditions ..... Chart 12, HEC-12
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Storm Drain Computation Table (Open Channel) .... DelDOT Attachment B; Appendix D, HEC-22
Hydraulic Gradeline Computation Table (Pressure Flow Design) ..... DelDOT Attachment B
Appendix D, HEC-22

PIPES AND PIPE CULVERTS
REFERENCES

- FHWA
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  - HDS-4 *Introduction to Highway Hydraulics* (FHWA-NHI-01-019)
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  - HEC-13 *Hydraulic Design of Improved Inlets for Culverts*, 1972
  - HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)

In addition to the above references there are manuals available from various pipe manufacturers, including the *Concrete Pipe Design Manual (CPDM)* (viewable online at http://www.concrete-pipe.org/techdata.htm) and the *Handbook of Concrete Culvert Pipe Hydraulics* by the American Concrete Pipe Association; and *Modern Storm Sewer Design* by the American Iron and Steel Institute. These give pipe size data, general hydrology and hydraulic design, structural design, durability, and recommended installation methodology. Much of this information is not available in other publications.

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Energy Concept in Pipe Flow ..... Figure III-8 and III-9, HDS-5; Figure 7-8, HEC-22
Manning's Roughness Coefficients *n* for Pipes ..... DelDOT Figure 6-27; Table 14, HDS-4
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Charts 26 and 27, HEC-22
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Flow for Circular Pipe Flowing Full ..... Figures 2 thru 5, *CPDM*
Flow for Horizontal Elliptical Pipe Flowing Full ..... Figures 6 thru 9, *CPDM*
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Solution of Manning's Equation for Flow in Storm Drains ..... Chart 25B, HEC-22
Flow for Circular Pipe Flowing Full ..... *CPDM*
Flow for Horizontal Elliptical Pipe Flowing Full ..... *CPDM*
Flow Coefficients for Circular Pipe ..... *CPDM*
Uniform Flow for Pipe Culverts ..... Del DOT Web Site
Uniform Flow for Concrete Elliptical Pipes ..... Del DOT Web Site
Critical Depth of Flow for Circular Conduits ..... Chart 4B, HDS-5; Chart 27B, HEC-22
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Headwater Depth for C.M. Pipe Culverts with Inlet Control ..... Chart 2B, HDS-5
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Headwater Depth for Oval Concrete Pipe Culverts with Inlet Control ..... Chart 29B, HDS-5
Head for Elliptical Concrete Pipe Culverts Flowing Full ..... Chart 33B, HDS-5
Allowable Fill Heights for RCP Pipe ..... DelDOT Web Site
Allowable Fill Heights for Round Corrugated Steel Pipe ..... DelDOT Web Site
Allowable Fill Heights for Pipe Arch Corrugated Steel Pipe ..... DelDOT Web Site
Allowable Fill Heights for Round Corrugated Aluminum Pipe ..... DelDOT Web Site
Allowable Fill Heights for Pipe Arch Corrugated Aluminum Pipe ..... DelDOT Web Site
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Elliptical Concrete Pipe Characteristics ..... Figure 7-2, HEC-22

Minimum Slopes for Full Flow in Pipes at 3 ft/s ..... DelDOT Figure 6-24; Table 7-7, HEC-22
Friction Slope for Concrete — Full Flow ..... DelDOT Figure 6-29
Circular Pipe Conveyance ..... DelDOT Web Site
Flow Coefficients for Circular Pipe ..... DelDOT Web Site

STORMWATER QUANTITY MANAGEMENT - DETENTION PONDS

REFERENCES

- DNREC
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  - *DURMM: The Delaware Urban Runoff Management Model*
    - *Technical Manual - Green Technology: The Delaware Urban Runoff Management Approach*
    - *User's Manual*
    - *Computer Program*
- DelDOT - *ES_2M Design Guide*
- FHWA
  - HDS-2 *Highway Hydrology* (FHWA-NHI-02-001)
  - HEC-11 *Design of Riprap Revetment* (FHWA-IP-89-016)
  - HEC-14 *Hydraulic Design of Energy Dissipaters for Culverts and Channels*, 1975
  - HEC-15 *Design of Roadside Channels with Flexible Lining* (FHWA-IF-05-114)
  - HEC-19 *Hydrology*, 1985 (replaced by HDS-2)
  - HEC-22 *Urban Drainage Manual* , 2001(FHWA-NHI-01-021)
- NRCS
  - *Engineering Field Handbook*, 1984
  - *National Engineering Handbook*, 1997
  - Pond Code 378
  - *Urban Hydrology for Small Watersheds, Technical Release No. 55* (TR-55), 2003
- *Open-Channel Hydraulics*, Ven Te Chow (1959)
- *City of Tacoma Surface Water Management Manual*
- *Low Impact Development——Technical Guidance Manual for Puget Sound*
- Various Internet sites
- *TP-61 Handbook of Channel Design for Soil & Water Conservation*
- City of Tacoma Surface Water Management Manual Volume V Runoff Treatment BMP's
- "Sand Filter Design for Water Quality Treatment" Shaver, E., and Baldwin, R., 1991, Stormwater Conference Proceedings, Mt. Crested Butte, CO, ASCE, Washington, D.C. (Also see "A Catalog of Ultra Urban Best Management Practices Using Sand Filters" Bell, W., 1992, City of Alexandria, Alexandria, VA.)
- Grass Swales: "Biofiltration Systems for Storm Runoff Water Quality Control" Horner, R., 1988, Washington State Department of Ecology, Seattle, WA.
- Website link - http://lowimpactdevelopment.org

DESIGN AIDES: FIGURES, NOMOGRAPHS AND TABLES

Design Steps for a Wet Extended Detention Stormwater Pond ..... DelDOT Figure 6-30
Degree of Retardance ..... Table 5, HDS-3

Chapter Six

ATTACHMENT B
EXAMPLE PROBLEMS

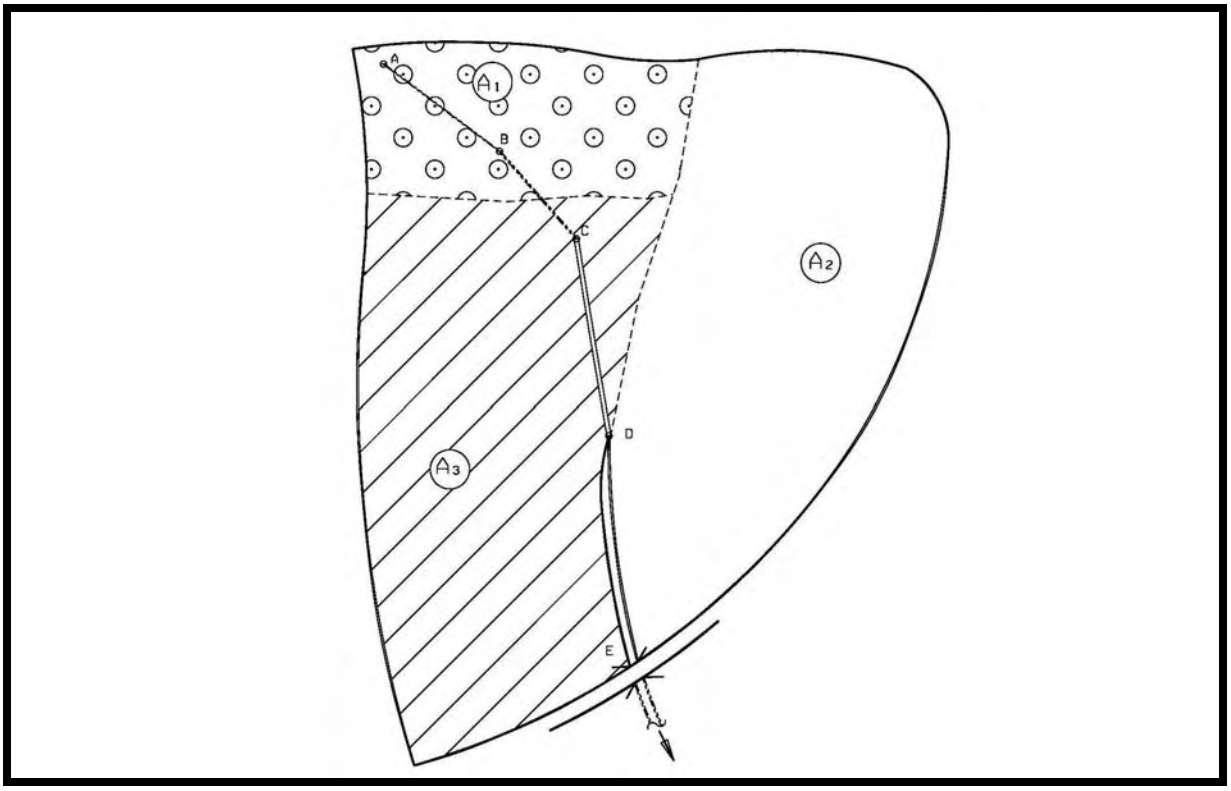
Introduction: These example problems are being presented to show the basic steps for drainage design. It is expected that the designer will use current available software and spreadsheets as much as possible to facilitate drainage calculations. To ensure compliance with the criteria and expedite review, the designer should ensure that all final calculations, size selections, water spread, etc., are compared with the Department's approved drainage criteria and are highlighted in the summary and/or within the drainage report itself.

EXAMPLE 1—RATIONAL METHOD

PROBLEM STATEMENT

The watershed area in Figure 6B-1 has a composite watershed of 36.5 acres and drains to a culvert at E on a local road in Kent County. The surface characteristics of the watershed and the stream flow path are as shown in figures 6B-2 and 6B-3. The design problem is to determine the peak discharge at point E for a 25-yr storm frequency using the Rational Method.

Figure 6B-1
Watershed for Rational Method Example



Solution

Step 1. Determine t_c

Figure 6B-2
Flow Path of Watershed

Segment	Description	Slope %	Length (ft)
A-B	Residential lawn	1.0	50
B-C	Forest and meadow	2.0	50
C-D	Shallow gully	0.8	250
D-E	Open channel	0.8	475

Compute the sheet flow time, T_{t1}***, for segments A-B and B-C.

Use equation 6.4:

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}^{\ 0.5} s^{0.4}}$$

where:

Segment A-B
n = 0.15 from Table 3-2, HEC-22
L_1 = 50 ft
P_{24} = 3.3 from Figure 6-11
s = 0.01

Segment B-C
n = 0.30 from Table 3-2, HEC-22
L_1 = 50 ft
P_{24} = 3.3 from Figure 6-11
s = 0.02

$$T_{t1} = \frac{0.42[(0.15)(50)]^{0.8}}{(3.3)^{0.5}(0.01)^{0.4}} + \frac{0.42[(0.30)(50)]^{0.8}}{(3.3)^{0.5}(0.02)^{0.4}} = 7.31 + 9.65 = 16.96 \text{ min}$$

*Note: A great deal of the literature indicates that sheet flow should be limited to 100 ft.

Compute the shallow concentrated flow time, T_{t2} for segment C-D. Use the Shallow (Gutter) Gully Flow Line of Figure 6B-2 and 0.8% watercourse slope. From Figure 37, HEC-19, V_2 = 1.35 f/s. Using equation 6.5 and V_2 = 1.35 ft/s

$$T_{t2} = \frac{L_2}{60 V_2} = \frac{250}{60 \ x \ 1.35} = 3.08 \ minutes$$

Compute the open channel flow time, T_{t3}, for segment D-E. From survey data, the small stream has a base width (*B*) of 3 ft with 4:1 side slopes (*z* = 4), banks approximately 2 ft high (d), and a streambed slope of 0.008 ft/ft. This meandering channel has some weeds with light brush. Therefore, *n* = 0.045 (Table 1, HDS-3, or Table 3-4, HEC-22).

The streambed cross sectional area (*A*) and wetted perimeter (*P*) from Figure 5-6, HEC-22:

$$A = 3 \ x \ 2 + (4 \ x \ 2^2) = 22 \ ft^2$$

$$P = 3 + 2 \ x \ 2\sqrt{4^2 + 1} = 19.49 \ ft$$

Therefore using equation 6.7a:

$$R = \frac{A}{Pw} = \frac{22}{19.49} = 1.13 \ ft$$

With s = 0.008, *n* = 0.045 and *R* = 1.13, then V_3 = 3.21 ft/s from Chart 83, HDS 3, or equation 6.7:

$$V = \frac{1.49}{n} R^{0.67} s^{0.5}$$

$$V_3 = \frac{1.49}{0.045}(1.13)^{0.67}(0.008)^{0.5} = 3.21$$

$$T_{t3} = \frac{L_3}{60V_3} = \frac{475}{60x3.21} = 2.47 \text{ min}$$

Using equation 6.8:

$t_c = T_1 + T_2 = T_3 = 16.96 + 2.31 + 2.47 =$ 21.74 rounded to 22 min

Step 2. Determine I

Using Figure 6-6, for t_c=22 minutes (duration) and 25-yr frequency, I =4.06 in/hr.

Step 3. Determine C_f

From HEC-22, C_f=1.10 for a 25-yr frequency.

Step 4. Determine the weighted average runoff coefficient using Figure 6B-2, Equation 6.2 and runoff coefficients for the respective ground characteristics (Figure 6-8).

$$C = \frac{\sum C_x A_x}{A_{total}}$$

$$C = \frac{0.35 \text{ x } 6.0 + 0.40 \text{ x } 16.9 + 0.25 \text{ x } 13.6}{6.0 + 16.9 + 13.6}$$

C = 0.34

Figure 6B-3
Surface Characteristics of Watershed*

Area (ac)	Description	Runoff Coefficient
A_1 = 6.0	Residential, single family, large lots	0.35
A_2 = 16.9	Residential, single family, small lots	0.40
A_3 = 13.6	Woods, light vegetation, loam soil	0.25

*Assumed for this example.

Step 5 Calculate Q_{25}, peak discharge, by using Equation 6.1.

$$Q_{25} = C_f (CIA) = 1.10x0.34 \text{ x } 4.46x36.5$$

= 60.9, rounded to 61 ft^3/s

Useful formulas for various channel shapes are as follows. (Also see Appendix B, HEC-15 or Figure 5-6, HEC-22). z is dimensionless and represents the horizontal ditch slope rate, for a 2:1 slope z = 2; 3:1 slope z=3.

V-Shape

Symmetrical

$$A = zd^2$$

$$P = 2d\sqrt{z^2 + 1}$$

Unsymmetrical

$$A = \frac{(z_1 + z_2)d^2}{2}$$

$$P = \left(\sqrt{z_1^2 + 1} + \sqrt{z_1^2 + 1}\right)d$$

Trapezoidal

Symmetrical

$$A = Bd + zd^2$$

$$P = B + 2d\sqrt{z^2 + 1}$$

Unsymmetrical

$$A = Bd + \frac{(z_1 + z_2)d^2}{2}$$

$$P = B + d\left(\sqrt{z_1^2 + 1} + \sqrt{z_2^2 + 1}\right)$$

Parabolic

$$A = \frac{2}{3}Td$$

$$P = T + \frac{8d^2}{3T}$$

Approximation $0 < d/T \leq 0.25$

For $d/T \geq 0.25$ use:

$$P = \frac{1}{2}\sqrt{16^2 + T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$$

$$T = 1.5\frac{A}{d}$$

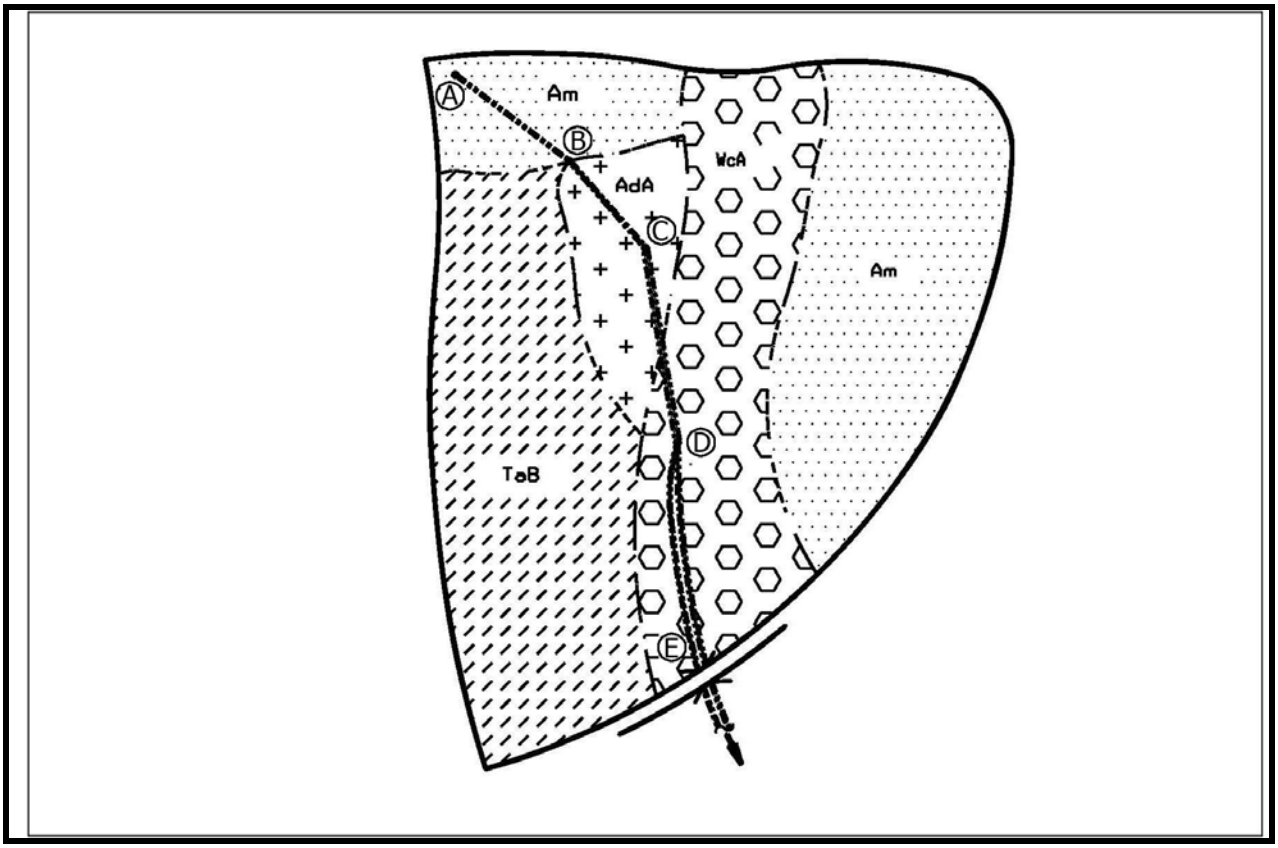
EXAMPLE 2─NRCS TR-55 METHOD

PROBLEM STATEMENT

The watershed area shown in Figure 6B-4 has a composite watershed area of 36.5 acres draining into a culvert at E on a local road in New Castle County. The design frequency is 25-yr. Estimate the peak discharge at point E using the NRCS TR-55 Graphical Peak Discharge method.

The designer should be aware that when using the TR-55 Method the methodology, variables and approach to solving this problem is exclusive to this method. Mixing of variables, runoff coefficients etc. from the Rational Method will lead to inaccurate results.

Figure 6B-4
Watershed for NCRS Method Example



SOLUTION

For this problem since the watershed has homogenous areas and has no stormwater management facilities requiring analysis of several storm frequencies and routing hydrographs, the Graphical Peak Discharge Method is applicable.

The peak discharge equation 6.14 is:

$$Q = q_u A_m q_d F_p$$

First, the runoff depth (q_d) is calculated, second, the time of concentration is calculated and used to obtain the unit peak discharge,(q_u), a pond and then a swamp factor (F_p) is selected and then Q determined.

Step 1. Obtain the 24-hr rainfall for the 25-yr frequency from Figure 6-11. P_{24} = 6.0.

Step 2. Establish hydrologic soil groups for the composite watershed and obtain CN_T.

The respective hydrologic soil groups were obtained from the Soils Maps for New Castle County, the curve numbers were obtained from Figure 6B-5 based on the land use, and the weighted average curve number is computed using the data from Figure 6B-5 and the equation:

$$\overline{CN} = \frac{\sum (CN_m \times A_m)}{\sum A_m}$$

The results are:

$$\overline{CN} = \frac{2696.75}{36.50} = 73.88 \text{, Rounded to 74}$$

Since impervious areas exist only on residential areas, and the CN values for residential areas, depending on their sizes, have accounted for these impervious areas. Therefore, for this example CN_T = 74

Figure 6B-5
Watershed Data

Soil Type	Hydrologic Soil Group	Land Use	CN	Area (ac)
Am	C	Residential - 1 Ac	79	2.50
Am	C	Residential - 1/2 Ac	80	12.60
AdA	C	Residential - 1/4 Ac	83	0.75
AdA	C	Woods (Good hydrological Conditions)	70	1.10
TaB2	B	Residential - 1/4 Ac	75	1.00
TaB2	B	Woods (Good hydrological Conditions)	55	9.00
WcA	D	Residential - 1 Ac	84	1.75
WcA	D	Residential - 1/2 Ac	85	4.30
WcA	D	Woods (Good hydrological Conditions)	77	3.50

Other than the given information all the other data is the same as in Example 1.

Step 3. Calculate Runoff Depth q_d from Equations 6.9 and 6.10.

$$q_d = \frac{(P_{24} - 0.2S))^2}{P_{24} + 0.8S}$$

$$S = \frac{1000}{CN} - 10$$

$$q_d = \frac{\left[6.0 - 0.2\left(\frac{1000}{74} - 10\right)\right]^2}{6.0 + 0.8\left(\frac{1000}{74} - 10\right)} = 3.2 \text{ in}$$

Step 4. Calculate t_c Note that TR-55 uses hours when calculating and using t_c.

Overland (Sheet) Flow (Two segments: A-B & B-C) Refer to the topographic data on Figure 6B-6 for T_{t1}, using equation 6.11:

$$T_{t1} = \frac{0.007(nL_1)^{0.8}}{P_2^{0.5} s^{0.4}}$$

Manning's runoff coefficient n in this equation is selected from Table 3-1, TR-55. These coefficients have been developed exclusively for use when using the TR-55 method for sheet flow time of concentration.

For segment A-B, n = 0.24 for lawn from Table 3-1, TR-55

For segment B-C, n = 0.40 for woods (light underbrush) from Table 3-1, TR-55.

Figure 6B-6
Flow Path of Watershed

Segment	Description	Slope %	Length (ft)
A-B	Residential lawn	1.0	50
B-C	Forest and meadow	2.0	50
C-D	Shallow gully	0.8	250
D-E	Open channel	0.8	475

Therefore:

$$T_{t1} = \frac{0.007(0.24\ x\ 50)^{0.8}}{(3.2)^{0.5}(0.01)^{0.4}}$$

$$+ \frac{0.007(0.40\ x\ 50)^{0.8}}{(3.2)^{0.5}(0.02)^{0.4}} = 0.34hr$$

Shallow Concentrated Flow (C-D)

TR-55 has three equations that are acceptable for determining T_{t2}. The designer, after field reviewing the watershed, may decide that using the equation, $V_2 = 32.8ks^{0.5}$ and assigning a k factor from Figure 6-4 will give the most realistic velocity value. In this case:

V_2 = 32.8(0.457) $(0.008)^{0.5}$ = 1.34 ft/s

The other two equations develop an average k for unpaved areas resulting in equation 6.12:

Unpaved $\quad V_2 = 16.10(s)^{0.5}$

$$V_2 = 16.10(0.008)^{0.5} = 1.44\ ft/s$$

Select the higher of the velocities, since it will result in a more conservative peak discharge, to find T_{t2} and use equation 6.5 modified to calculate hours:

$$T_{t2} = \frac{L_2}{3600V_2} = \frac{250}{3600\ x\ 1.44} = 0.05\ min$$

Open Channel Flow

The travel time for the open channel segment calculation involves the same procedure as in the Rational Method. From Example 1, T_{t3} = 2.47 minute or 0.04 hr.

Therefore, from equation 6.8:

$$t_c = T_{t1} + T_{t2} + T_{t3}$$

$$t_c = 0.34 + 0.05 + 0.04 = 0.43hr$$

<u>Step 5.</u> Determine Unit Peak Discharge q_u from equation 6.15 First, determine I_a/P using equation 6.10 and that I_a is assumed to be 20% of S:

$$I_a/P = \left(\frac{200}{CN_T} - 2\right) \Big/ P_{24} = \left(\frac{200}{74} - 2\right) \Big/ 6.0$$

$$= 0.12$$

I_a/P can also be obtained from Table 4-I, TR-55.

This value and t_c are used to obtain the value of q_u from Exhibit 4-II, TR-55, by

interpolating between the curves for $I_a/P = 0.12$

$$q_u = 575\left(ft^3 s / mi^2 / in\right)$$

Step 6. Obtain Pond Adjustment Factor, F_p.

Since the percent of pond and swamp area is zero for the watershed, F_p = 1.0 from Table 5.6, HDS-2, Table 3-9 HEC-22, or Table 4-2, TR-55.

Step 7. Calculate Q from equation 6.14:

$$Q_i = q_u A_m q_d F_p$$

A_m = Drainage Area = 36.5/640 = 0.057 square miles. Therefore, the peak discharge is:

Q = 575 x 0.057 x 3.20 x 1.00

= 105 ft^3/s

Exhibit 4-II, which is not easily or accurately read, is based on the equation:

$$\log(q_u) = C_o + C_1 \log(t_c) + C_2 \left[\log(t_c)\right]^2$$

C_o, C_1, C_2 are coefficients derived from interpolation of Table F-1, TR-55.

Figure 6B-7
Coefficients for Unit Peak Discharge

I_a/P	C_o	C_1	C_2
0.10	2.55323	-0.61512	-0.16403
0.30	2.46532	-0.62257	-0.11657
0.35	2.41896	-0.61594	-0.08820
0.40	2.36409	-0.59857	-0.05621
0.45	2.29238	-0.57005	-0.02281
0.50	3.20282	-0.51599	-0.01259

The designer may find it simpler to use the equation and coefficients.

EXAMPLE 3—OPEN CHANNEL HYDRAULICS

PROBLEM STATEMENT

A grass-lined, unsymmetrical trapezoidal channel with a base width of 4 feet, a 4:1 foreslope, a 2:1 backslope, a stream bed slope of 0.012 ft/ft (1.2%), and n = 0.05, carries a design discharge of 115 ft^3/s. Determine the depth of flow, velocity, and if the flow is supercritical. There are charts and nomographs available in HDS-3, HEC-15 (1986 or 1988 version) and HEC-22 that can be used to quickly solve for these two values.

SOLUTION

Using the channel equations given at the end of Example 1, the depth of flow (d) will be obtained by trial and error.

Trial 1.

Try d = 2 ft

$$A = Bd + \frac{(z_1 + z_2)d^2}{2}$$

$$A = (4x2) + \frac{(2+4)2^2}{2} = 20\,ft^2$$

$$P = B + d\left(\sqrt{z_1^2 + 1} + \sqrt{z_2^2 + 1}\right)$$

$$P = 4 + 2\left(\sqrt{2^2 + 1} + \sqrt{4^2 + 1}\right)$$

$$P = 4 + 2(6.359) = 16.718\,ft$$

$$R = A/P = \frac{20}{16.718} = 1.196\,ft$$

(equation 6.7a)

$$V = \frac{1.49}{n} R^{2/3} s^{1/2}$$

$$= \frac{1.49}{0.05}(1.196)^{2/3}(0.012)^{1/2} = 3.58\,ft/s$$

(equation 6.7)

$$Q = AV = 20x3.58 = 73.6\,ft^3/s$$

(too low for 115 ft^3/s)

Trial 2.

Try d = 2.5 feet

$$A = (4x2.5) + \frac{(4+2)x2.5^2}{2} = 28.75\,ft^2$$

$$P = 4 + (6.359x2.5) = 19.898\,ft$$

$$R = A/P = 1.445\,ft$$

$$V = \frac{1.49}{0.05}(1.445)^{2/3}(0.012)^{1/2} = 4.17\,ft/s$$

$$Q = 28.75x4.16 = 119.60\,ft^3/s$$

(greater than 115 ft^3/s)

Trial 3.

Try d = 2.45 feet

$$A = (4x2.45) + \frac{(4+2)2.45^2}{2} = 27.81\,ft^2$$

$$P = 4 + (6.359x2.45) = 19.58\,ft$$

$$R = A/P = 1.42\,ft$$

$$V = \frac{1.49}{0.05}(1.42)^{2/3}(0.012)^{1/2} = 4.125\,ft/s$$

$$Q = 27.81x4.125 = 114.71\,ft^3/s$$

≈115 ft^3/s)

Therefore:

Depth of flow d = 2.45 ft and

V = 4.12 ft/s

Instead of performing the calculations, Chart 1, HEC-15, or Chart 22, HEC-22, could have been used. From these charts and using:

$$\frac{d}{B} = \frac{2.45}{4} = 0.6125 \text{ and, } z_1 + z_2 = 6.0$$

R/d is estimated at 0.58.

Therefore:

R = 0.58 x 2.45 = 1.421ft.

A/Bd = 27.81/ (4) (2.45) = 2.84, and

A = 2.84 x 2.45 x 4 = 27.83 ft^2.

The designer can also use Chart 83, HDS-3, or Chart 14B, HEC-22, to solve Manning's equation, connect the slope-scale at 0.012 with the *n*-scale of 0.05 then draw a line passing through R = 1.42 and the point of intersection of s and n at the turning line, and extend it to the velocity scale which is read at 4.15 ft/s. Therefore, Q = 27.44 x 4.15 = 113.88 ft^3/s which is close to 115 ft^3/s, and the assumed d = 2.45 ft is O.K.

For example, assume $z_1 + z_2 = 3$. On Chart 3, HEC-15 (May 1986), join s = 0.05 with Qn = 115 x 0.05 = 5.75 and extend to the turning line. Connect that point of intersection with B = 4 and let the line intersect z = 0 from there proceed horizontally until z = 3. That point of intersection corresponds to d/B = 0.615 from which d = 4 x 0.615 = 2.46ft.

Froude Number:

$$F_r = \frac{V}{\sqrt{gD}}$$

$$D = \frac{A}{T} = \frac{27.81}{6x2.45 + 4} = 1.49\,ft$$

$$F_r = \frac{4.12}{\sqrt{32.2x1.49}} = 0.60 < 1$$

Therefore, the flow is subcritical.

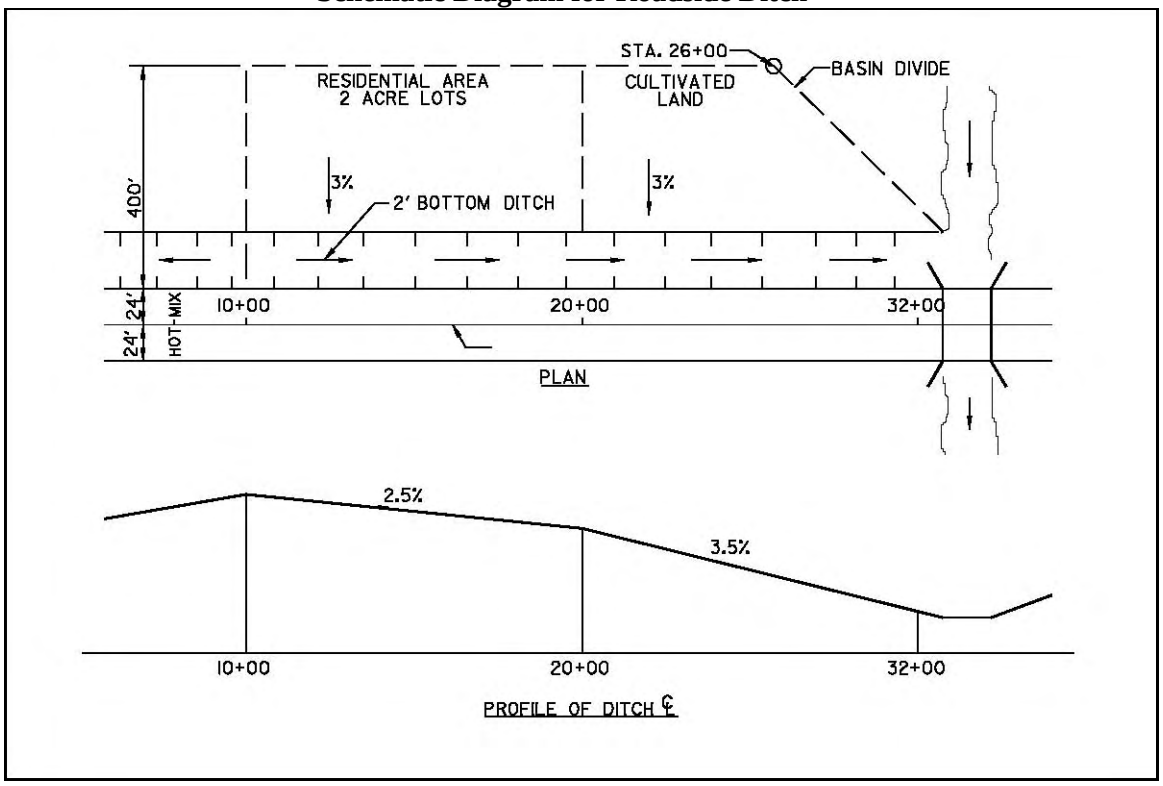
EXAMPLE 4—ROADSIDE DITCH

PROBLEM STATEMENT

Figure 6B-8 shows a roadside ditch along a Sussex County rural collector with trapezoidal cross section, 2-foot bottom width, and 4:1 side slopes. The ditch lining is average grass in sandy loam.

Compute the velocities of flow along the ditch, and determine if additional erosion protection is necessary. From Figure 6-1, the design frequency shall be for a 10-yr period.

Figure 6B-8
Schematic Diagram for Roadside Ditch



SOLUTION

Enter the following critical stations on a Roadside Ditch Design computation form:

Station 20 + 00 (Back)

Station 20 + 00 (Ahead)

Station 32 + 00 (Outfall Point)

Leave blank spaces for intermediate stations at this time. The calculations proceed as follows. The sequence numbers for each station pertain to the respective columns on the computation sheet.

<u>Station 20+00 (Back)</u>

Step 1. From Sta. 10+00 to Sta. 20+00,

- Pervious Area = (400)(1000)/43560 = 9.18 ac.
- Impervious Area = (24)(1000)/43560 = 0.55 ac.
- Total Area = 9.18 + 0.55 = 9.73 ac

Step 2. Select C values from Figure 6-8.

- Pervious: For residential areas with 2-acre lots, use C = 0.40.

- Impervious: For asphalt pavement (hot-mix roadway & shoulder), C = 0.95.

Step 3. Calculate total CA

Use $CA = C_pA_p + C_IA_I$

CA = (0.40)(9.18) + (0.95)(0.55) = 4.19

Step 4. Determine t_c

(a) Since the watershed is homogenous and not complex, the Rational Method can be used to find to the find the discharge flow. Therefore, use equation 6.4:

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}^{\ 0.5} s^{0.4}}$$

where:

n = 0.40 from Figure 6-16
L_1 = 100 ft
P_{24} = 3.4 in/hr from figure 6-11
s = 0.03

Therefore:

$$T_{t1} = \frac{0.42[(0.40)(100)]^{0.8}}{(3.4)^{0.5}(0.03)^{0.4}}$$

= 17.71 min

The remaining 300 ft is to treated as shallow concentrated flow using equations 6.5 and 6.6:

$$T_{t2} = \frac{L_2}{60v_2}$$

$$V_2 = 32.8ks^{0.5}$$

The latter equation has been modified into the condition for unpaved areas in equation 6.12:

Unpaved $V_2 = 16.10(s)^{0.5}$

Therefore:

$$V_2 = 16.10(0.03)^{0.5} = 2.79\ ft/s$$

$$T_{t2} = \frac{300}{60\ x\ 2.79} = 1.79 min$$

(b) For 1000 feet open channel flow, T_{t3}, assume maximum permissible velocity of 4.0 ft/s for grass on sandy loam from Figure 6-15.

Therefore:

$$T_{t3} = \frac{1000}{60\ x\ 4} = 4.17\ min$$

and, using equation 6.8:

$$t_c = T_{t1} + T_{t2} + T_{t3}$$

= 17.71+1.79+4.17 = 23.67 min

Step 5. Find I from Figure 6-7 for the 10-year frequency and t_c = 23.67 minutes

I = 3.83 in/hr (interpolating between 15 and 30 min)

Step 6. Calculate Q using equation 6.1:

$Q = C_f CIA = C_f (CA) I$

Q = 1.00 x 4.19 x 3.83 = 16.05, rounded to 16 ft^3/s

Step 7. Enter ditch cross sectional data

z_1 = 4, B = 2 feet, z_2 = 4, and S = 0.025

Step 8. Select Manning's n

n = 0.05 for grass from Figure 6-16

Step 9. Calculate d

Use Chart 3, HEC-15 (1986), or Chart 14B, HEC-22. For s = 0.025, Qn = 16 x 0.05 = 0.8, and B = 2, d/B reads 0.45 when z = 4. Thus d = 0.45 x 2 = 0.9 feet.

Step 10. Calculate Average Velocity

Use $V = Q/A$

$$A = dB + zd^2 = 0.9\ x\ 2.0 + 4\ x\ 0.9^2 = 5.04\ ft^2$$

$$V = \frac{16}{5.04} = 3.18\ ft/s$$

NOTE: Check the computed velocity against the assumed velocity used to determine Step 4. If the velocities differ by more than 20%, repeat Steps 4 through 9 using the velocity calculated in Step 10. At this location, the velocities differ by ≈14%.

Step 11. Check the computed velocity in Step 10 against the permissible velocity from Figures 6-13 through 6-15. If the computed velocity is greater than the maximum permissible velocity, a stronger protective lining is required.

Step 12. In this case, V = 3.18 ft/s and V_{max} = 4 ft/s. Since $V<V_{max}$ for grass, specify grass.

Step 13. Grass must cover the entire ditch. From a practical consideration, grass should cover the entire area within the limits of consideration.

Station 20+00 (Ahead)

Based on the allowable velocity for a grassed lined channel, the calculated velocity at Station 20+00 and the increase in ditch slope after Station 20+00, it is likely that after Station 20+00 and before the outfall that; (1) some change in the type of ditch lining may need to be made; (2) an increase in the ditch cross section or; (3) some type of velocity dissipation structure(s) will be needed.

To find the point at which a lining change may be advisable and since excessive outfall velocities have to be considered in the design at all locations, a good point of reference would be to next calculate the maximum expected velocity at the outfall based on the conditions as presented in the problem. Knowing this value will give an indication of the maximum erosion potential.

After determining this value iterations of the process would begin at Station 20+00 and proceed toward the outfall and determine the location for making a decision on addressing erosion. Using a form similar to that included in this problem will allow the designer to keep the data in a systematic manner.

Station 32+00 (Outfall Point)

Step 1. From Sta. 20+00 to Sta. 32+00,

Pervious Area

$$A_p = \frac{(600)(400) + 1/2(600)(400)}{43560}$$

= 8.26 ac

$$A_{Ptotal} = 9.18 + 8.26 = 17.44ac$$

Impervious Area

$$A_{I\,totsl} = \frac{(24)(2200)}{43560} = 1.21ac$$

Step 2. Select C values from Figure 6-8.

Pervious C from Sta. 10+00 to Sta. 20+00 = 0.40 (For residential lots)

Pervious C from Sta. 20+00 to Sta. 32+00 = 0.30 (For cultivated land from Figure 6-8)

Weighted Average Pervious C from equation 6.2*:*

$$C_p = \frac{(0.4)(9.18) + (0.30)(8.26)}{17.44} = 0.35$$

Impervious C from Sta. 10+00 to Sta. 32+00

$C_I = 0.95$

Step 3. Find total CA

Use the equation $CA = Cp\,Ap + C_I A_I$

CA = (0.35)(17.44) + (0.95)(1.21) = 7.25

Step 4. Determine t_c

Find T_{t1} (equation 6.4) and T_{t2} (equation 6.5) for the cultivated area.

$$T_{t1} = \frac{0.42(nL_1)^{0.8}}{P_{24}^{\;0.5} s^{0.4}}$$

where:

n = 0.17 from Table 3-2, HEC-22
L_1 = 100 ft
P_{24} = 3.4 in/hr from Figure 6-11
s = 0.03

Therefore:

$$T_{t1} = \frac{0.42[(0.17)(100)]^{0.8}}{(3.4)^{0.5}(0.03)^{0.4}}$$

= 8.93 min

The remaining 300 ft is to treated as shallow concentrated flow using the following equations 6.5 and 6.6:

$$T_{t2} = \frac{L_2}{60v_2} \text{ and,}$$

$$V_2 = 32.8ks^{0.5}$$

This equation has been modified for unpaved areas in equation 6.12:

Unpaved $V_2 = 16.10(s)^{0.5}$

Therefore:

$$V_2 = 16.10(0.03)^{0.5} = 2.79\ ft/s$$

$$T_{t2} = \frac{300}{60\ x\ 2.79} = 1.79\ min$$

Assuming a grass lining from Sta. 20+00 to Sta. 32+00 (1200 feet open channel flow) for which the maximum permissible velocity is 4 ft/s (Figure 6-15), the travel time is:

1200/(60 x 4) = 5 min

Add this to the travel time at Sta. 20+00, which is 23.67 min.

Thus using equation 6.8,

t_c = 23.67+5+8.93+1.79 = 39.39 min.

Step 5. From Figure 6-7, I = 2.94 in/hr.

Step 6. Calculate Q as before from equation 6.1

Use $Q = (CA)\ I$

Q = (7.25) (2.94) = 21.32, rounded to 21 ft^3/s

Step 7. Enter ditch cross sectional data

z_1 = 4, B = 2 feet, z_2 = 4, and s = 0.025

Step 8. Select Manning's n

n = 0.05 for grass from Figure 6-16

Step 9. Calculate d

Use Chart 3, HEC-15 (1986), or Chart 14B, HEC-22.

For s = 0.025, Qn = 21 x 0.05 = 1.05, and B = 2, d/B reads 0.51 when z = 4

Thus d = 0.51 x 2 = 1.02 feet

Step 10. Calculate Average Velocity

Use $V = Q/A$

$$A = dB + zd^2 = 1.02x\ 2.0 + 4\ x\ 1.02^2 = 6.20\ ft^2$$

$$V = \frac{21}{6.20} = 3.39\ ft/s$$

NOTE: Check the computed velocity against assumed velocity used to determine Step 4. If the velocities differ by more than 20%, repeat Steps 4 through 9 using the velocity calculated in Step 10. At this location, the velocities differ by about 22%. The second iteration yields the results in Figure 6B-9.

If the velocity had been greater than 4.0 ft/s, it would be necessary to begin at an intermediate location after Station 20+00 and determine the station at which a protective lining, ditch cross section change or energy dissipater should be introduced. This is a trial and error process and could be worked from the outfall back or from an upstream station. Since the drainage procedure has several assumptions, it is a matter of engineering judgment in defining a point at which an alternative ditch lining and/or riprap would be considered. In this problem, based on the data given, it appears that the grass lining would be sufficient. However, it would be prudent from an environmental and erosion control perspective to place riprap for at least the last 25 ft.

Figure 6B-9
Roadside Ditch Design Form – Completed for Example 4

ROADSIDE DITCH DESIGN

Prepared by:_____ **SHEET NO.** 1 **of** 1
Checked by:_____ **LOCATION: Andrews Lake RD.** **CONTRACT NO:**_____
Storm Frequency: 10-yr

Station/Location	1			2		Ditch Section							8	9	10	11	12	13	14
	Area (ac)			C															
	Total	Pervious (ac)	Impervious (ac)	Pervious (n)	Impervious (n)	3 Total C x A	4 t_c (min)	5 I (in/hr)	6 Q (ft^3/s)	7 z_1	7 B	7 z_2	S (ft/ft)	n	d(ft)	V (ft/s)	Permissible V (ft/s)	Erosion Control Type	Width (ft)
20 (Bk)	9.73	9.18	0.55	0.4	0.95	4.19	23.67	3.83	16	4	2	4	0.025	0.05	0.9	3.17	4.0	grass	total
32	17.44	9.18	0.55	0.4	0.95														
		8.26	0.66	0.55	0.95														
						7.25	39.39	2.94	21	4	2	4	0.025	0.05	1.02	3.39	4	grass	total

Figure 6B-10
Roadside Ditch Design Form

ROADSIDE DITCH DESIGN

Prepared by:_____ **SHEET NO.**____**of**_____
Chk'd. by:_____ **LOCATION:**_____ **CONTRACT NO:**_____
Storm Frequency:_____**-yr**

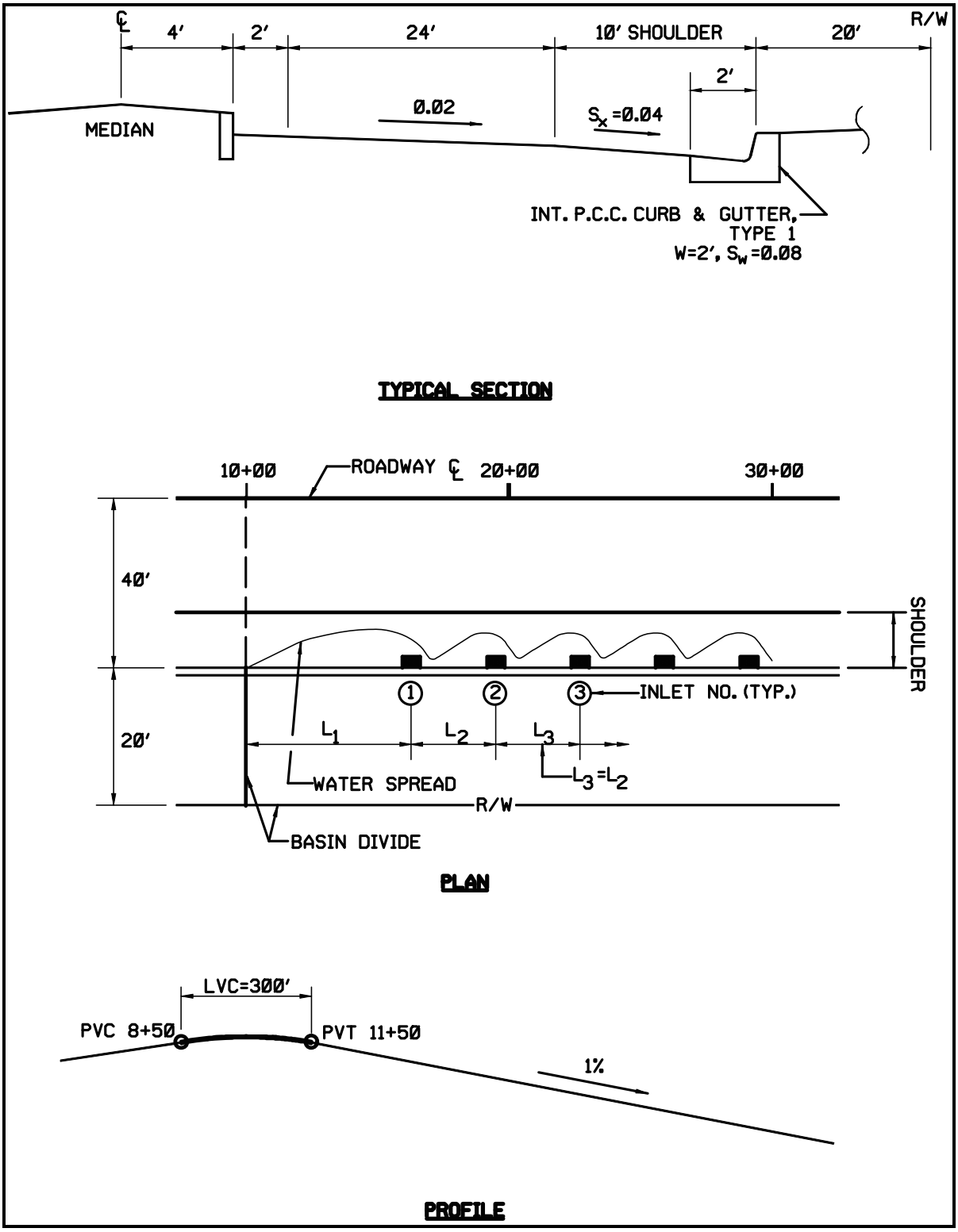
Station/Location	1			2		Ditch Section							8	9	10	11	12	13	14
	Area (ac)			C									S (ft/ft)	n	d(ft)	V (ft/s)	Permissible V (ft/s)	Erosion Control Type	Width (ft)
	Pervious	Impervious	Total	Pervious	Impervious	3	4	5	6	7									
						Total C x A	t_c (min)	I (in/hr)	Q (ft^3/s)	z_1	B	z_2							

EXAMPLE 5 – INLET SPACING

PROBLEM STATEMENT

Figure 6B-11 shows data for a 4-lane divided New Castle County urban arterial with a *C* of 0.88.

Figure 6B-11 – Typical Section, Plan and Profile



The following describes the procedure for determining inlet spacing based on the equations in Section 6.8.1, the hydraulic data given, use a C = 0.88, and DelDOT's standard 20"x36" Type 1 grate. This example has been simplified and does not include site-specific factors, such as maximum spacing based upon access for maintenance, location of commercial and residential entrances, curb ramps, superelevation transitions, curb height, etc. HEC-22 provides a comprehensive description of the procedure for determining inlet capacity and placement. HEC-12 (1984) presents a more basic understanding of the concept of gutter flow and for determining inlet spacing using charts and nomographs.

<u>Solution:</u>

Step 1. Select the design frequency.

From Figure 6-1, the design frequency is 10-yr return period for an urban arterial.

Step 2. Select the allowable spread.

From Figure 6-2, T = 10 ft.

Note: Tables for values of Q_t and Q_i with variables of S_L, S_X, and S_w can be found on DelDOT's web site. These values can be used to check the calculated values but are based on untested performance data

The given composite cross section has an S_x = 0.04, S_w =- 0.08, W = 2 ft, S_L = 0.01, W/T = 0.2 and S_w/S_x = 2

Step 3. From Charts 1B and 2B, HEC-22:

E_o = 0.485

Q_s = 4.2

$Q_t = Q_s / (1-E_o)$

Q_t = 4.2 / (1- 0.485) = 8.2 ft^3/s

Chart 1B represents solutions to the equation: $Q = (K_u/n)\, S_x^{1.67}\, S_L^{0.5}\, T^{2.67}$ where K_u= 0.56. This is used in conjunction with Chart 2B together they solve the equation for composite gutter flow**.** Q_t = 8.25 ft^3/s using the equation. Using the design aid chart Q_t = 8.26 ft^3/s.

E_o for composite gutter sections can be derived from the equation:

$$E_o = \frac{1}{\left\{1 + \dfrac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{T / W - 1}\right]^{2.67} - 1}\right\}}$$

Step 4. Find the grate interception capacity using the following equations 6.45 and from Figure 6-22:

$$Q_i = Q\left[R_f E_o + R_s\left(1 - E_o\right)\right]$$

$$R_s = \frac{1}{\left[1 + 0.15\left(V^{1.8}\right) / S_x L^{2.3}\right]}$$

R_f = 1.0 if $V \le 6.0$

From DelDOT's web site:

Q_i = 4.99 ft^3/s (

Step 5. Therefore $E = Q_i / Q_T$ = 4.99/8.2

= 0.61 ≤ 0.70

Try T = 8 ft for which: W/T = 0.25

E_o = 0.6

Q_s = 1.94

$Q_t = Q_s / (1-E_o)$

Q_t = 4.85 ft^3/s

Q_i = 3.32 ft^3/ from DelDOT's web site

E = 3.32 / 4.85 =0.68 < 0.70

Try T = 6 for which W/T = 0.333

E_o = 0.73

Q_s = 0.66

$Q_t = Q_s / (1-E_o)$

Q_t = 2.44 ft^3/s

Q_i = 1.96 ft^3/ from DelDOT's web site

E = 1.962 / 2.44 =0.80 > 0.70

Based on the various assumptions used in drainage calculation, using T = 6 is probably too conservative.

Therefore, use T = 8 ft and Q_t =4.85

$Q_B = Q_T - Qi$

Q_B = 4.85 – 3.32 = 1.53 ft^3/s

Spacing of Inlet 1

Assume t_c = 5 minutes

From Figure 6-5, I = 6.42 in/hr

$$L_1 = \frac{Q_{iT}\,x43560}{C_1 I_1 W_1} = \frac{4.85x43560}{0.88x6.42x60}$$

L_1 = 623.25 ft (round to a more convenient 625 ft

The location of Inlet 1 equals station:

(10 + 00) + (6 + 25) = 16 + 25

Check t_c: V = 2 ft/s for shallow gutter flow on a 1% slope, so using equation 6.5:

$$T_t = \frac{L}{60V} = \frac{625}{(60)(2)} = 5.21\text{min}$$

Therefore, t_c = 5.21 min as this is the minimum recommended

I = 6.36 in/hr

Spacing of Inlet 2

Using equation 6.61,

$$L_2 = \frac{Q_{2T} - Q_{B1}}{C_2 I_2 \overline{W}} x43560$$

$$= \frac{4.85 - 1.53}{0.88x6.36x60} x43560 = 430.66$$

The location of Inlet 2 equals station:

(16+25) + (4 +30) = 20+55

Subsequent inlets shall be spaced at 430 feet as long as the geometric controls remain unchanged and there is clean-out access at a minimum distance of 300 ft (Figure 6-3). Stationing of inlets can be rounded to the nearest foot for construction convenience and can be adjusted slightly to accommodate other project requirements. An exaggerated degree of precision in calculations is rarely justified.

All inlets of the drainage system shall be numbered and the computed results shall be presented in a tabular format using the Inlet Spacing Computation sheet.

<u>Inlet Spacing Design Form</u>

S_x, S_w, and W are the cross sectional elements of the gutter section from the constructions plans and Standard Construction Details. For concrete gutter sections the recommended n = 0.016. Refer to Section 6.8 for details on the procedure, applicable equations and additional references.

The drainage design begins by using a set of preliminary construction plans and marking preliminary drainage areas and known inlet locations such as curb ramps, entrances, sags etc. as specified in Section 6.8.1.8. The initial area is 300 to 500 feet long below the high point.

The primary control is the spread. The upstream inlet will define the initial spread and the design continues by locating inlets as necessary to control the spread as defined by the design criteria. To locate these inlets incremental areas of flow are selected and calculations are performed. The flow for the area is determined and including any flow that has bypassed previous inlets and the next inlet placed to intercept this flow before the design spread is exceeded.

The form and the following description allow for an orderly determination and recording of this process. For a continuous longitudinal slope the process is relatively simple if the contributing subsequent areas have uniform runoff characteristics. After determining the spacing between the first two inlets all downstream inlets are similar spaced until there is a change in the defining drainage data.

Procedure:

1. Enter in Col. 1 the inlet number. All inlets in a storm drain shall be numbered in sequence starting with the first inlet from the crest down to the inlet at the low point of the sag curve where the interception is 100%.
2. Enter in Col. 2 the inlet's location by its baseline station and whether it is left or right of the baseline.
3. Enter in Col. 3 the partial drainage area, ΔA (ac), which discharges into the inlet. It

is the uppermost part of the entire drainage area for the first inlet, or the partial area above the inlet up to its upstream inlet for subsequent inlets. Note that in some cases areas outside the pavement may have to be included in ΔA.

4. Enter in Col. 4 the runoff coefficient of the drainage area ΔA of the inlet. If the drainage area is composite, then a weighted value of C using Equation 6.2 should be used. C values are to be selected from Figure 6-8.
5. Enter in Col. 5 the travel time T_t (min) for the surface runoff to travel distance, L, as in Section 6.6.
6. Enter in Col. 6 the intensity of rainfall I (in/hr) for the inlet corresponding to t_t for the 10-yr frequency, the values of I are obtained from Figures 6-6, 6-7 or 6-8.

   If drainage inlets are located on a depressed roadway, then a 50-yr frequency must be used for most types of highways as indicated in the design criteria.
7. Enter in Col.7, the gutter flow, Q, the product of Columns 4, 5and 6.
8. Enter in Col.8 the longitudinal slope of the gutter at the inlet, S_L (ft/ft). For vertical curves determine S_L from the slope equation for a parabolic curve. This value of S_L is used to determine the gutter flow rate and the water spread on the pavement.
9. Enter in Col. 9 the cross slope of the gutter/pavement section.
10. Enter in Col. 10 the bypass flow rate from the up stream inlet as shown in column 18 of the previous row.
11. Enter in Col. 11 the total gutter flow, Q_T, approaching the inlet. Q_T is equal to ΔQ in Column 7 plus the bypass flow rate of the upstream inlet Q_B of Column 18 of the previous row of the table.
12. Enter in Col. 12 the depth of flow at curb, d (ft). It must be 0.5 ft or less. Follow the procedure in 14 if d controls T so that T may be decreased.
13. Enter in Col. 13 the width of the gutter or grate.
14. Enter in Col. 13 the width of water spread, T, on the gutter section, due to Q_T (ft). T can be calculated for the gutter section with either a uniform cross slope or composite cross slope as explained in Sections 6.8.1.5.1 and 6.8.1.5.2. T must not exceed the permissible water spread specified in the design criteria. If T is excessive, reduce Q_T to the amount that can be accommodated within the permissible T. Use a trial and error procedure by decreasing the distance between inlets resulting in a smaller ΔA to find the matching Q_T
15. Enter in Col. 15 the ratio of gutter/grate width divided by the spread. This ratio is needed when using the charts in HEC-22 to determine the interception efficiency and solving for other design values.
16. Enter in Col. 16 the inlet type, be sure to indicate if using other than the standard grate types
17. Enter in Col. 17 the intercepted flow, Q_i. Compute Q_i as explained in Section 6.8.1.6.2.1. For standard grates on grade, Q_i can be obtained from DelDOT'S web site**.**
18. Enter in Col. 18 the by-pass flow, Q_B, by subtracting column 17 from column 11.
19. Enter in Col. 18 remarks such as sump condition, 50-yr frequency, etc

One of the final checks is to determine the interception efficiency, E, of the inlet. E is equal to Q_i in column 17 divided by Q_T in column 11. If E is less than 0.7, use a trial and error procedure by decreasing T (see 14) and repeating the computations until $E \approx 0.7$.

Figure 6B-12
Inlet Spacing Computation Form

INLET SPACING COMPUTATION SHEET	Project No._____ Sheet No._____ of _____ Computed By: _____ Chk. By: _____

INLET		GUTTER DISCHARGE Design Frequency_____-yr						GUTTER DISCHARGE Allowable Spread _____ ft							INLET DISCHARGE		
No.	Sta.	Drain Area ΔA (ac)	Runoff Coefficient C	Time of Concentration t_t (min)	Rainfall Intensity I (in/hr)	$\Delta Q = \Delta C I \Delta A$ (ft^3/s)	Longitudinal Slope S_L (%)	Cross Slope S_x or S_w (%)	Previous Bypass Flow (ft^3/s)	Total Gutter Flow (ft^3/s)	Depth d (ft)	Grate or Gutter Width W (ft)	Spread T (ft)	W/T	Inlet Type	Interception Q_i (ft^3/s)	Bypass Flow Q_b (ft^3/s)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

REMARKS:

EXAMPLE 6 ─ STORM DRAINS

Detailed examples, acceptable forms, and available software for the design of storm drains are found in HDS-3, HDS-4, HDS-5, HEC-22 and other references in Sections 6.8 and 6.9. Below is an acceptable process for using the forms.

PROCEDURE FOR COMPLETING STORM DRAIN COMPUTATION FORM

Col. 1 ─ Pipe Unit Number. Enter the designated number of the pipe unit. The computations begin with the first pipe unit in the uppermost location of the drainage area, gradually proceeding downstream in sequence.

Col. 2 ─ Upper Inlet Number. Enter the inlet (junction) number at the upper end of the pipe unit under consideration. Below this inlet number and within the space, indicate its baseline station and whether to the left or right of the baseline.

Col. 3 ─ Lower Inlet Number. Enter the inlet (junction) number at the lower end of the pipe being designed. As in Column 2, indicate its location.

Col. 4 ─ Drainage Area. Enter the incremental area, ΔA, of the (portion of) watershed draining into the inlet in Column 2. For the first pipe, ΔA is the uppermost area of the watershed where the storm water flows on the surface into the inlet. For subsequent pipes, it is the drainage area bounded between the inlet in Column 2 and the previous upstream inlet.

Col. 5 ─ Runoff Coefficient. Enter the runoff coefficient C for the drainage area in Column 4. It is the composite runoff coefficient for the drainage area ΔA as in Equation 6.2:

$$C = \frac{\sum C_x A_x}{A_{Total}}$$

The incremental drainage area may be assumed to be composed of only pervious and impervious runoff coefficients. Select the appropriate runoff coefficients from Figure 6-8 and compute the weighted average for the composite runoff coefficient, C.

Col. 6 ─ Product. Enter the product of Column, 4 and 5, which is $C \times \Delta A$.

Col. 7 ─ Cumulative Product. Enter the sum of entries of Column 6 applicable to the pipe unit. Determine CA for the n'th pipe unit as follows:

$$(CA)_n = (CA)_{n-1} + (C \times \Delta A)_n$$

Col. 8 ─ Time of Concentration. Enter the time of concentration for the pipe, t_c (min), which is the travel time of stormwater along the hydraulic length to reach the inlet in Column 2. Analyze all possible routes. Use the route with the longest travel time.

For the first pipe, the hydraulic length includes only surface flow from the uppermost areas of the watershed that drains into the inlet in Column 2. The travel time for surface flow is computed by using Equation 6.3, 6.4 and/or 6.5.

For subsequent pipe segments, the hydraulic length (surface flow) is underground along the storm drain. Thus the time of concentration for a pipe is the time of concentration for stormwater to travel from the upstream pipe plus the time required by storm water to travel the length of the pipe. For the n'th pipe, it is represented by the equation:

$$(t_c)_n = (T_c)_{n-1} + (T_t)_{n-1}$$

On the form, t_c is entered by adding Column 8 and Column 16 of the previous row. The minimum period for t_c is 5 minutes.

Col. 9 ─ Rainfall Intensity. Enter the intensity of rainfall corresponding to the time of concentration in Column 8, I (in/hr). Use Figures 6-6, 6-7 or 6-8 for the design frequency.

Col. 10 ─ Design Discharge. Enter the design discharge for the pipe unit, Q. It is the product of the entries in Columns 7 and 9.

Col. 11 — Pipe Size. Enter the pipe diameter in inches. Use Figure 6-29 to size concrete pipes for the design discharge at friction slope or slightly steeper. Otherwise the equations and referenced resources can be used to find a suitable pipe size and an acceptable slope. See equations, charts and tables referenced in Section 6.9.

Col. 12 — Length of Pipe. Enter the length of the pipe, L (ft).

Col. 13 — Friction Slope. Enter the friction slope of the pipe unit, S_f (ft/ft). Use Equations 6.64 and 6.65.

Col. 14 — Actual Slope. Enter the actual slope of the pipe, S (ft/ft). Note that $S \geq S_f$.

Col. 15 — Flow Velocity. Enter the velocity of flow in the pipe unit, V. See Section 6.9.

Col. 16 — Travel Time. Enter the travel time for the flow to travel over distance L in the pipe, T_t (equation 6.5):

$$T_t = \frac{L}{60V} = \frac{(Col.12)}{(60)(Col.15)}$$

Col. 17 — Upper Invert Elevation. Enter the invert elevation of the pipe at the inlet in Column 2. The elevation for the first pipe is normally based on the minimum cover requirement for the selected type of pipe.

Col. 18 — Lower Invert Elevation. Enter the invert elevation at the lower end of the pipe. Col. 18 = Col. 17 – (Col 12 x Col. 14)

Figure 6B-13
Storm Drain Computation Form

STORM DRAIN COMPUTATION SHEET									Project No._____ Sheet No._____ of _____ Computed By: _____ Chk. By: _____								
Pipe Unit			ΔA	C	C x ΔA	CA	t_c (min)	I (in/hr)	Q = CIA (ft^3/s)	Diameter (in)	Length (ft)	S_f (ft/ft)	S (ft/ft)	V (ft/s)	t_t (min)	Invert Elevation	
No.	From	To														Upper	Lower
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

REMARKS:

PROCEDURE FOR COMPLETING HYDRAULIC GRADELINE COMPUTATION FORM

Col. 1—Junction Number. Enter the junction number that represents the catch basin, manhole or junction box immediately upstream of the outflow pipe. HGL computations begin at the outfall and are calculated for each junction in the storm drain.

Col. 2—Baseline Station. Enter the baseline station for the junction indicating whether it is to the left or right of the baseline.

Col. 3—Outlet Water Surface Elevation. Enter the outlet water surface elevation. For the first junction it is the outfall design tailwater elevation; for subsequent junctions it is the inlet water surface elevation of the previous downstream junction.

Col. 4—Outfall Pipe Diameter. Enter the diameter of the outflow pipe unit, D_o (in).

Col. 5—Design Discharge. Enter the design discharge for the outflow pipe, Q_o.

Col. 6—Outflow Pipe Length. Enter the length of the outflow pipe, L_o.

Col. 7—Friction Slope. Enter the friction slope of the outflow pipe flowing full, S_{fo} (ft/ft). Refer to Figure 6-29 for concrete pipes, or use equations 6.67 and 6.69 to derive:

$$S_{fo} = \frac{H_f}{L_o} = \left(\frac{Qn}{0.46D^{2.67}}\right)^2$$

Col. 8—Friction Head Loss. Enter the head loss due to friction for the outflow pipe, H_f by using equation 6.71:

$$H_f = L_o S_{fo}$$

On curved alignment, the additional head loss due to friction, H_c is given by:

$$H_c = \frac{0.002\,\varnothing\,V_o^2}{2g}$$

Where,

$\varnothing$ = Angle of curvature in degrees,

V_o = Velocity in the outflow pipe (ft/s)

$$V_o = \frac{Q_o}{A}$$

A = Area of outflow pipe (ft^2)

g = 32.2 ft/s^2.

Add H_c to H_f and enter the sum as total H_f in Column 8.

Col. 9—Outflow Velocity. Enter the velocity of flow in the outflow pipe, V_o (ft/s), as determined in Step 8.

Col. 10—Contraction Head Loss. Enter the contraction head loss, H_o, by using the formula:

$$H_o = \frac{0.25\,V_o^2}{2g}$$

Col. 11—Inflow Pipe Velocity. Enter the velocity of flow in the inflow pipe that flows into the junction, V_i (ft/s). V_i equals the design discharge for the inflow pipe (ft^3/s) divided by its area (ft^2).

When more than one pipe is flowing into the junction, V_i is determined as follows. For each pipe flowing into the junction, compute the product of its velocity of flow and the discharge. The pipe that has the greatest product will be considered; its V_i will be entered in Column 11. In this situation, the time of concentration that influences the design discharge may have to be adjusted depending on the controlling pipe whose V_i is entered in Column 11.

Column 12—Expansion Loss. Enter the controlling expansion loss, H_i using the equation:

$$H_i = \frac{0.35\,V_i^2}{2g}$$

Col. 13—Skew Angle. Enter the angle of skew between the controlling inflow pipe Δ (degrees).

Col. 14—Bend Loss. Enter the bend loss, H_Δ, which is obtained from the equation:

$$H_\Delta = \frac{K\,V_i^2}{2g}$$

Where:

K is a loss coefficient,

The values of K are listed at the bottom of form for various skew angles.

Col. 15—Total Head Loss. Enter the total head loss at junction, H_t, which is equal to the sum of H_o in Column 10, H_1 in Column 12, and H_Δ in Column 14. That is: $H_t = H_o + H_i + H_\Delta$

Col. 16—Adjusted Head Loss. Enter the adjusted total head loss at junction, $\overline{H_t}$, determined by multiplying H_t with a coefficient as follows:

- If the junction incorporates a drainage inlet that intercepts at least 10% of mainline flow, then the coefficient is 1.3.
- When the interception is under 10%, the coefficient is 1.0.
- If the junction incorporates a manhole or a junction box, then the coefficient is 0.5.

Column 17—Gross Total Head Loss. Enter the gross total head loss, H, which is the sum of friction loss, H_f, in Column 8 and adjusted junction loss $\overline{H_t}$, in Column 16.

Col. 18—Potential Water Surface Elevation. Enter the sum of the elevation in Column 3 and H in Column 17. This elevation is the potential water surface elevation for the junction under consideration.

Col. 19—Top Elevation of Structure. Enter the top of manhole elevation or the inlet elevation on the gutter flow line (elevation at the top of catch basin). If this elevation is not at least one foot higher than the elevation in Column 18, then the hydraulic gradeline must be lowered to satisfy the design criteria.

The HGL can be lowered by increasing slopes and sizes of pipe, or by reducing the design discharge that can be achieved through proper stormwater management.

Figure 6B-14
Hydraulic Gradeline Computation Form

Project No._____ Sheet No._____ of _____

Computed By: _____ Chk. By: _____`

Junction No.	Sta.	Outlet Water Surface El.	Pipe Loss					Junction Loss								H	Inlet Water Surface El.	Top EL of MH or CB.
			D_o	Q_o	L_o	S_{fo}	H_f	V_o	H_o	V_i	H_i	Δ	H_Δ	H_t	$\overline{H_t}$			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

$H_i = 0.35\frac{V_i^2}{2g}$ $H_o = 0.25\frac{V_o^2}{2g}$ $H_\Delta = K\frac{V^2{}_i}{2g}$ $H_t = H_o + H_i + H_\Delta$ **Final** $H = H_i + H_t$

K Value

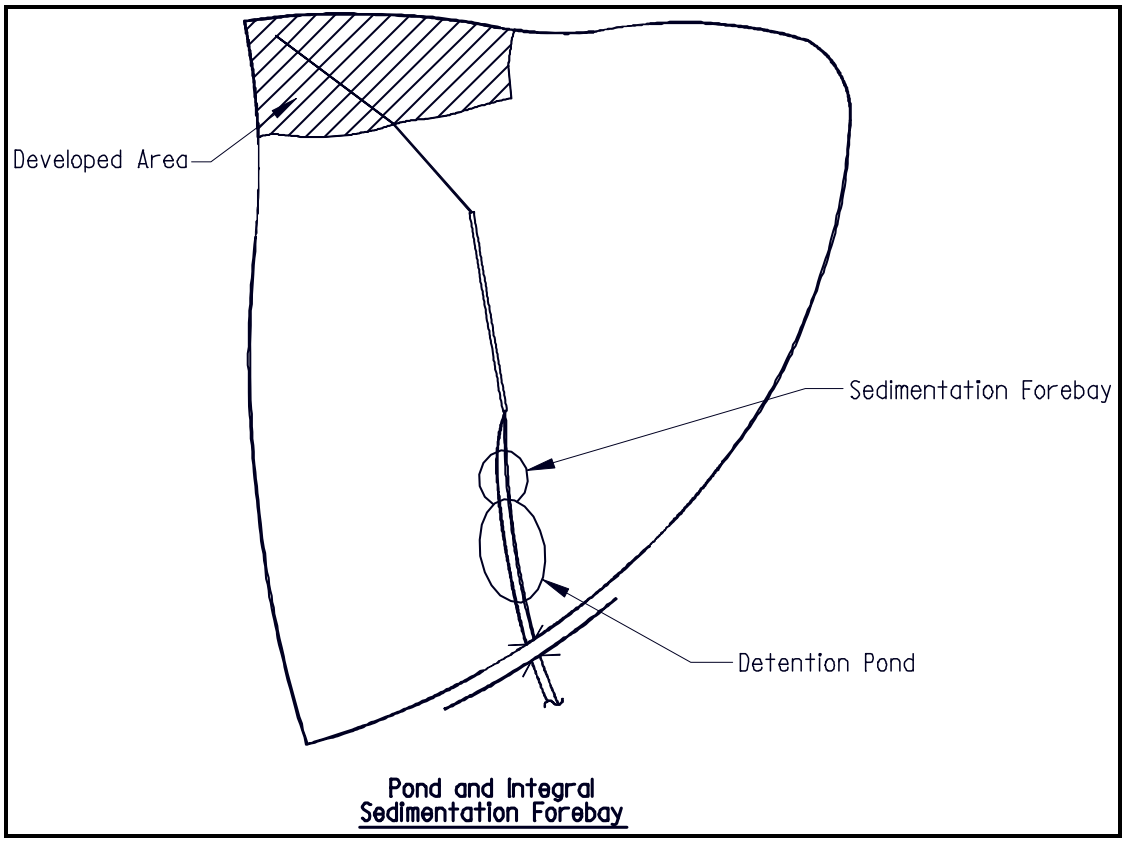
90°	0.70	70°	0.61	50°	0.47	30°	0.28	20°	0.16
80°	0.66	60°	0.55	40°	0.38	25°	0.22	15°	0.10

EXAMPLE 7 – POND DESIGN

PROBLEM STATEMENT:

Design a stormwater detention pond for Example 2 using the TR-55 method. The area to be commercially developed is classified as Am and a detention pond with a release structure will control stormwater runoff increases back into the existing channel due to the construction of the commercial site. Since the project is located above the C&D Canal, the design will be based on the TR-55 Tabular Hydrograph Method for developing inflow and out flow routing and stage-discharge relationships that allow for determining the pond size and most efficient control structure.

Figure 6B-15
Location of Pond



This example is simplified to illustrate the basic concept of controlling stormwater runoff through the use of a retention basin. In reality, the regulations would require the new commercial site to provide onsite stormwater quantity and quality features to limit the site discharge to the pre-developed peak flow and reduce the total suspended solids to 80%. The current State regulations should be consulted for design criteria. A detention pond permitted at the location in this example would have to be designed combining each subarea's inflow and outflow hydrographs and routings to the outfall. The analysis would be made for the 2-, 10- and 100-yr Type II 24-hr storms. Bridge Design would be requested to analyze the downstream culvert and channel for possible flooding, as channel stability and wetland encroachment.

Figure 6B-16
Watershed for Example 6

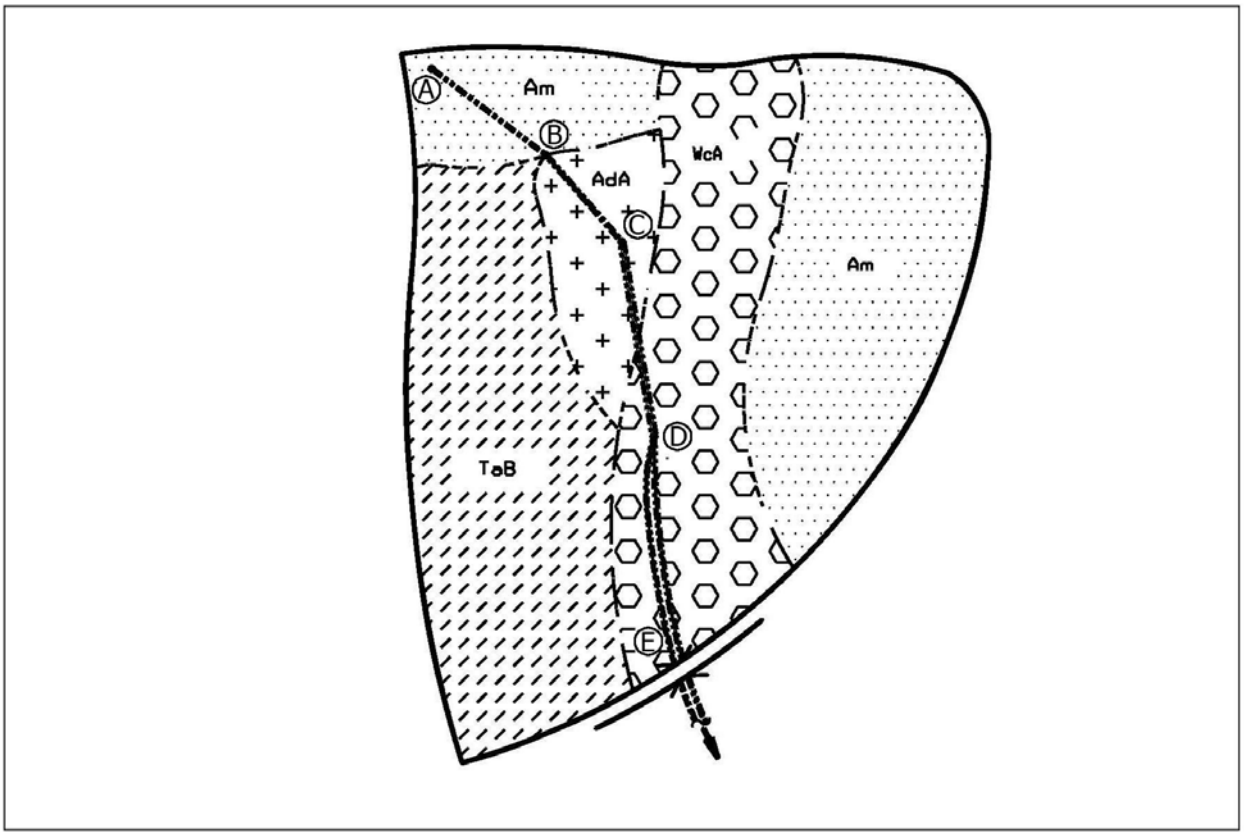


Figure 6B-17
Flow Path of Watershed

Segment	Description	Slope %	Length (ft)
A-B	Commercial and Business	1.0	50
B-C	Forest and meadow	2.0	50
C-D	Shallow gully	0.8	250
D-E	Open channel	0.8	475

The first step is to establish all of the hydrologic parameters and regulatory design controls. The design data for this problem is based on Example 2. The area to be developed is the upper watershed, segment A-B, designated as Am that is to be developed from "1 ac Residential" to "Commercial and Business". A 25-yr frequency is to be used. The data developed in Example 2 applies.

The pre-development design data from Example 2 is:

A_{total} = 36.5 ac or 0.057 mi^2

$\overline{CN}$ = 74 q_d = 3.2 in

t_c = 0.43 hr I_a/P = 0.12

q_u = 575 ft^3s/mi/in Q = 105 ft^3/s

Determine the post-development data.

The weighted average curve number due to the proposed construction is computed as:

$$\overline{CN} = \frac{\Sigma\,(CN\ x\ A)}{\Sigma\,A} = \frac{2734.25}{36.50} = 74.9 \cong 75$$

Calculate Runoff Depth q_d from equations 6.9 and 6.10:

$$q_d = \frac{(P_{24} - 0.2S))^2}{P_{24} + 0.8S}$$

$$S = \frac{1000}{CN} - 10$$

$$q_d = \frac{\left[6.0 - 0.2\left(\frac{1000}{75} - 10\right)\right]^2}{6.0 + 0.8\left(\frac{1000}{75} - 10\right)} = 3.28in$$

CALCULATE t_C

Use the topographic data from Figures 6B-16 and 17.

Sheet (Overland) Flow (segments A-B & B-C) this type of flow is assumed to be no greater than 3 inches in depth.

Find T_{t1} using equation 6.11:

$$T_{t1} = \frac{0.007\,(nL_1)^{0.8}}{P_2^{\,0.5}\,s^{0.4}}$$

From Table 3-1, TR-55, $n_{A\text{-}B}$ = 0.011 (asphalt) and $n_{B\text{-}C}$ = 0.40 (woods, light underbrush). So,

$$T_{t1} = \frac{0.007\,(0.011\,x\,50)^{0.8}}{(3.2)^{0.5}(0.01)^{0.4}} + \frac{0.007\,(0.40\,x\,50)^{0.8}}{(3.2)^{0.5}(0.02)^{0.4}} = 0.22hr$$

Shallow Concentrated Flow (segment C-D) assumed to remain the same. Flow depths for this type of flow are assumed to be in the 4 to 6 inch range.

$$T_{t2} = \frac{250}{3600\,x\,1.44} = 0.048hr$$

Open Channel Flow (segment D-E)

The travel time is calculated the same as for the Rational Method, so T_{t3} = 0.041 hr.

Therefore, from equation 6.8:

t_c = 0.22 + 0.048 + 0.041 = 0.31 hr

DETERMINE UNIT PEAK DISCHARGE q_U

The unit discharge, q_U is determined from Exhibit-II, TR-55.

Find the ratio I_a/P using Table 4-1, TR-55, or using equation 6.10 and that I_a is assumed to be 20% of S:

$$\frac{I_a}{P} = \frac{\frac{200}{CN} - 2}{P_{24}} = \frac{\frac{200}{75} - 2}{6.0} = 0.11$$

Therefore, from Exhibit 4-II:

q_u = 660 ft^3/s /sq mi/in

Pond Adjustment Factor F_p remains F_p = 1.0

CALCULATE Q

Drainage Area A_m = 36.5/640 = 0.057 sq mi

From equation 6.14:

$Q_{25\text{-}developed} = q_u\,A_m\,q_d\,F_p$

= 660 x 0.057 x 3.28 x 1.00 = 123 ft^3/s

Therefore, the developed condition peak discharge Q_{25} equals 123 ft^3/s.

Figure 6B-18
Curve Number Computations

Soil Type	Hydrologic Soil Group	Land Use (Good Hydrological Conditions)	*CN*	Area, *A* (Acres)	*CN x A*
Am	C	Business	94	2.50	235.00
Am	C	Residential, ½ Acre	80	12.60	1008.00
AdA	C	Residential, 1/4 Acre	83	0.75	62.25
AdA	C	Woods	70	1.10	77.00
TaB_2	B	Residential, 1/4 Acre	75	1.00	75.00
TaB_2	B	Woods	55	9.00	495.00
WcA	D	Residential, 1 Acre	84	1.75	147.00
WcA	D	Residential, 1/2 Acre	85	4.30	365.50
WcA	D	Woods	77	3.50	269.50
			Total	36.50	2734.25

Figure 6B-19
Impervious Area Computations

Land Use	Acres	Percent Impervious	Impervious Acres
Woods	13.60	12*	1.63
Residential 1 acres	1.75	20	0.35
Residential ½ acres	16.90	25	4.23
Residential ¼ acres	1.75	38	0.67
Business	2.50	85	2.13
		Total	9.01
		Percent	25

*Assumes at least a minimum future change in land use

RETENTION/DETENTION DESIGN

The problem statement is to design a storage facility and an outfall structure that discharges at the pre-development rate. Storage basins are frequently designed to function as a wet pond (retention facility for quantity and quality control) and a detention facility releasing runoff volume at an allowable discharge rate. The design is based on hydrologic routing of inflow and outflow based on several design storms and various types of outfall structures. For the example the pond is being sized for the 25-yr storm that may or may not be in conformance with the current requirement.

Computer models are readily available to perform all the design steps. Even though the example has limited the problem to a single storm event, this is still a very time consuming and substantial undertaking if done manually. The example will describe the process in designing a stormwater management pond.

The following subsection will allow the designer to manually size a trial pond to use in assessing whether a proposed location is even physically feasible and should be further evaluated through the software. The method is from TR-55. As pointed out in the document this method is only about 25% accurate and is to be confirmed by using complete hydrograph routing and volume analysis through a software program.

For generating runoff hydrographs, the State has been divided into two parts with the Chesapeake and Delaware Canal as the dividing line. The reasoning is that the general hydrological soil conditions below the canal provide better initial absorption rate for runoff than above the canal and that the average watershed slope is <5% thus increasing the time to reach peak discharge. Therefore, the NRCS Type II Unit Dimensionless Hydrograph is used above the Canal and the Delmarva Unit Hydrograph is used below the Canal.

POND VOLUME DETERMINATION

The TR-55 approximation method consists of four steps:

Step 1. Based on the design frequency, determine the peak discharges Q_1 and Q_2 for the pre-and post development conditions. Based on the existing criteria, Q_1 is used as the allowable downstream peak discharge.

Step 2. Calculate the ratio Q_1/Q_2 and obtain the ratio V_s/V_r from Figure 6-1, TR-55, using the Type II curve. For areas below the canal, this value would be somewhat less for ratios above 0.3 and would give a greater storage volume than is probably required. V_s is the required storage volume in acre-feet and V_r is the total runoff volume in acre-feet.

Step 3. The runoff volume is calculated using the equation:

$$V_r = 53.33\ q_d\ A_m \text{ ac-ft}$$

Where:

q_d = Runoff depth in inches for the post-development condition

Am = Drainage area in mi^2

Therefore for this example:

Q_1 = 105 ft3/s Q_2 = 123 ft3/s

$Q_1/Q_2 = 0.85$

Previously calculated:

q_d = 3.28 in $A_m = 0.057\ mi^2$

From TR-55 Figure 6-1, $V_s/Vr = 0.2$

The equation for determining this value is:

$$\frac{V_s}{V_r} = 0.683 - 1.43(Q_1/Q_2) + 1.64(Q_1/Q_2)^2 - 0.804(Q_1/Q_2)^3$$

Using this equation the value is 0.16 (use 0.2)

Using the equation for finding V_r:

V_r = (53.33)(3.28)(0.057) = 9.97 ac-ft

Step 4. Determine the storage volume, V_s in acre-feet by multiplying the known V_s/V_r ratio of Step 2 with V_r obtained from Step 3.

Therefore,

V_s = (9.97)(0.2) = 2.0 ac-ft or

V_s = 86,859 ft^3

Using this value an approximation of the average release rate can be determined as follows:

V_s is to be released over a 24-hr period, therefore:

86,859 ft^3/ (24 hrs x 3600 sec/hr)=1.0 ft^3s

Step 5. Determine an initial pond size using Equation 8-8 and 8-9 in TR-55. Assume that the side slopes are 3 to 1, therefore Z = 3. Because, we have selected this slope ratio a safety will be incorporated in the final pond layout. Slopes greater than 4:1 do no currently require a dry safety bench. However, ponds with side slopes 4:1 or less require a 10-foot wide aquatic bench one foot below the permanent pond level and a dry 10-foot safety bench 1 foot above the permanent pond level

Based on the site survey data there is 7 ft of topographic relief available at the proposed pond location and the water table level allows for the following:

Bottom elevation	93.0
Aquatic bench elevation	95.0
Permanent pool elevation	96.0
Outlet structure elevation	96.0
Safety bench elevation	97.0
Design water surface level	98.0
Emergency Outfall	100.0

It should be pointed out that because the designer would like this pond to be a wet pond and to ensure there is no rock or other problems in constructing the pond at this location, soil borings locating the normal water table and type of soils were conducted. If there had been a possibility that the natural aquifer would be contaminated, it would have been necessary to provide an impervious clay liner up to the desired pond depth depending upon the potential adverse affect to the underlying aquifer.

The estimating procedure for sizing a pond is (1) assume a trapezoidal shape with a length (in the direction of flow) to width ratio of 2:1 or greater, (2) side slopes not to exceed 3:1, preferably greater, and (3) provide at least 1-ft of freeboard above the highest design water elevation. It is generally preferred that ponds be irregular in shape with the longest dimension being parallel to the flow.

The formula for determining the volume of a trapezoidal basin is:

$$V = LWD + (L+W)ZD^2 + \frac{4Z^2D^3}{3}$$

where:

V= Volume (ft^3)

L = Length of bottom (ft)

W = Width of bottom (ft)

D = Depth of basin (ft)

Z = Side slope factor (ratio horizontal to vertical)

An approximate method to estimate a trial basin knowing the storage volume is to use the equation:

$$L = \frac{-ZD(r+1) + \left[(ZD)^2(r+1)^2 - 5.33(ZD)^2 r + \frac{4rV}{D}\right]^{0.5}}{2r}$$

r = Ratio of width to length of basin

Using this equation and the available data find the approximate size of the basin. The data is:

Z = 3 D = 2 ft r = 0.5 V_s = 86,569 ft^3

Results: L = 290 ft W = 145 ft

The depth of 2 ft is based on the depth available between the permanent pool level and the proposed outfall structure. Because of the inherent assumptions and approximations used in determining stormwater values, the

additional storage created by the use of a bench(s) is frequently ignored and the simple trapezoidal shape is used in the calculations.

In this example the benches have been used for determining available storage volumes etc. At elevation 95.0 L and W increase by 32 ft or L= 322 ft and W = 177 ft. at elevation 97.0 L and W increase by another 32 ft or L= 357 ft and W = 209 ft.

Using these dimensions, calculate the available volumes at various elevations.

Elev. 95.0

Data:
L = 290 ft W = 145 ft D = 2 ft Z = 3

V_{95} = 89,416 or 2.05 ac-ft

Elev. 96.0 Permanent Level

Data:
L = 290 ft W = 145 ft D = 2 ft Z = 3

V_{96} = 89,416 or 2.05 ac-ft plus the volume between elev.95.0 and 96.0 using L = 322 ft W = 177 ft D= 1 ft and Z= 3 the additional volume is 58,503 for a total volume at the permanent pool elevation of 147,919 ft^3 or 3.4 ac-ft.

For the storage calculations using the wet pond concept the available storage volume to be released at elevation 96.0 is 0.0. The storage volume available from Elevation 96.0 to 97.0 is:

Elev. 97.0

Data: L = 322 ft W = 177 ft D= 1 ft and Z= 3
V_{97} = 58,503 ft^3 or 1.27 ac-ft.

Next, find the volume at Elevation 98.0

Elev. 98.0

Data:
The volume at 98.0 includes the volume available at 97.0 (58,503 ft^3) plus the volume between these two elevations.
Data: L = 354 ft W = 209 ft D = 1 ft Z = 3

V_{98} = 58,503 + 75,687 = 134,190 ft^3 or 3.1 ac-ft.

The design has set elevation 96.0 as the elevation for any outfall structures with storage between elevation 96.0 and 98.0. The allowable storage must be at least equal to the calculated volume of 86,859 ft^3.

Elev. 99

Data: L = 354 ft W = 209 ft D = 2 ft Z = 3

V_{99} = 58,503 + 154,824 = 213,329 ft^3 or 4.9 ac-ft.

Elev. 100

Data: L = 354 ft W = 209 ft D = 3 ft Z = 3

V_{100} = 58,503 + 237,483 = 295,986 ft^3 or 6.8 ac-ft.

At this point a review of the initial basin configuration would indicate that the storage volume between elevations 96.0 and 98.0 is 3.15 ac-ft and is considerably larger than the 2.0 ac-ft required. Therefore iterations of several dimensions would probably be necessary. One of the easiest solutions would be to use the initial V_s divide by the allowable depth to find the required surface area at the design storage elevation. Than using the desired between length to width of at least 2:1 as selected in the example, L = 2W in the equation L x W = Surface area and making the substitution, $2W^2$ = Area. Therefore:

$V_s/2$ =A or 86,569/2 = 43285 ft^2

$2W^2$ = 43285 or W = 147 and L = 294.

These two dimensions apply at elevation 98.0 on a trapezoidal shape having a bottom elevation of 93.0. At this elevation, the two dimensions would be reduced by 30 feet would result in a trial size of L = 260 and W = 130. The volume equation for a trapezoid allows for a quick check of available storage between 93.0 and 96.0 V = 2.36 and between 93.0 and 98.0 V = 4.49. The available storage would be the difference between these two values or 2.13 ac-ft. This value is closer to the required volume. The proposed two benches will provide the additional storage to meet the inherent margin of error recognized in drainage design. Figure 6B-21 shows the tabulated values using the second trial size.

Figure 6B-20
Trial 1 (L=290 ft & W=145ft) Stage Storage

Elevation	Surface Area (ac)	Cumulative Storage (ac-ft)*	Usable Storage (ac-ft)**
93	0.97	0.00	0.00
94	1.03	1.0	0.00
95	1.09	2.05	0.00
95.1***	1.31	2.19	0.00
96	1.38	3.39	0.00
97	1.45	4.8	1.41
97.1	1.70	5.05	1.66
98	1.78	6.54	3.15
99	1.86	8.35	4.96
100	1.94	10.25	6.86

*Assumes pond is completely empty and is used for calculating quality control storage later.
**Assumes pond is full to Elevation 96.0 (permanent pool).
***Often on the first trial the bench elevations are not known. In this problem, assume there is a requirement for a permanent pool at Elevation 96.0, so the benches go at 95.0 and 97.0.

Figure 6B-21
Stage-Storage
Trial 2 (L=260 ft & W=130 ft)

Elevation	Surface Area (ac)	Cumulative Storage Volume (ac-ft)*	Usable Storage Volume (ac-ft)**
93	0. 78	0.00	0.00
94	0.83	0.80	0.00
95	0.88	1.66	0.00
95.1***	1.08	1.77	0.00
96	1.15	2.78	0.00
97	1.21	3.96	1.18
97.1	1.44	4.11	1.33
98	1.52	5.44	2.66
99	1.59	7.0	4.22
100	1.66	8.62	5.84

*Assumes pond is completely empty and is used for calculating quality control storage later.
**Assumes pond is full to Elevation 96.0 (permanent pool).
***Often on the first trial the bench elevations are not known. In this problem, assume there is a requirement for a permanent pool at Elevation 96.0, so the benches go at 95.0 and 97.0.

If the designer is confidant that the initial trial size is suitable for the designated site, then the design can be refined and verified through the appropriate computer software. The necessary input data for the software has been generated and the program will develop and route an inflow hydrographs through the pond, generate an outflow hydrograph based on the selected principal and emergency spillway configuration. The designer selects and inputs spillway types and sizes considered appropriate for the site and downstream conditions. This structure could be; a concrete wall built into the embankment with one or

series of broad crested or v-notch weirs; or a riser pipe with an outfall pipe (barrel); or a rectangular riser structure with one or more weir and orifice openings at different elevations to control a combination of storm frequencies and an outfall pipe. Risers with a pipe outlet are frequently referred to as drop inlets.

There are several types of flow calculations that have to be made. The pipe discharge flow capacity of any outfall pipe, the riser flow based on weir or orifice flow conditions, and weir flow when using either rectangular or trapezoidal (V-notch) shaped openings in the riser structure. In undeveloped areas with a natural outfall stream or channel, the broad crested weir or the V-notch weir without a riser structure is more economical and can be constructed as part of the pond embankment. Some of these flow analysis have to include structure losses, i.e. friction and various tailwater conditions. The software will include an analysis of the emergency spillway based on the designer's input data. The final product is the stage-discharge relationship shown as a curve or tabulation that sums all flows through both spillways based on the water depth in the pond. Using this data, the designer can determine if all design parameters have been met. If not adjustment in input data are made and the program rerun.

Now that the first goal of sizing the pond and determining the spillway design has been met, the other required post-development storm flows have to be routed through the pond to verify that the design also meets the criteria set for them. Normally, the analysis will include the 2-, 10-, and 100- year storms are evaluated. Current regulations set which one of these will control the pond size. The stage-discharge data for all the storms are used to confirm any predetermined pond elevations such as benches. Any requirement for providing a permanent pool volume also enters into this evaluation. Usually the 100-yr storm flow sets the emergency spillway capacity and required total depth of the pond.

As initially stated using computer software is almost a necessary in order to select the most feasible and economical design. However, the designer's input data and choices in alternative design selections are crucial in making this happen. HEC-22 and the other resources listed in Section 6-10, 11 and 12 have detailed examples for pond design.

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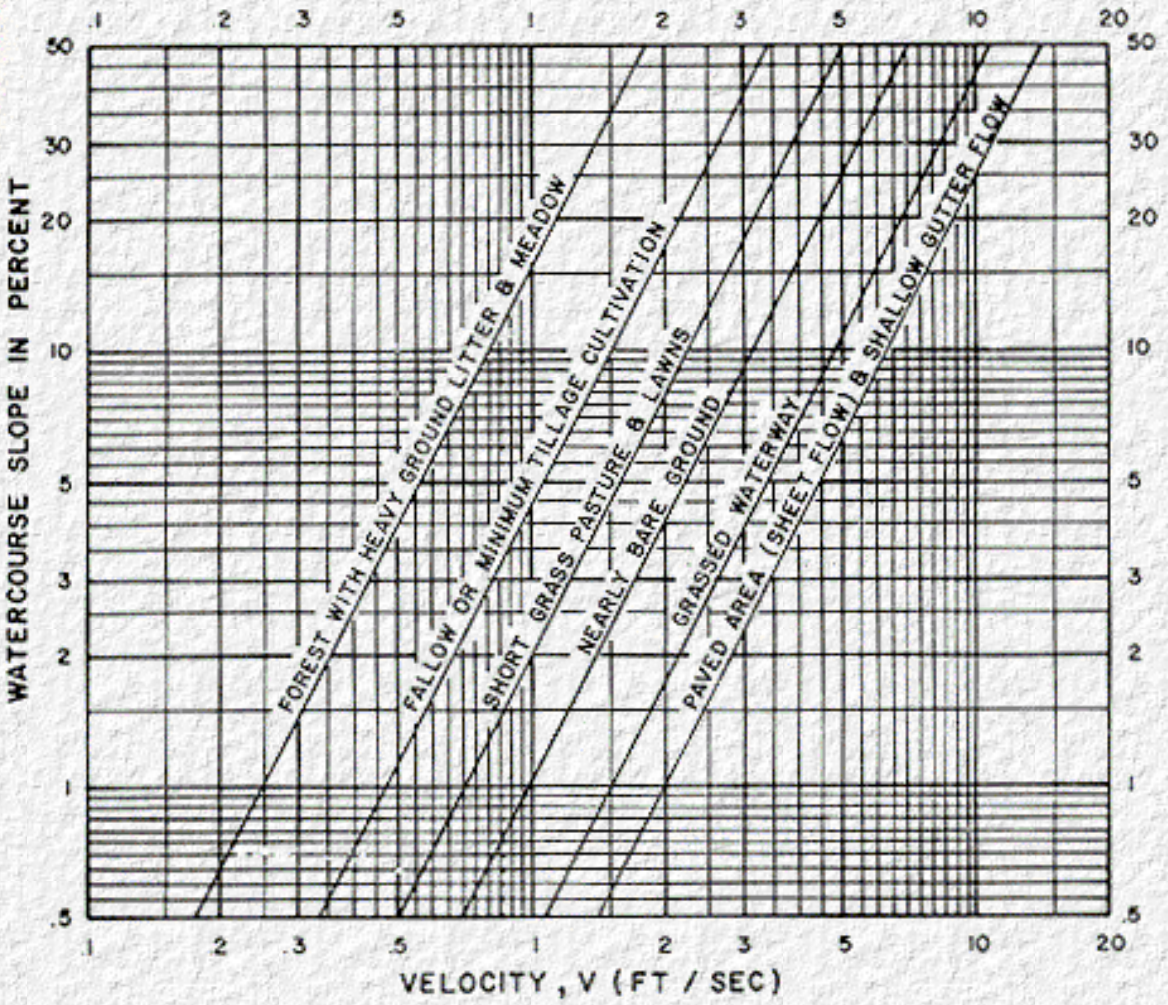
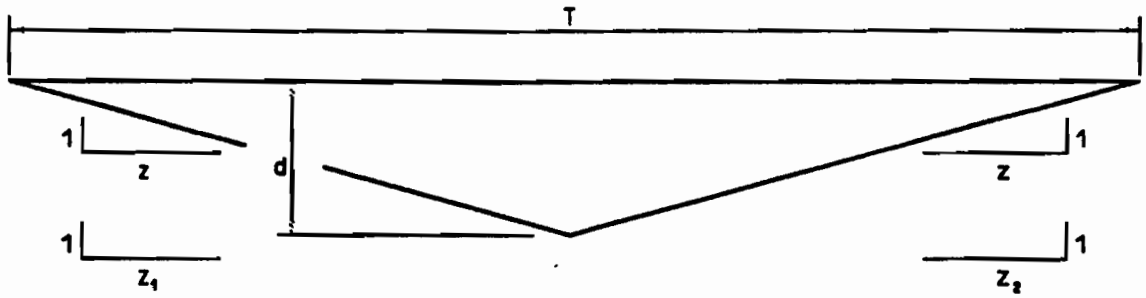


Figure 37. Velocities for Upland Method of Estimating Time of Concentration

Figure 6-20
Formulas for Various Channel Geometries



SYMMETRICAL SECTION

$A = zd^2$

$P = 2d\sqrt{z^2+1}$

$T = 2dz$

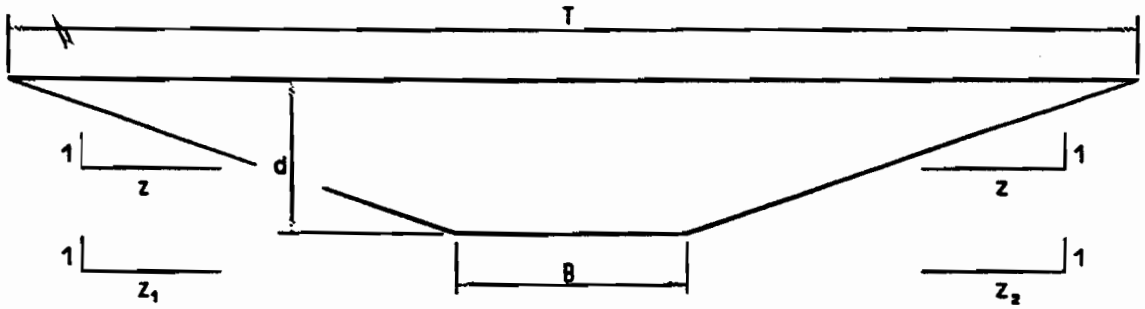
UNSYMMETRICAL SECTION

$A = (z_1+z_2)d^2/2$

$P = (\sqrt{z_1^2+1}+\sqrt{z_2^2+1})d$

$T = (z_1+z_2)d$

V - SHAPE



SYMMETRICAL SECTION

$A = Bd+zd^2$

$P = B+2d\sqrt{z^2+1}$

$T = B+2dz$

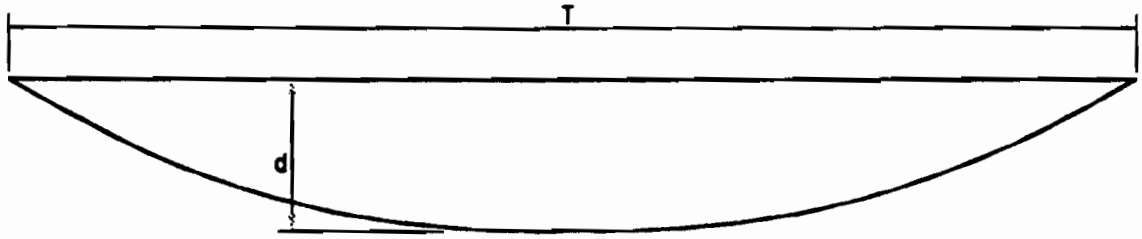
UNSYMMETRICAL SECTION

$A = Bd+(z_1+z_2)d^2/2$

$P = B+d(\sqrt{z_1^2+1}+\sqrt{z_2^2+1})$

$T = B+d(z_1+z_2)$

TRAPEZOIDAL



$$A = \frac{2}{3}Td$$

$$P = T+\frac{8d^2}{3T}$$

$$T = \frac{3}{2}\frac{A}{d}$$

PARABOLIC

Appendix 7C-1 Nomograph for Solution of Manning's Equation

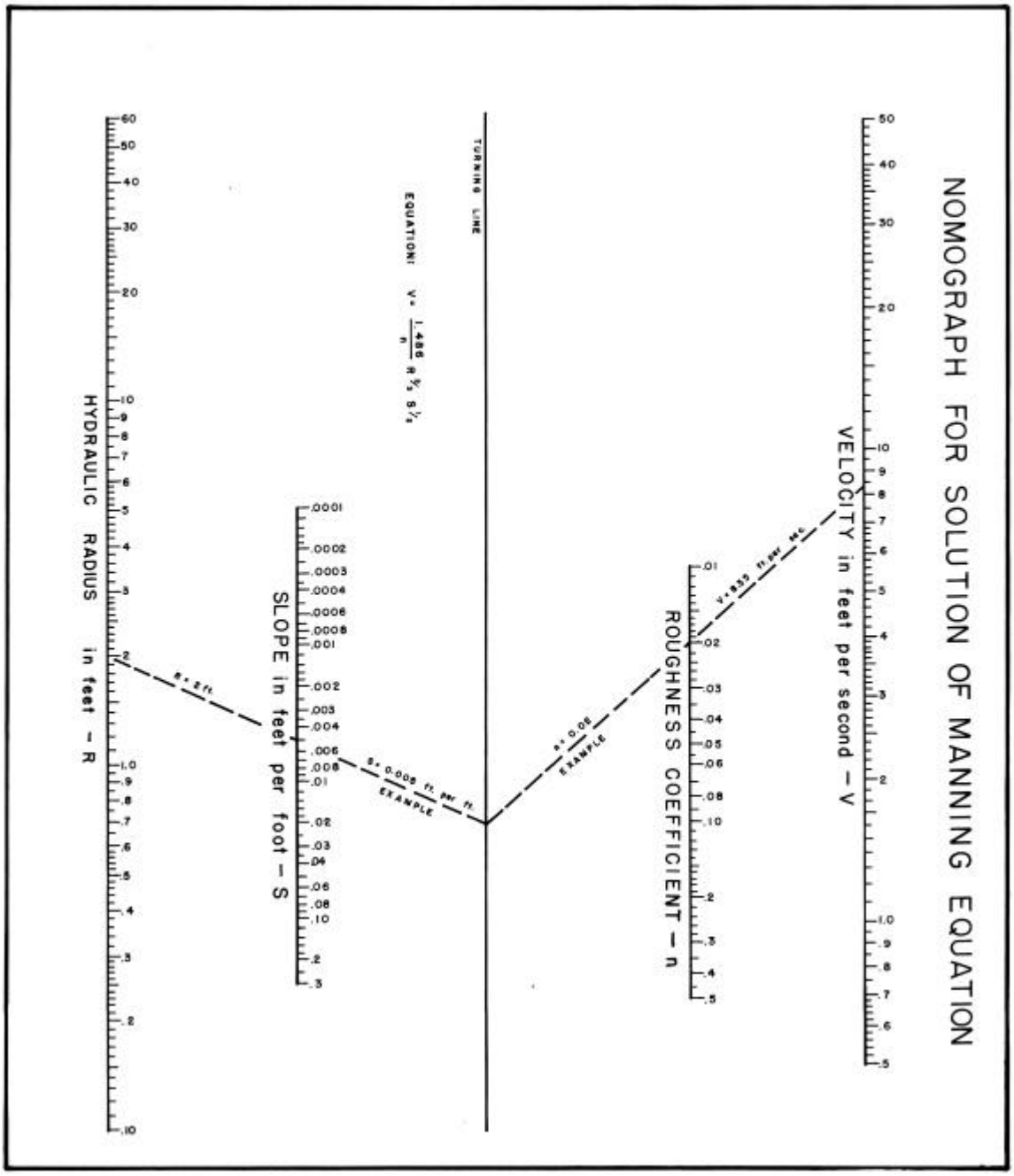
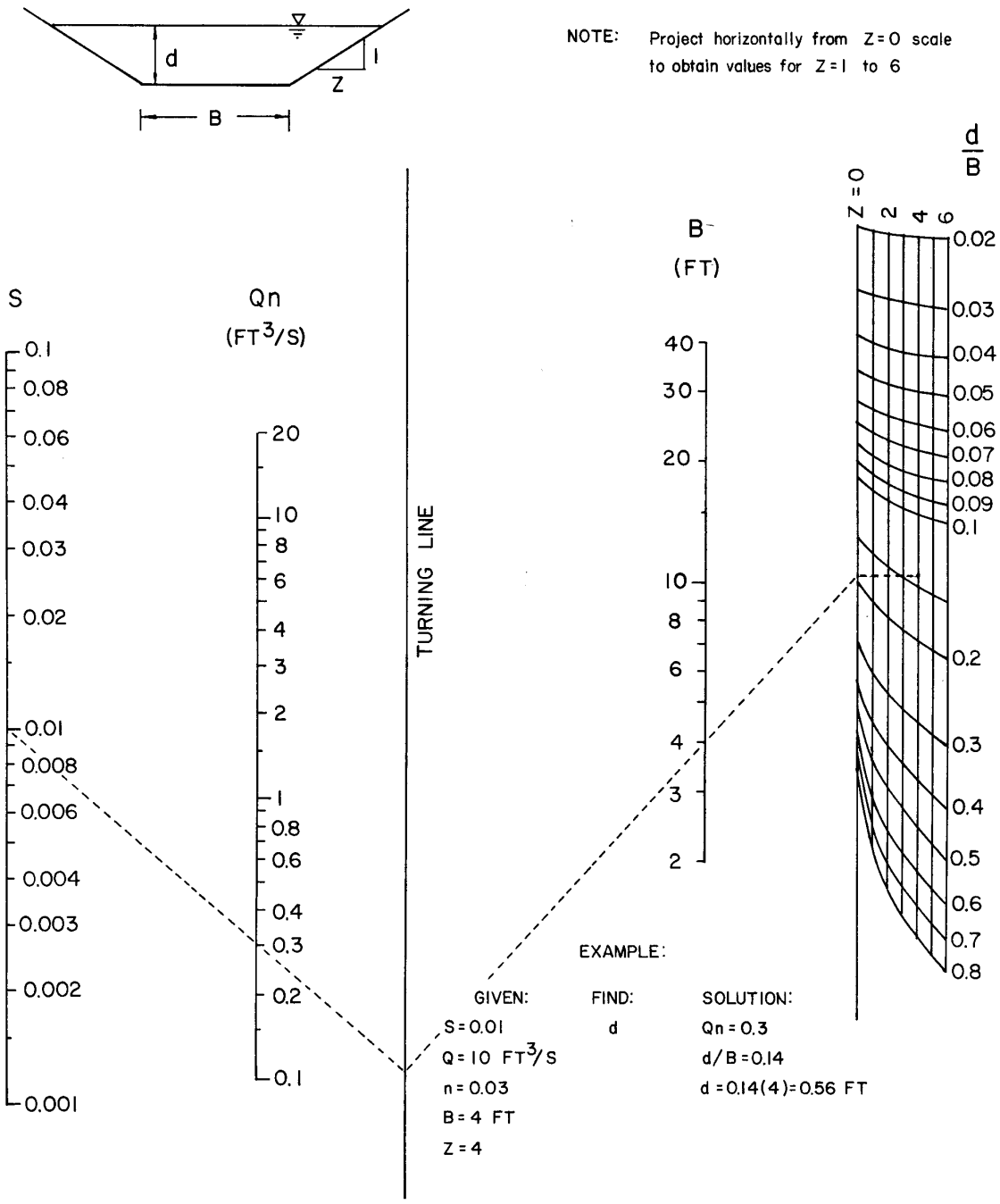


CHART 14B



Solution to Manning's Equation for Channels of Various Side Slopes - English Units

Figure 6-23
Capacity of Trapezoidal Section

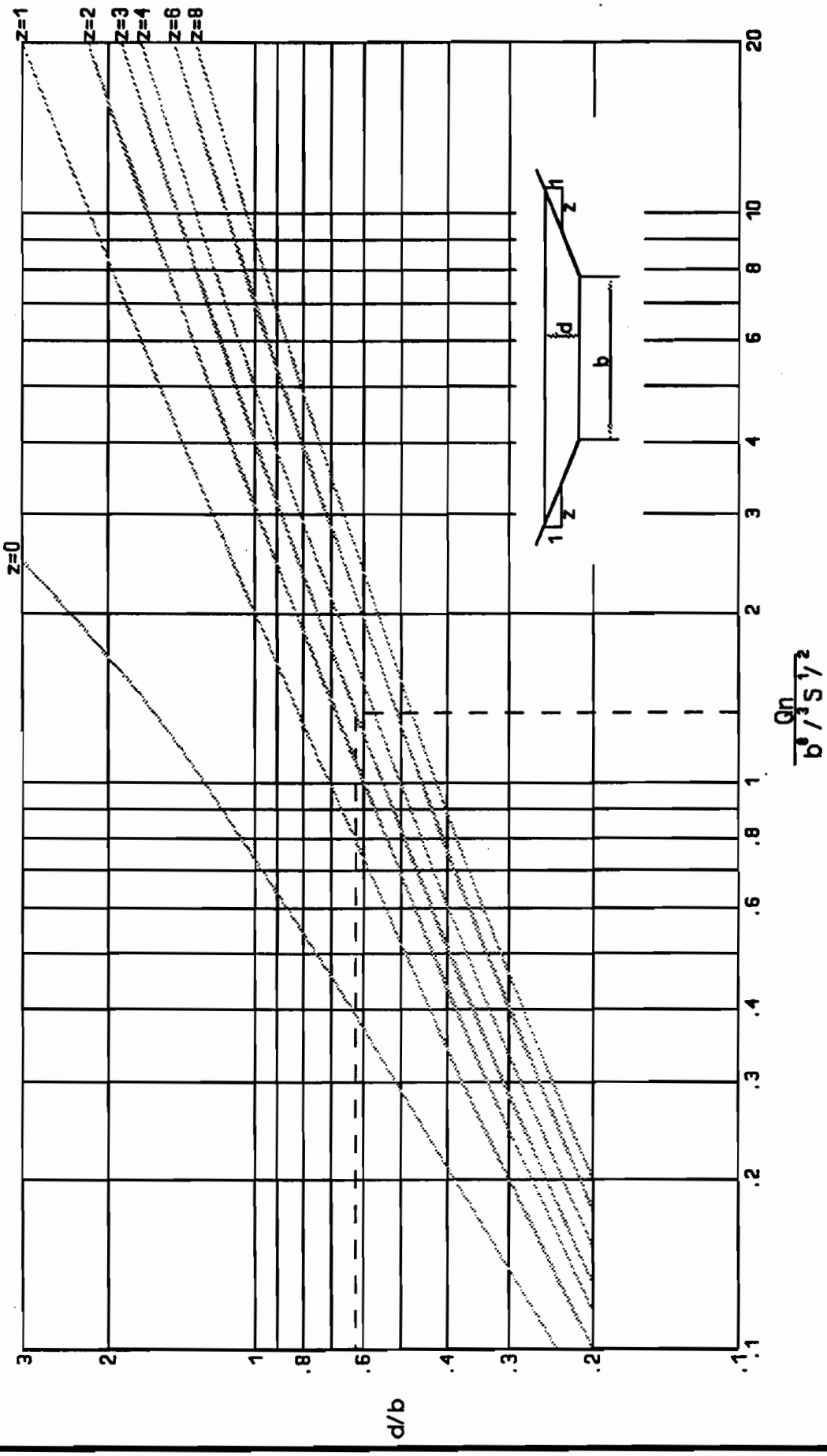
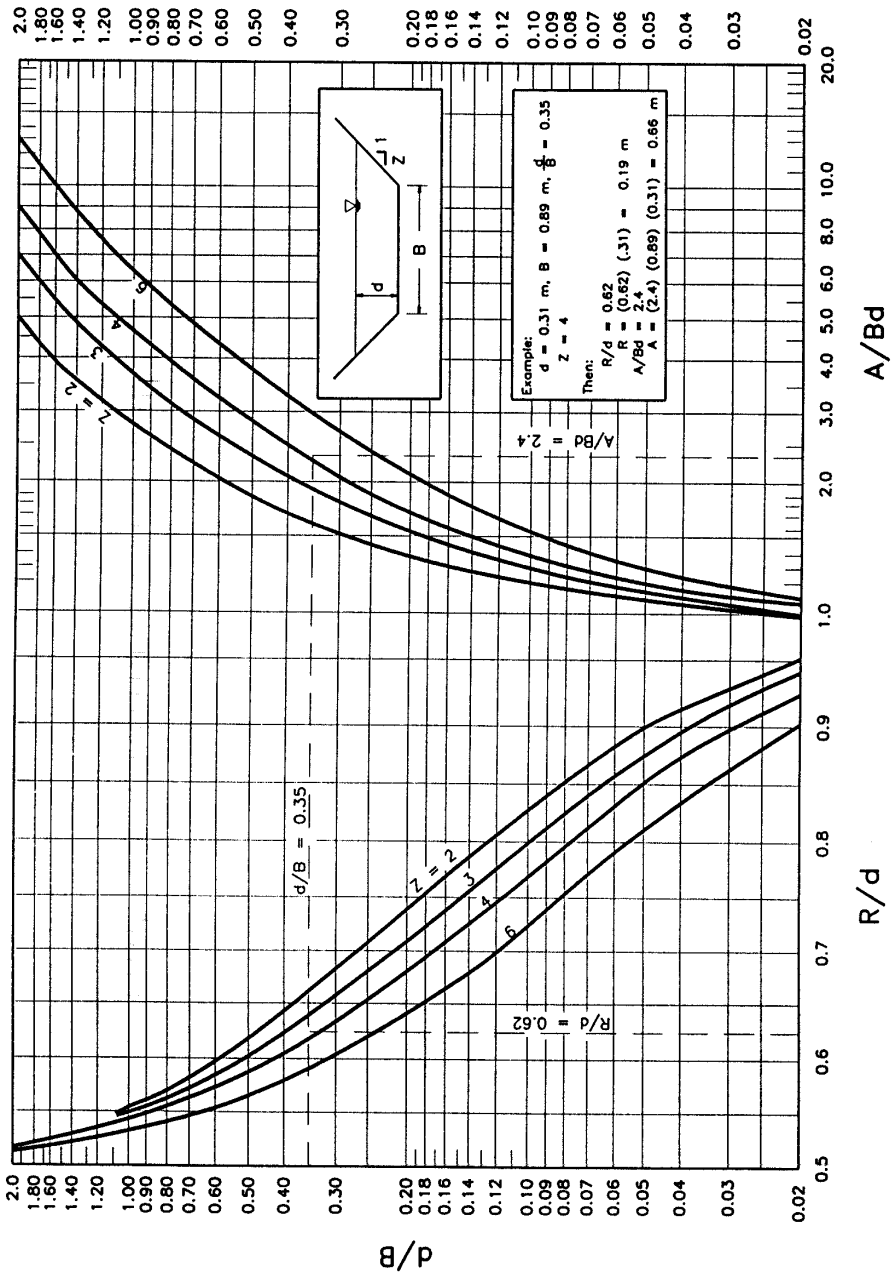
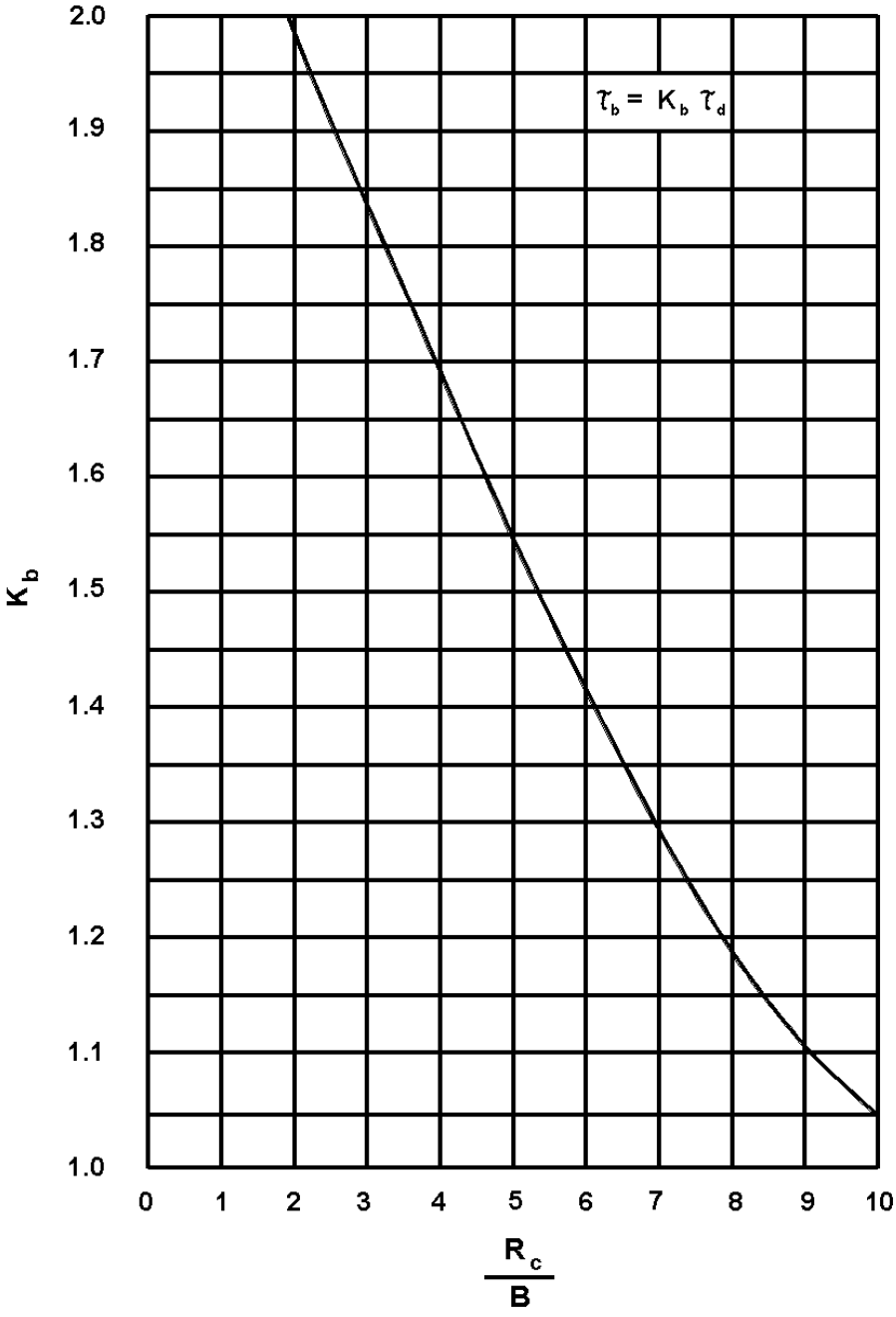


CHART 22



Geometric Design Chart For Trapezoidal Channels.

CHART 21



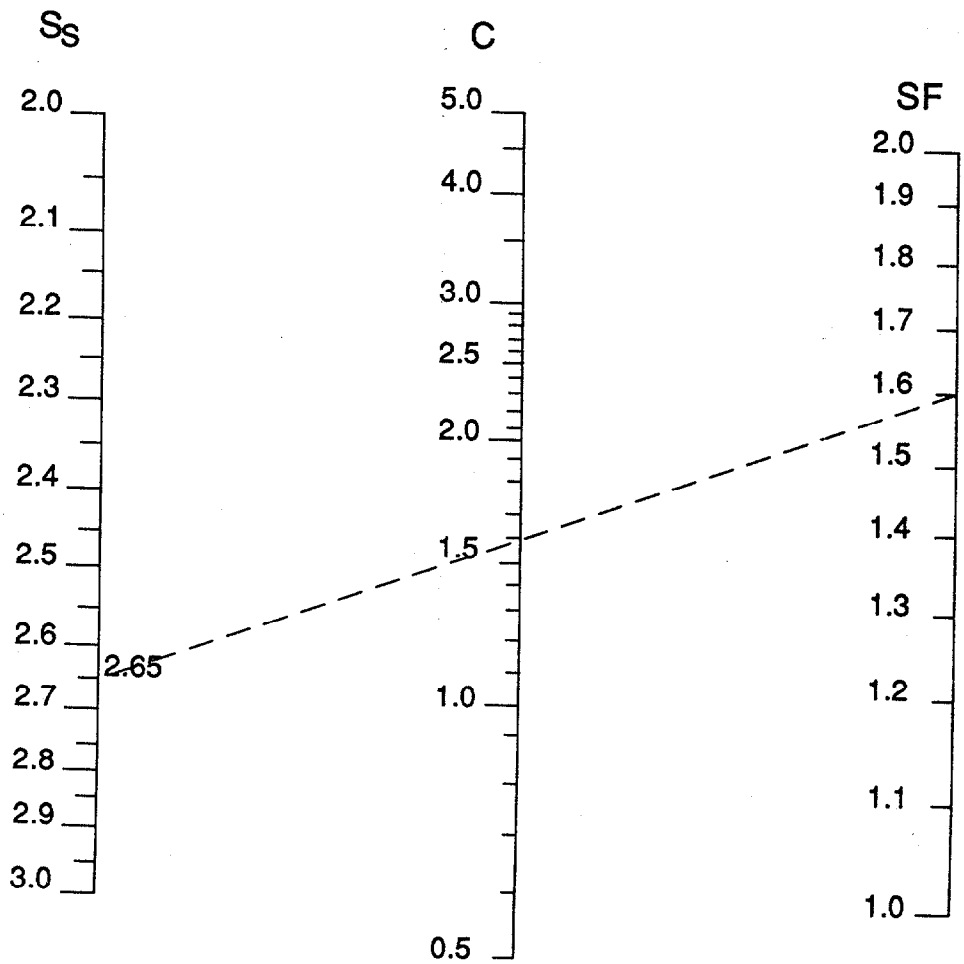
K_b Factor For Maximum Shear Stress On Channel Bends. (12)

Figure 6-30
Criteria for Selection of Stability Factors

Condition	Stability Factor Range
Uniform flow; Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius/channel width > 10); Impact from waves or floating debris moderate.	1.3 - 1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1 to 2 feet or 0.31 to 0.60 meters); High flow turbulence; Significant uncertainty in design parameters.	1.6 - 2.0

$$C=1.61SF^{1.5}/(S_S-1)^{1.5}$$

CORR=D_{50}CORRECTION FACTOR
SF = STABILITY FACTOR
S_S= SPECIFIC GRAVITY OF ROCK



Example:

Given:
S_S=2.75
SF= 1.60

Find:
C

Solution:
C=1.59

Chart 2. Correction factor for riprap size

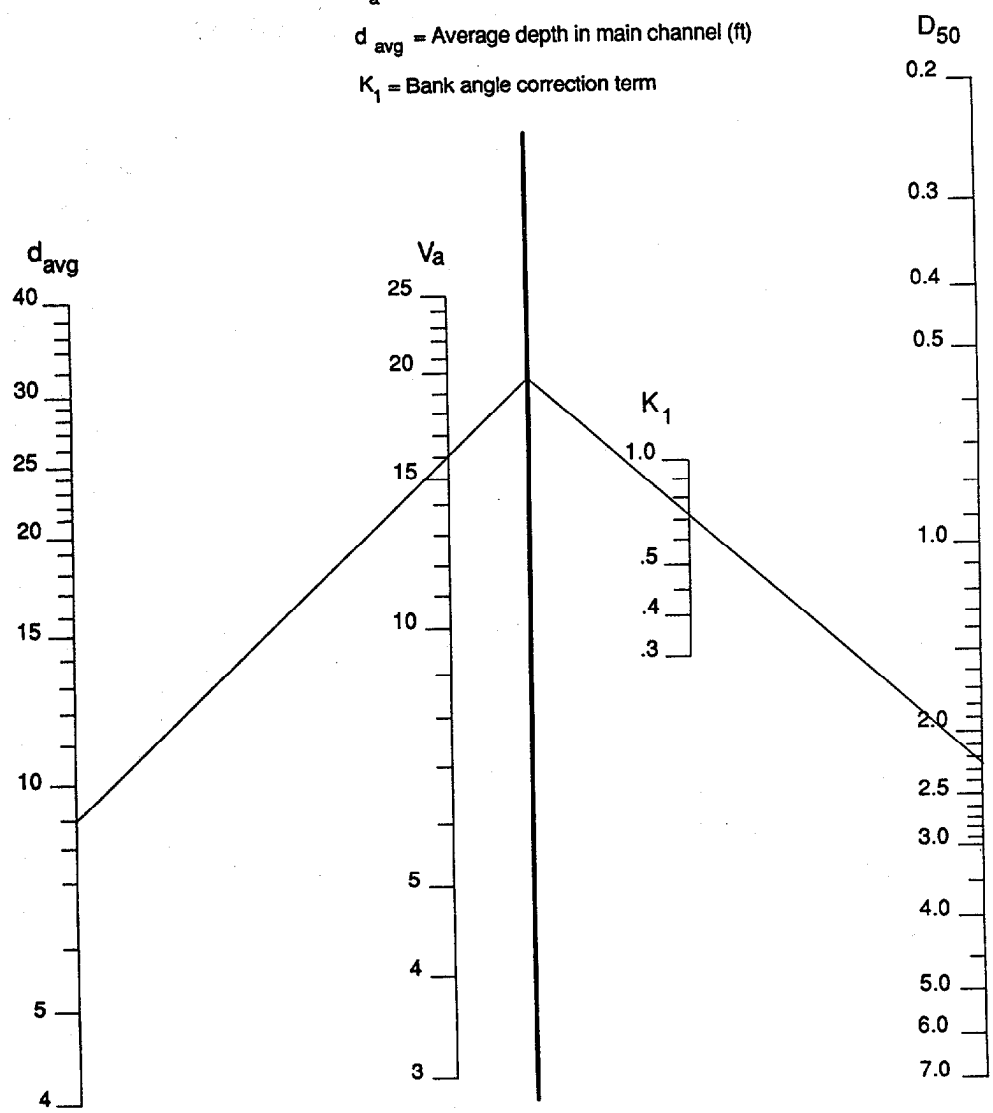
$$D_{50} = 0.001\, V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$$

D_{50} = Median Riprap Size (ft.)

V_a = Average velocity in main channel (ft/sec)

d_{avg} = Average depth in main channel (ft)

K_1 = Bank angle correction term



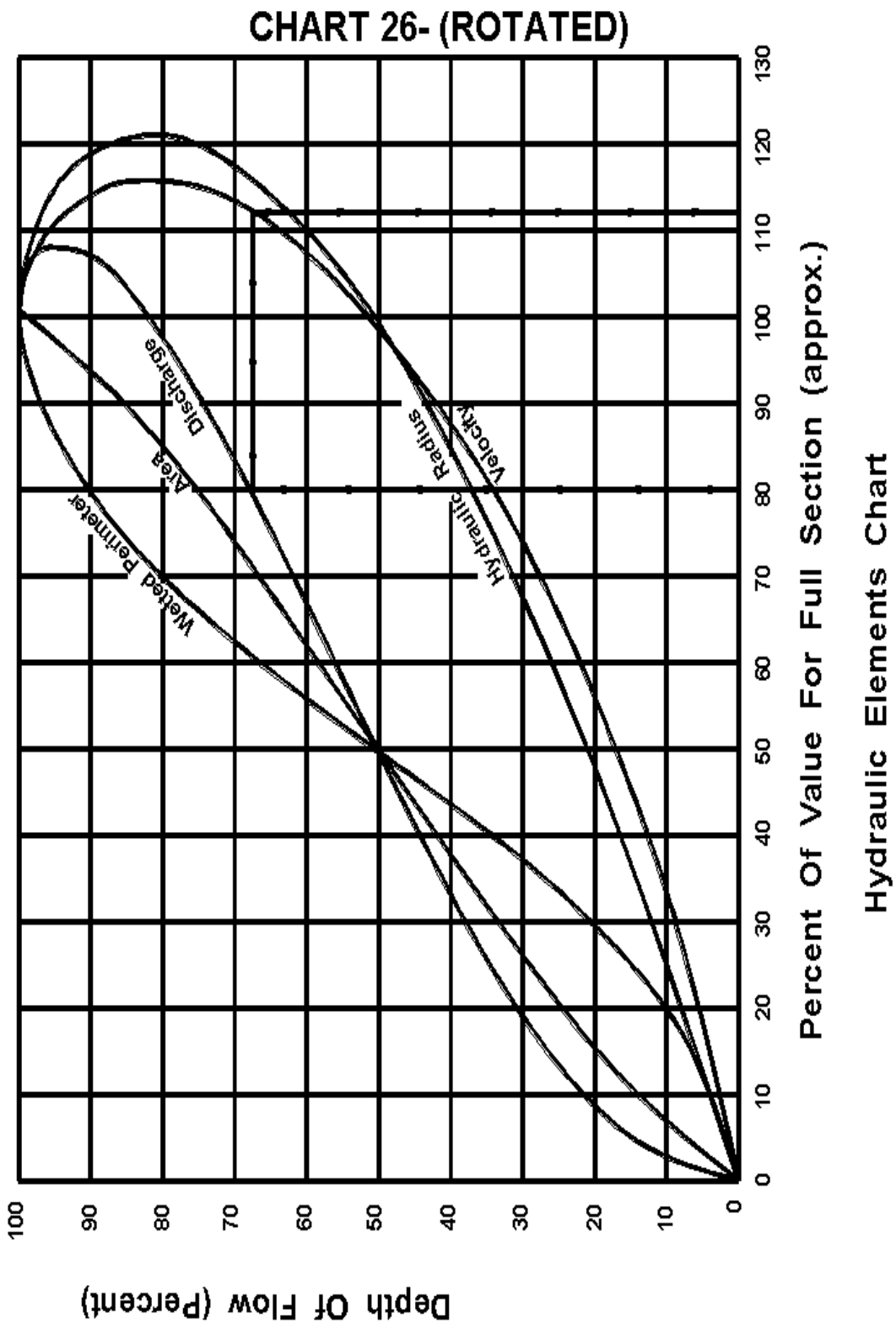
Example

Given:
V_a = 16 ft/sec
d_{avg} = 9 ft
K_1 = 0.72

Find:
D_{50}

Solution:
D_{50} = 2.25

Chart 1. Riprap size relationship



Hydraulic Elements Chart

Figure 6-51
Flow Coefficients for Circular Pipe

	Values of K_1									
$\frac{d}{D}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0		0.00007	0.00031	0.00074	0.00138	0.00222	0.00328	0.00455	0.00604	0.0077
0.1	0.00967	0.0118	0.0142	0.0167	0.0195	0.0225	0.0257	0.0291	0.0327	0.0366
0.2	0.0406	0.0448	0.0492	0.0537	0.0585	0.0634	0.0686	0.0738	0.0793	0.0849
0.3	0.0907	0.0966	0.1027	0.1089	0.1153	0.1218	0.1284	0.1352	0.1420	0.1490
0.4	0.1561	0.1633	0.1705	0.1779	0.1854	0.1929	0.2005	0.2082	0.2160	0.2238
0.5	0.232	0.239	0.247	0.255	0.263	0.271	0.279	0.287	0.295	0.303
0.6	0.311	0.319	0.327	0.335	0.343	0.350	0.358	0.366	0.373	0.380
0.7	0.388	0.395	0.402	0.409	0.416	0.422	0.429	0.435	0.441	0.447
0.8	0.453	0.459	0.463	0.468	0.473	0.477	0.481	0.485	0.488	0.491
0.9	0.494	0.496	0.497	0.498	0.498	0.498	0.496	0.494	0.489	0.483
1.0	0.463									

$$K_1 = \frac{Q\,n}{D^{8/3}\,S^{1/2}}$$

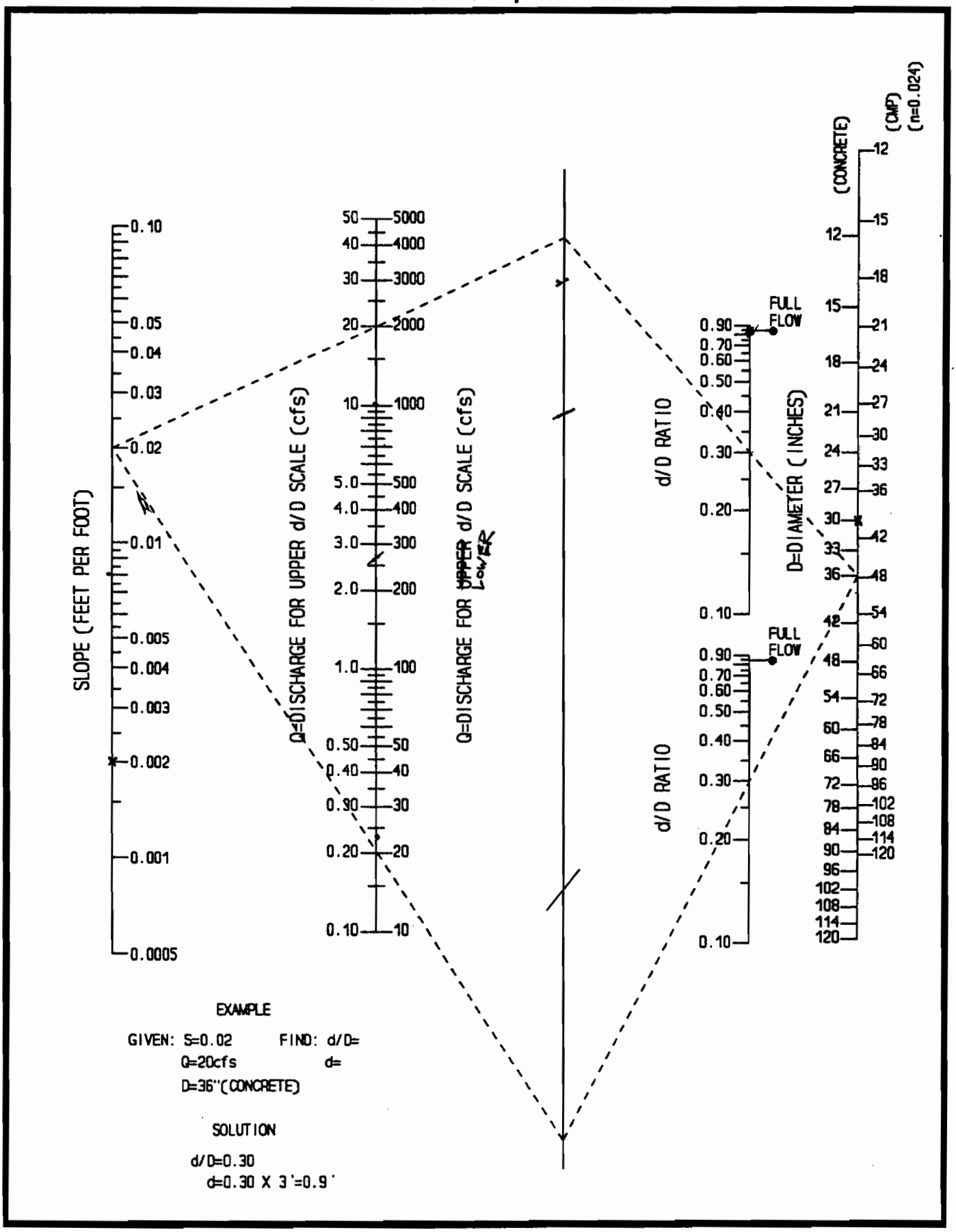
Where,

D = Diameter of pipe
d = Depth of flow
Q = Flow (cfs)
n = Manning's coefficient
S = Slope (feet/foot)

To determine Q:

1. Obtain K_1 from the Figure for the given d/D ratio.
2. Solve the equation for Q.

Figure 6-52
Uniform Flow for Pipe Culverts



EXAMPLE

GIVEN: S=0.02 FIND: d/D=
Q=20cfs d=
D=36"(CONCRETE)

SOLUTION

d/D=0.30
d=0.30 X 3'=0.9'

Figure 6-53
Uniform Flow for Concrete Elliptical Pipes

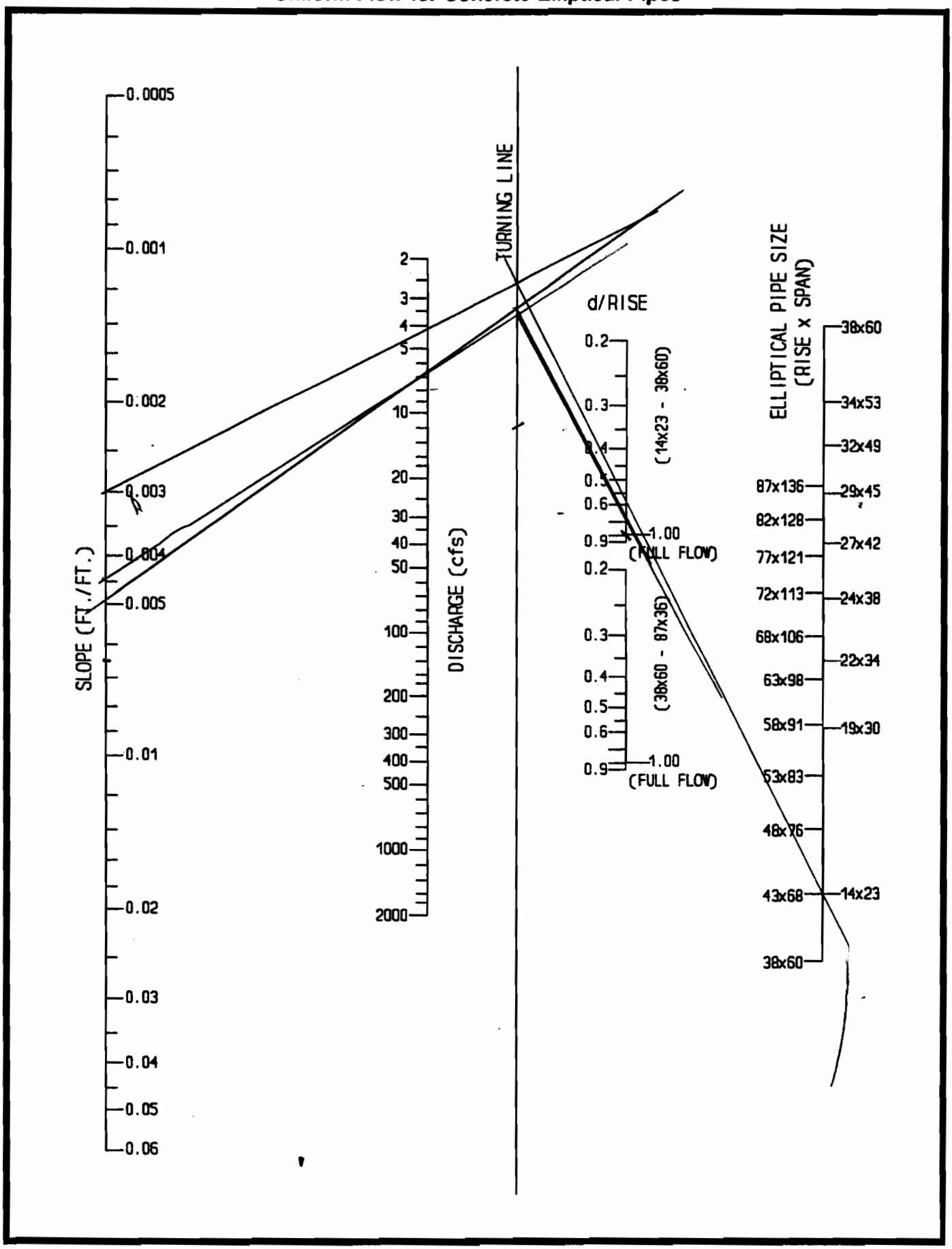


Figure 6-54
Uniform Flow for Pipe Arches

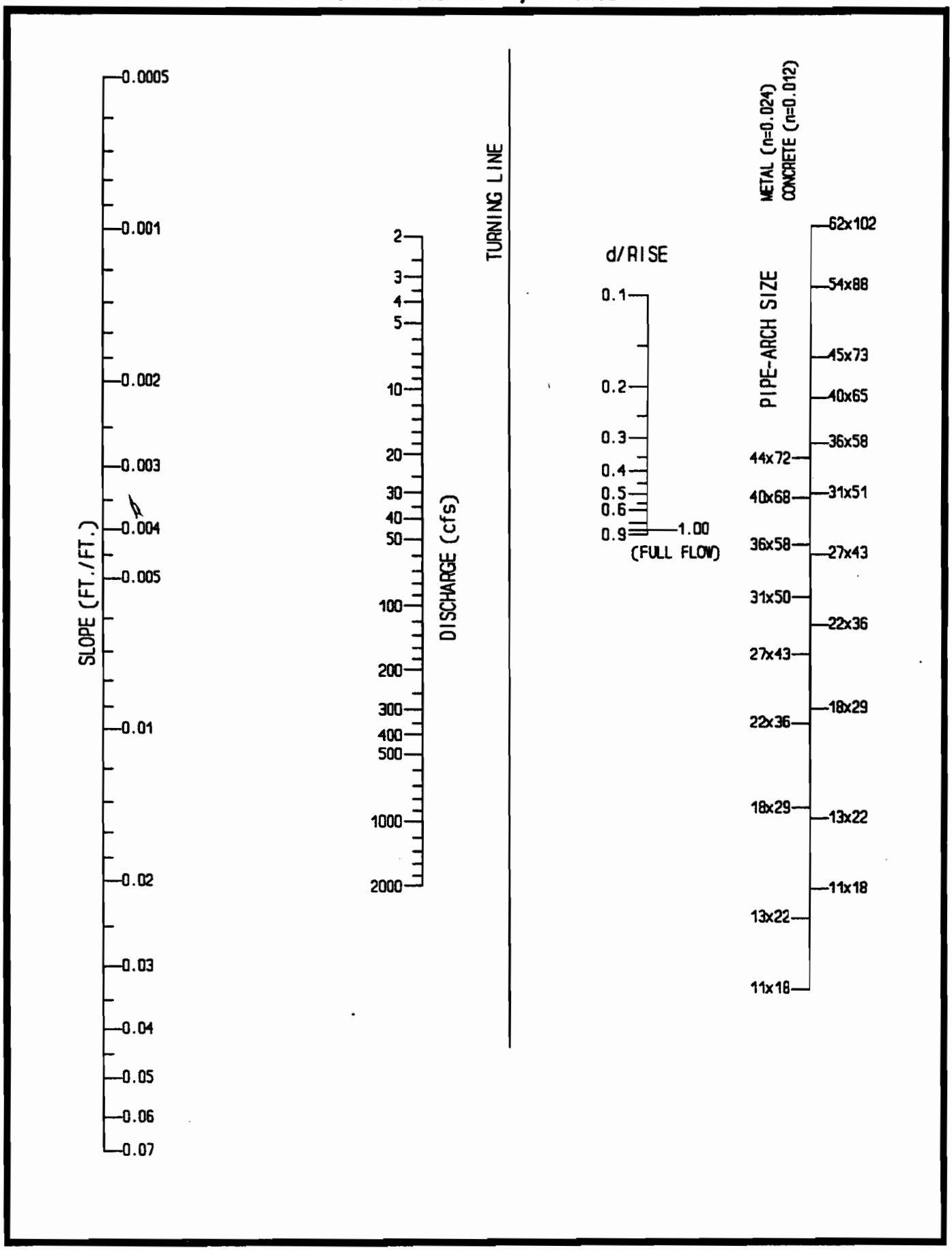


Figure 6-55
Velocity in Pipe Conduits

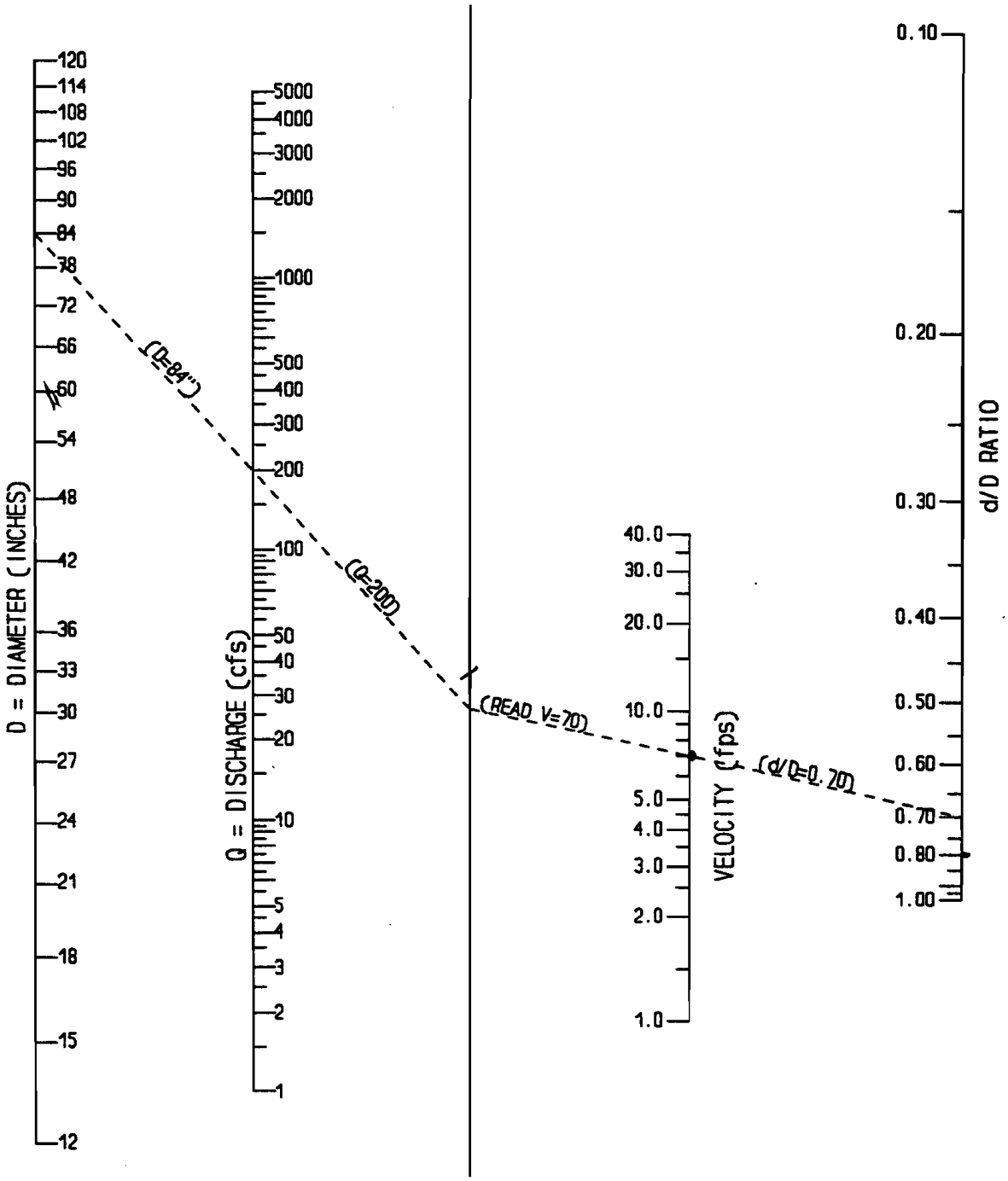


Figure 6-56
Velocity in Elliptical Pipes

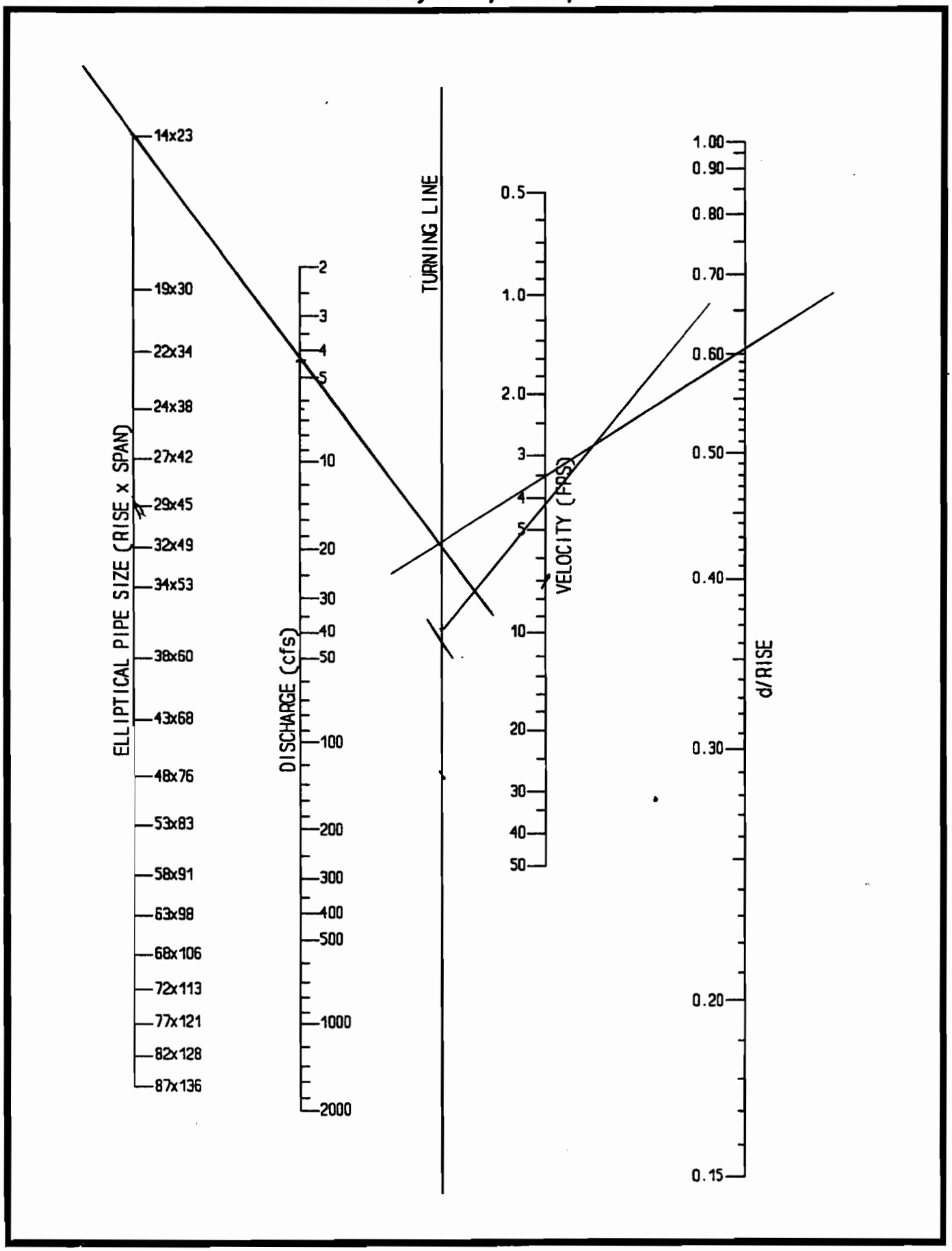


Figure 57
Velocity in Pipe Arches

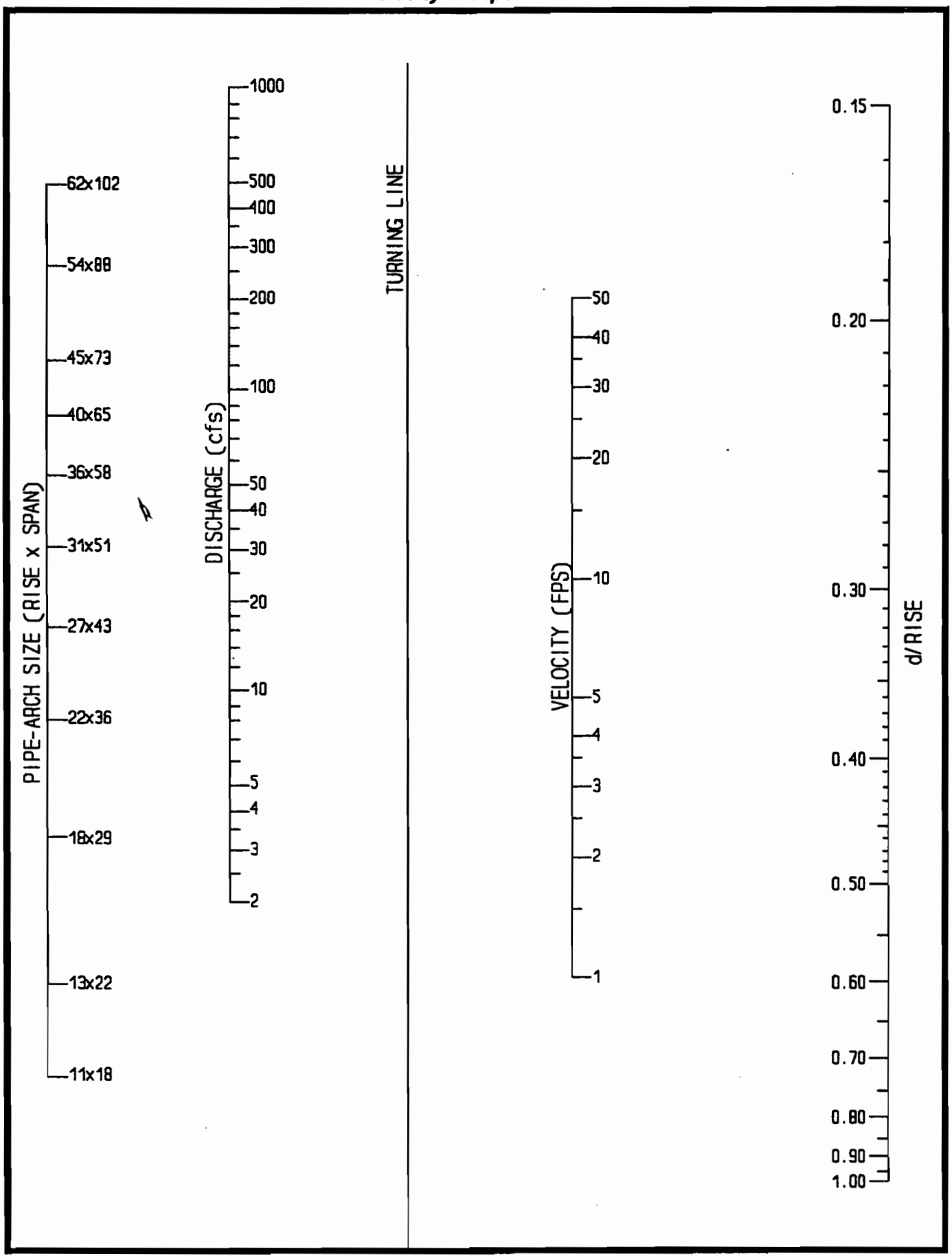


Figure 6-58
Critical Depth of Flow for Circular Conduits

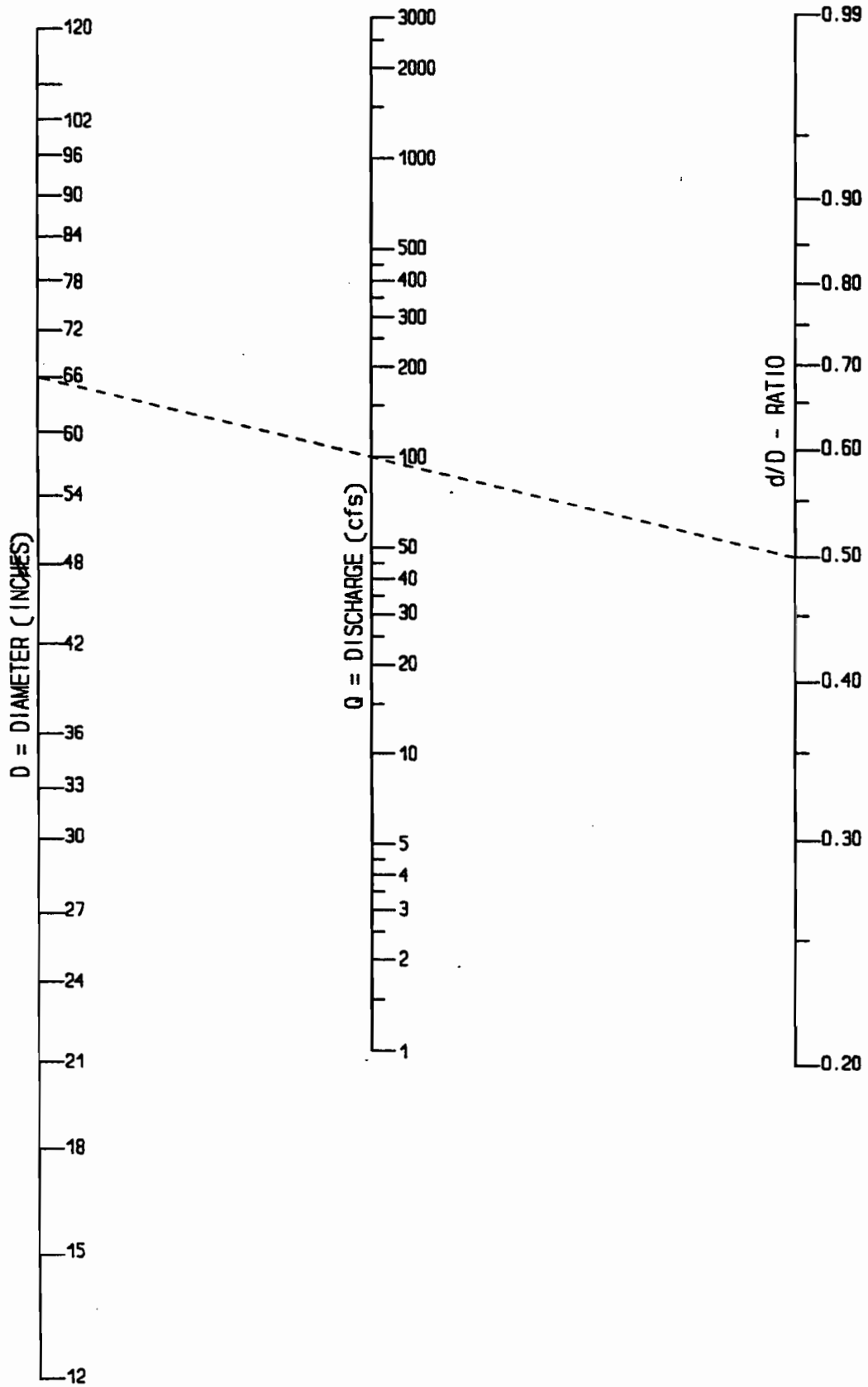


Figure 6-59
Critical Depth of Flow for Elliptical Pipes

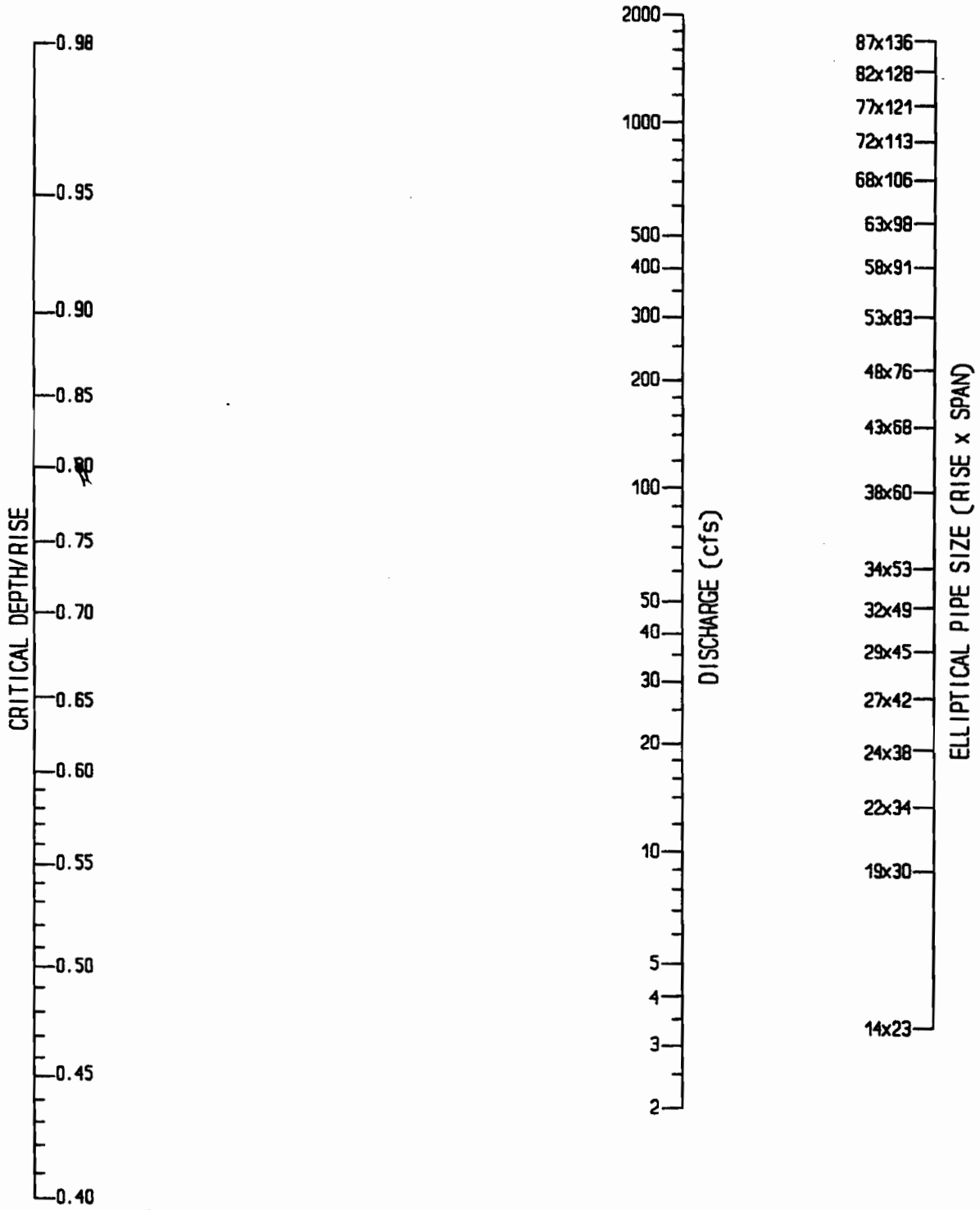


Figure 6-60
Critical Depth of Flow for Pipe Arches

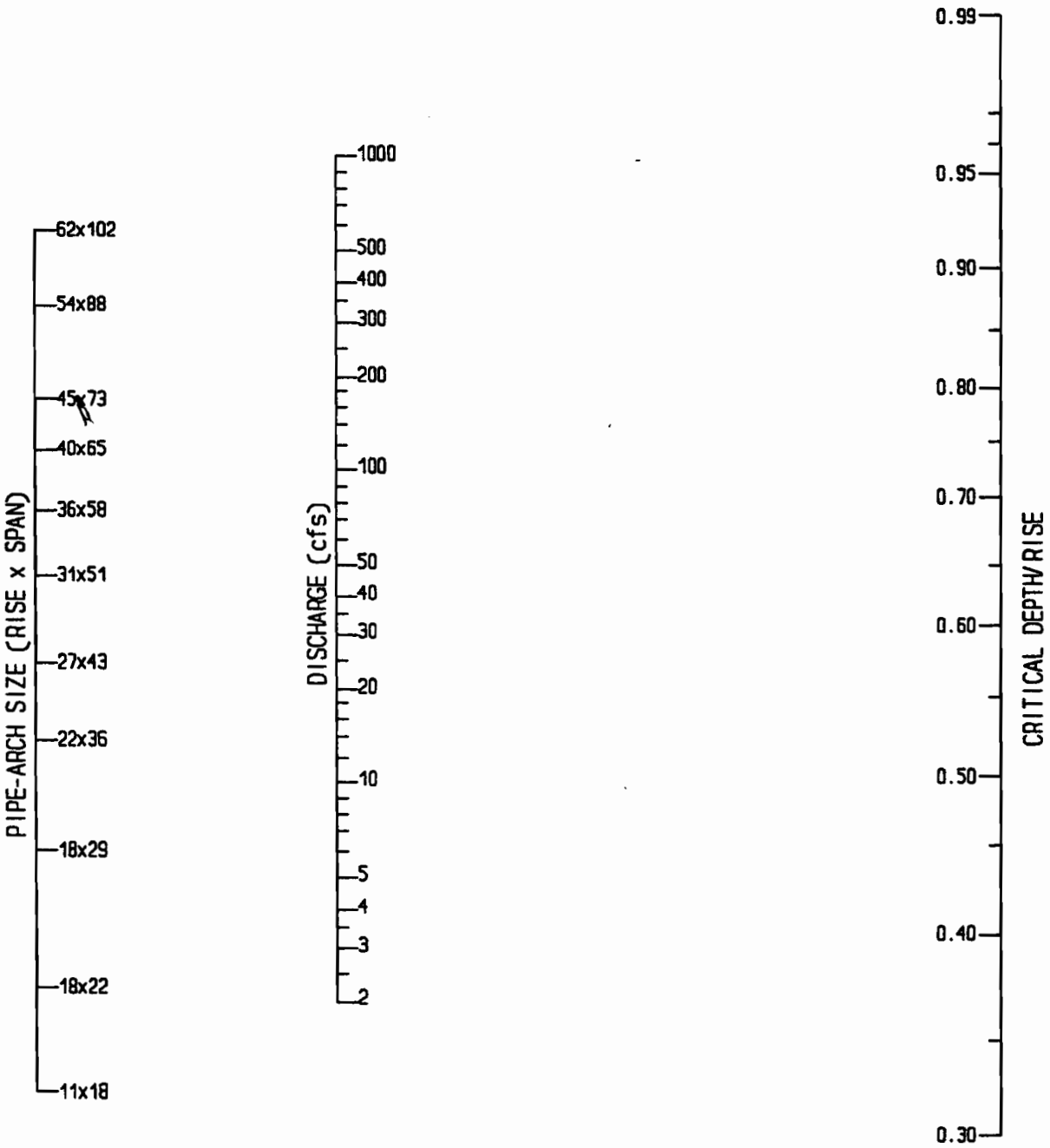
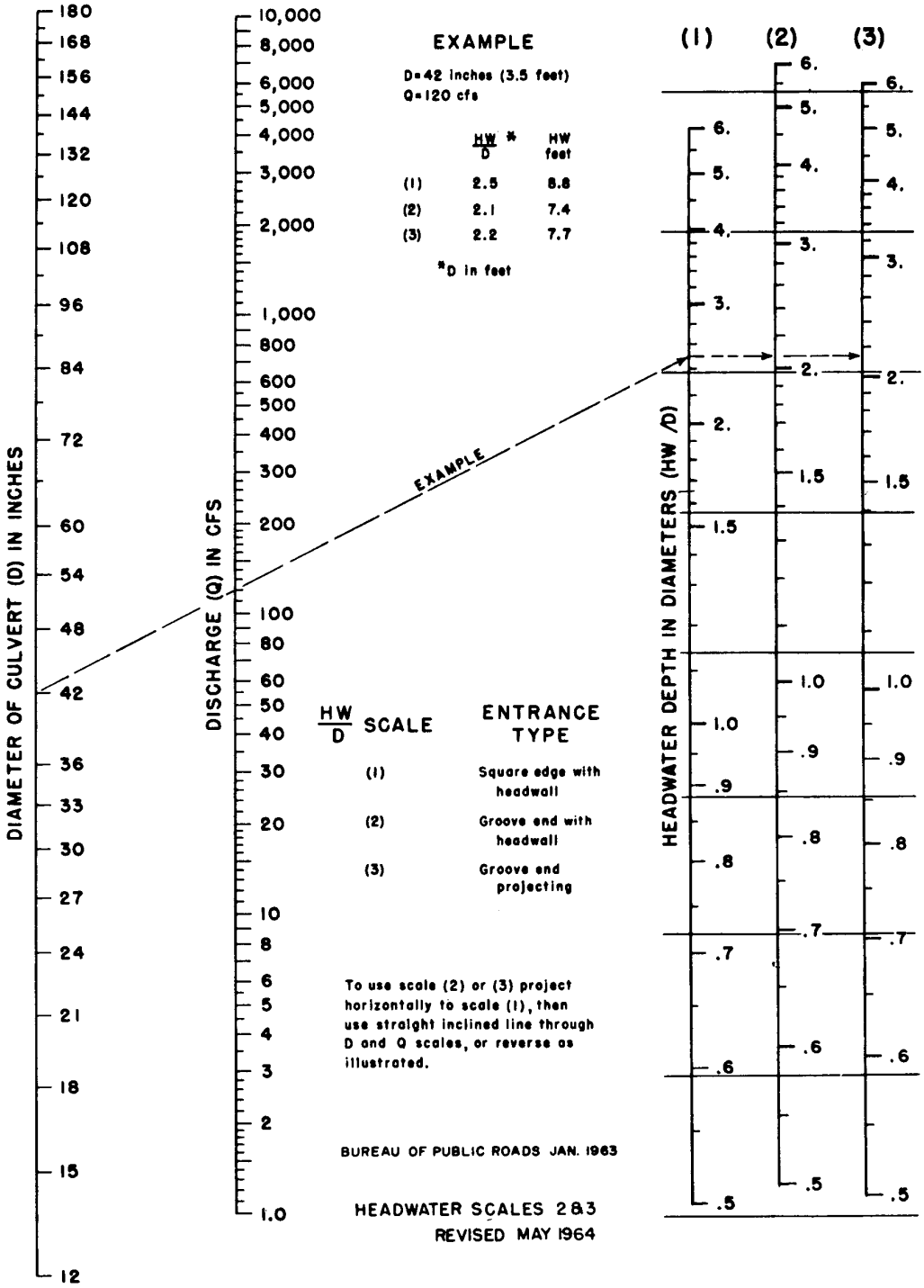
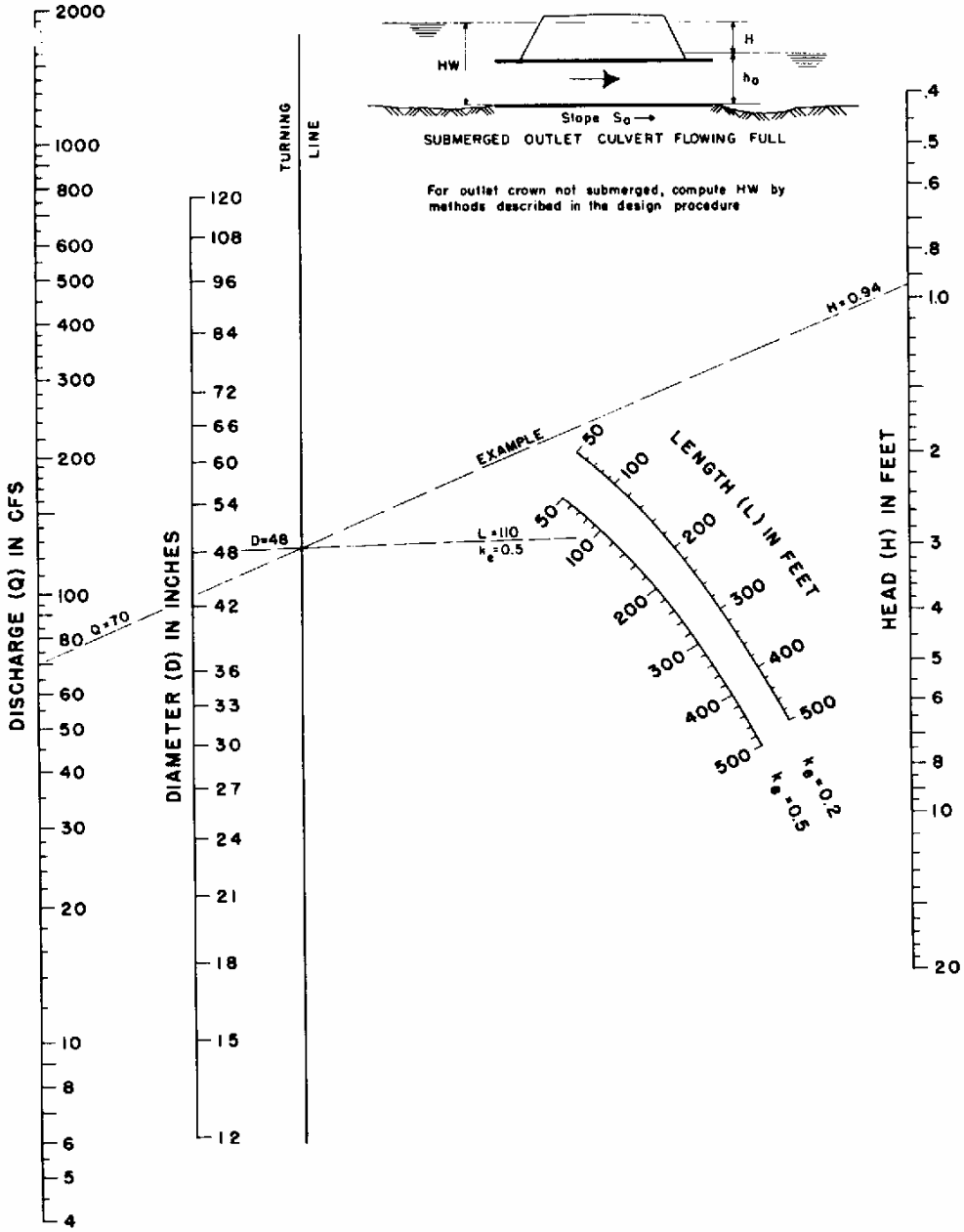


CHART 28B



Headwater Water Depth for Concrete Pipe Culverts
with Inlet Control - English Units

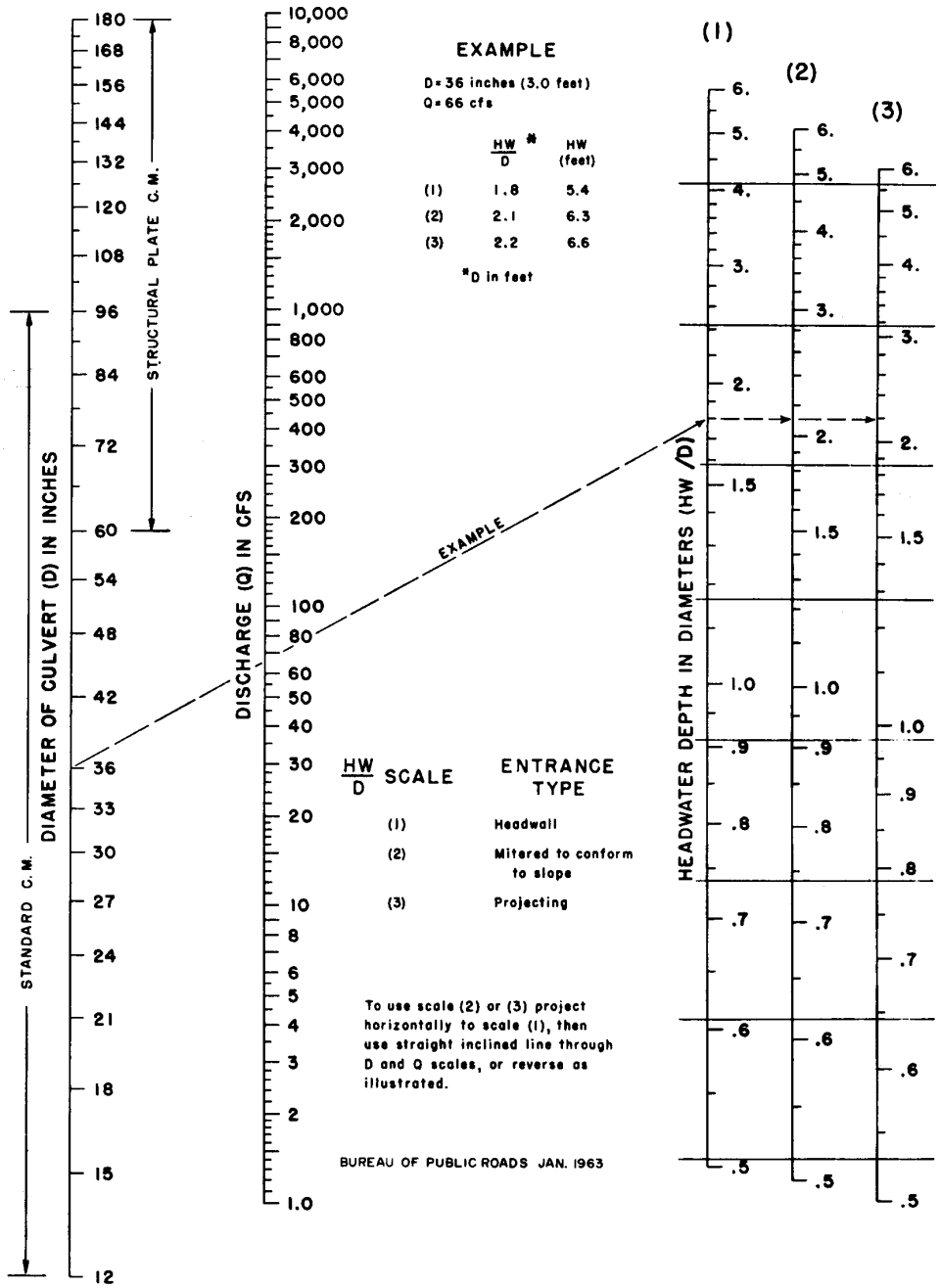
CHART 5B



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

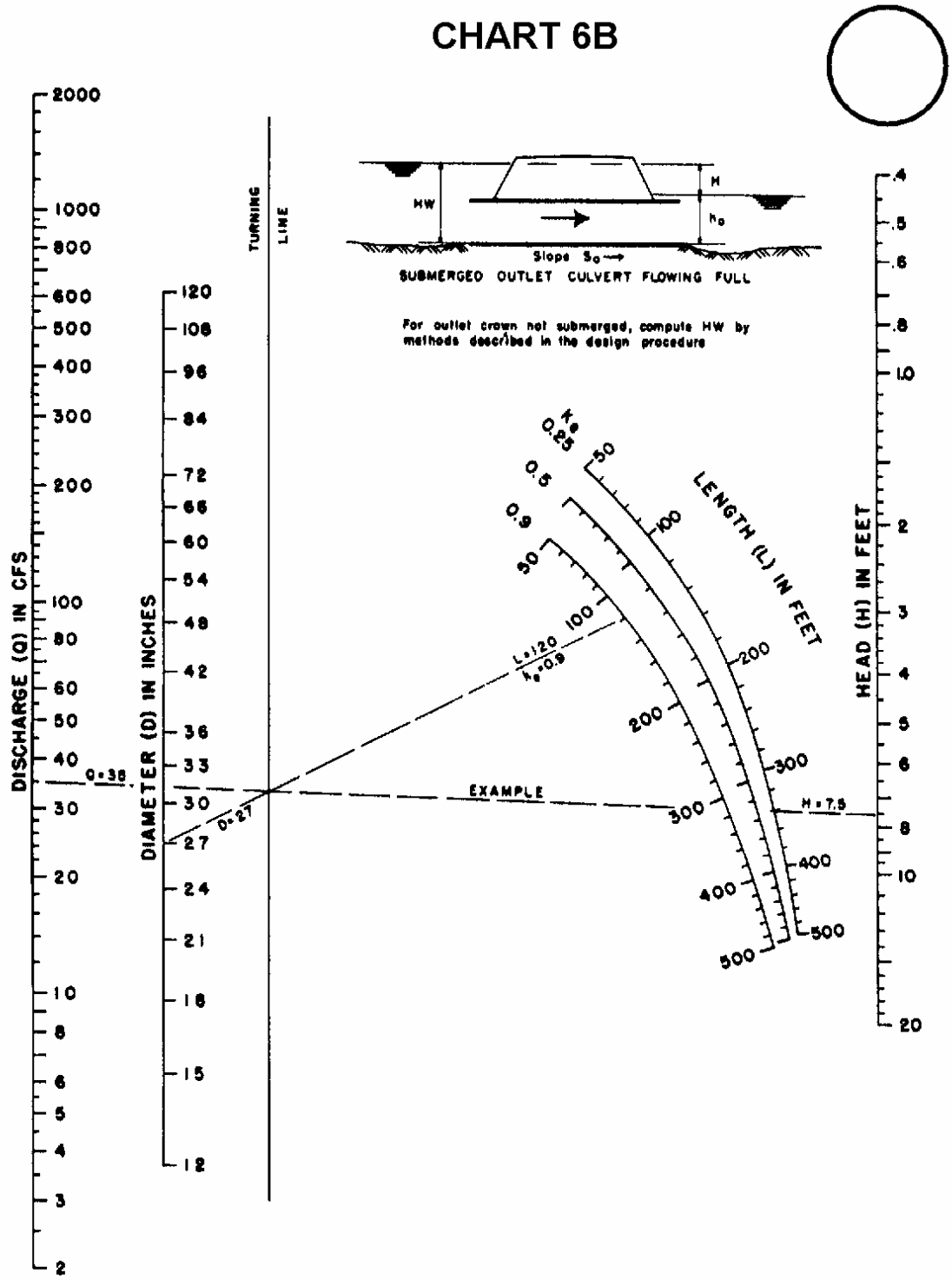
CHART 29B



Headwater Depth for C. M. Pipe Culverts with Inlet Control - English Units

A - 51

CHART 6B



HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963

Figure 6-67
Headwater Depth for Concrete Arch Culverts with Inlet Control

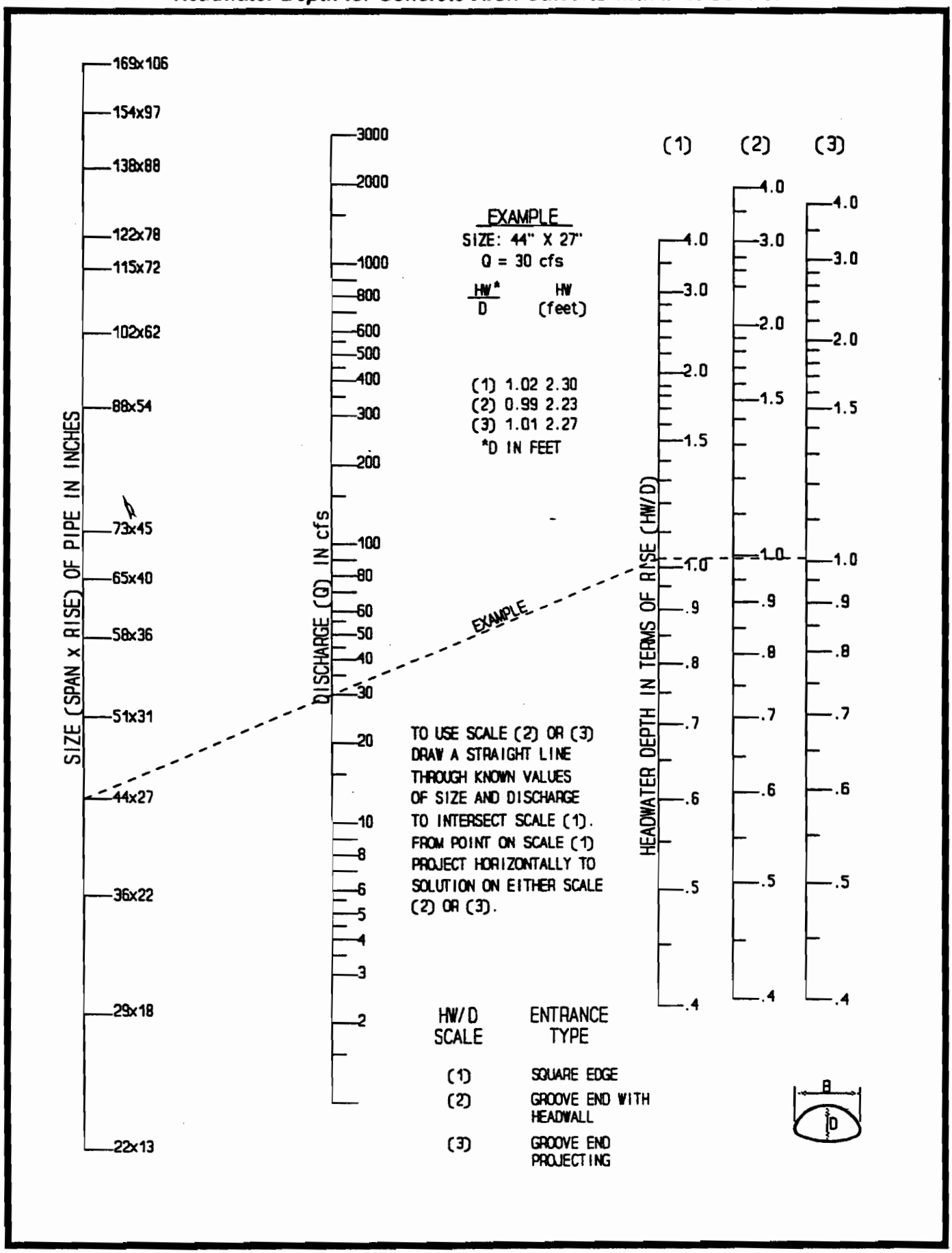


Figure 6-68
Head for Concrete Arch Culverts Flowing Full

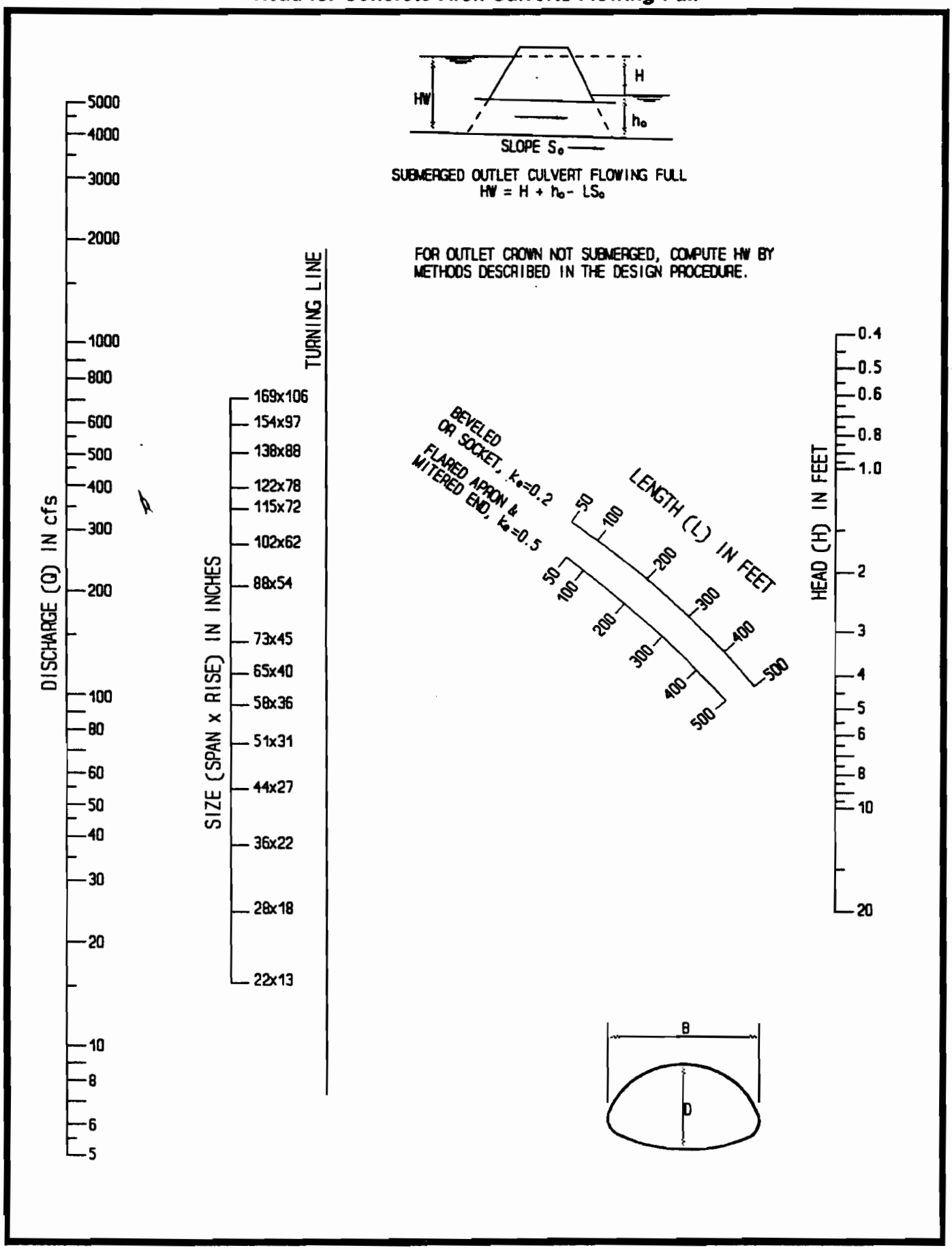
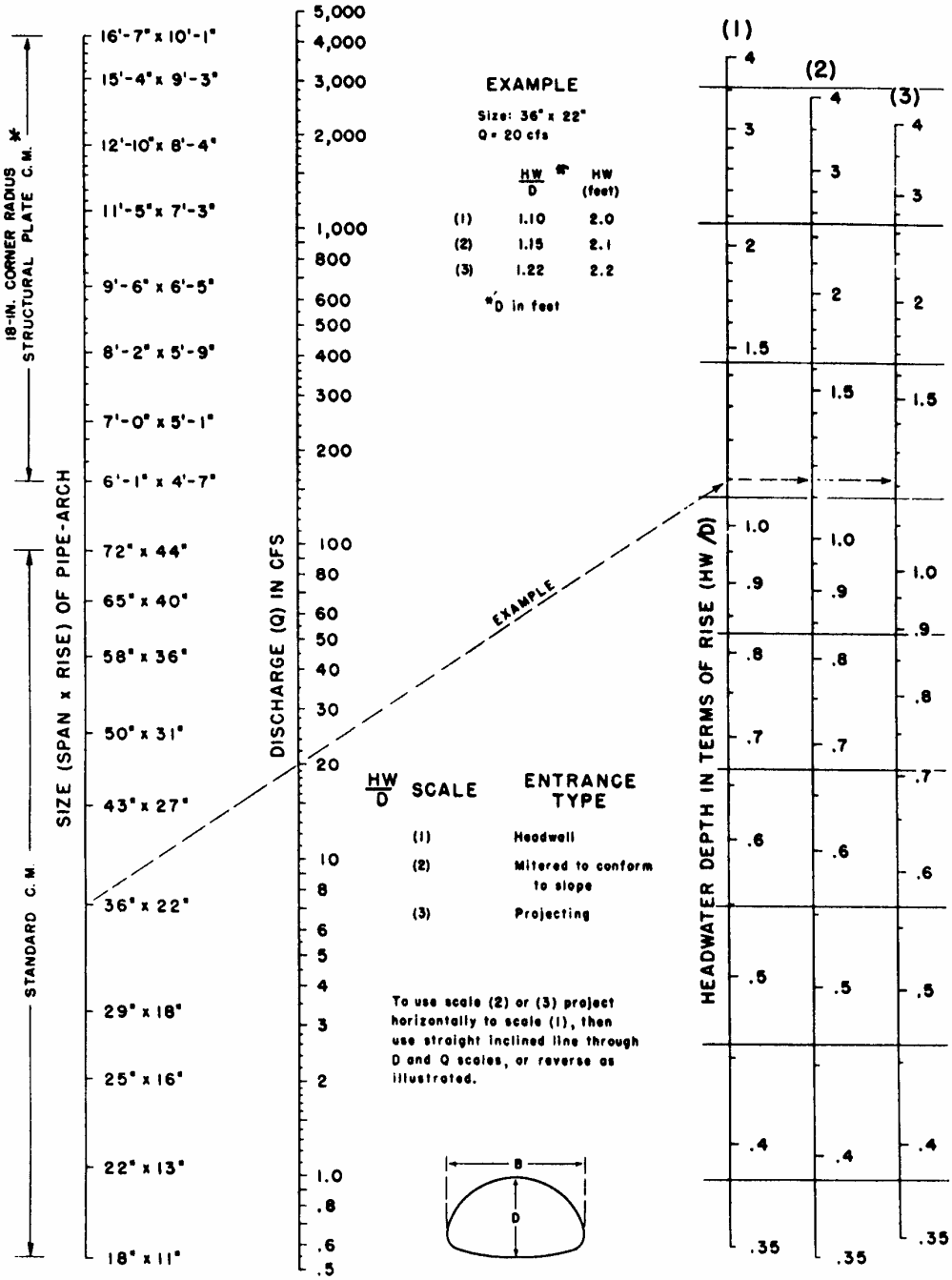


CHART 34B

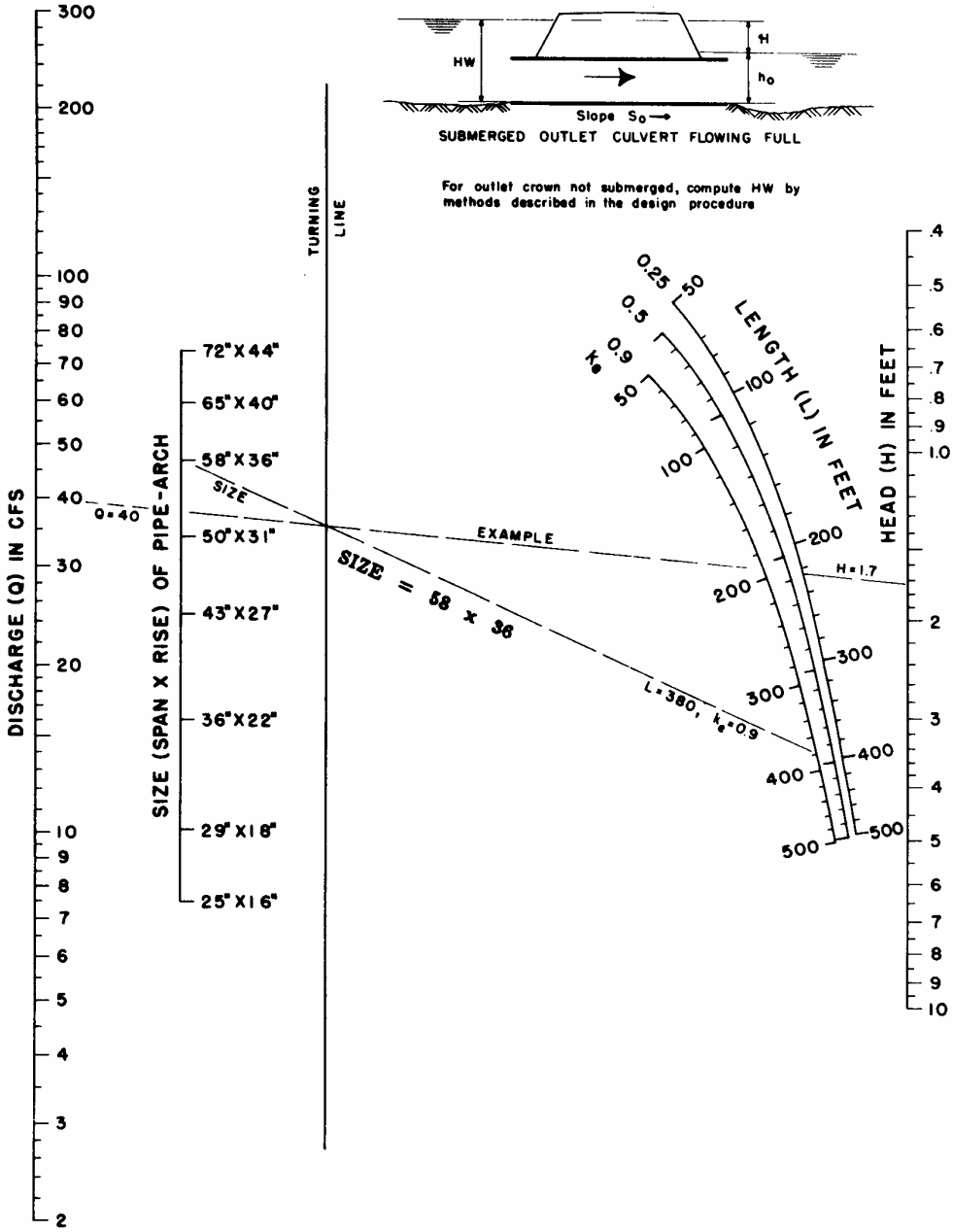


*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

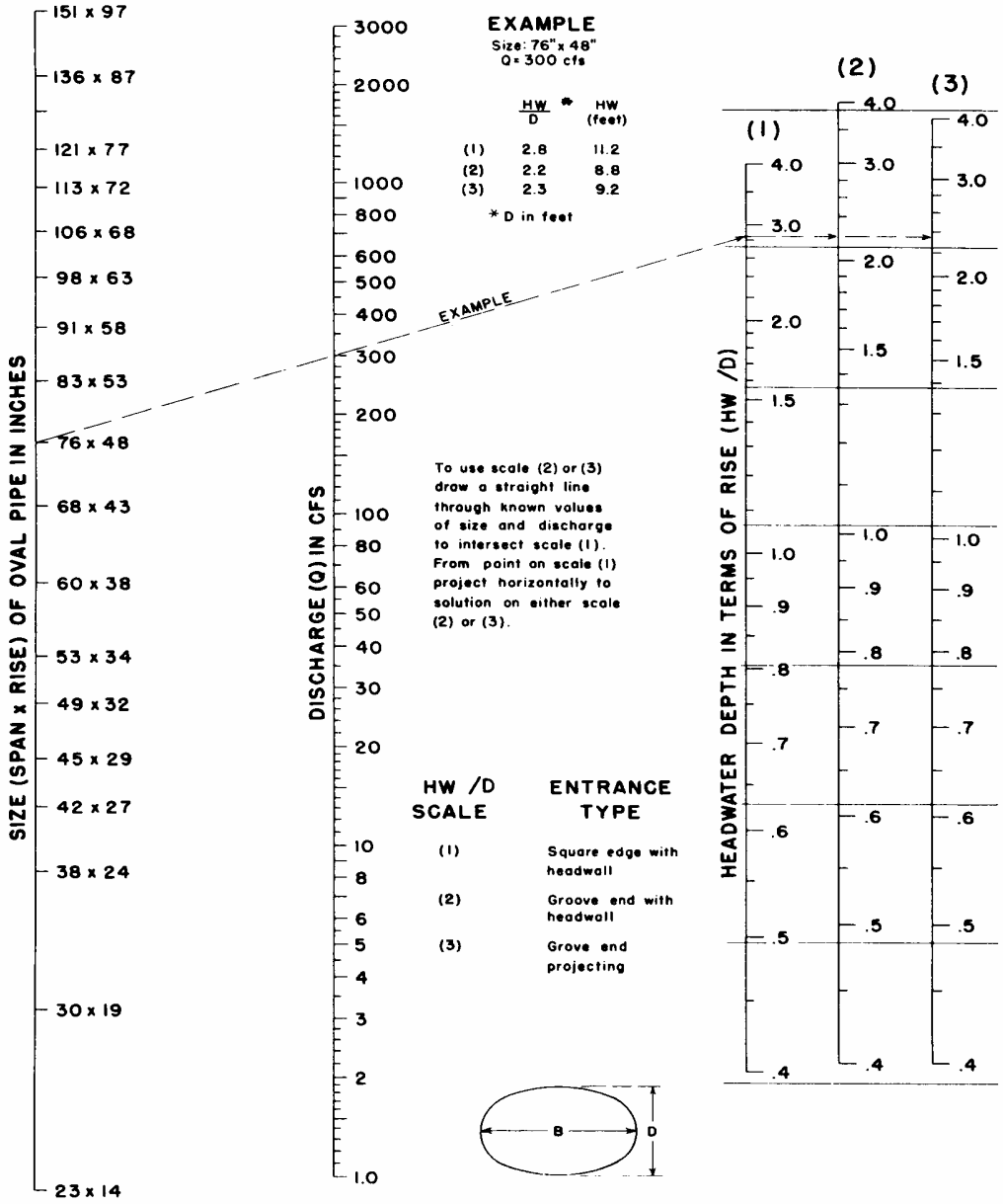
CHART 39B



HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n=0.024

BUREAU OF PUBLIC ROADS JAN. 1963

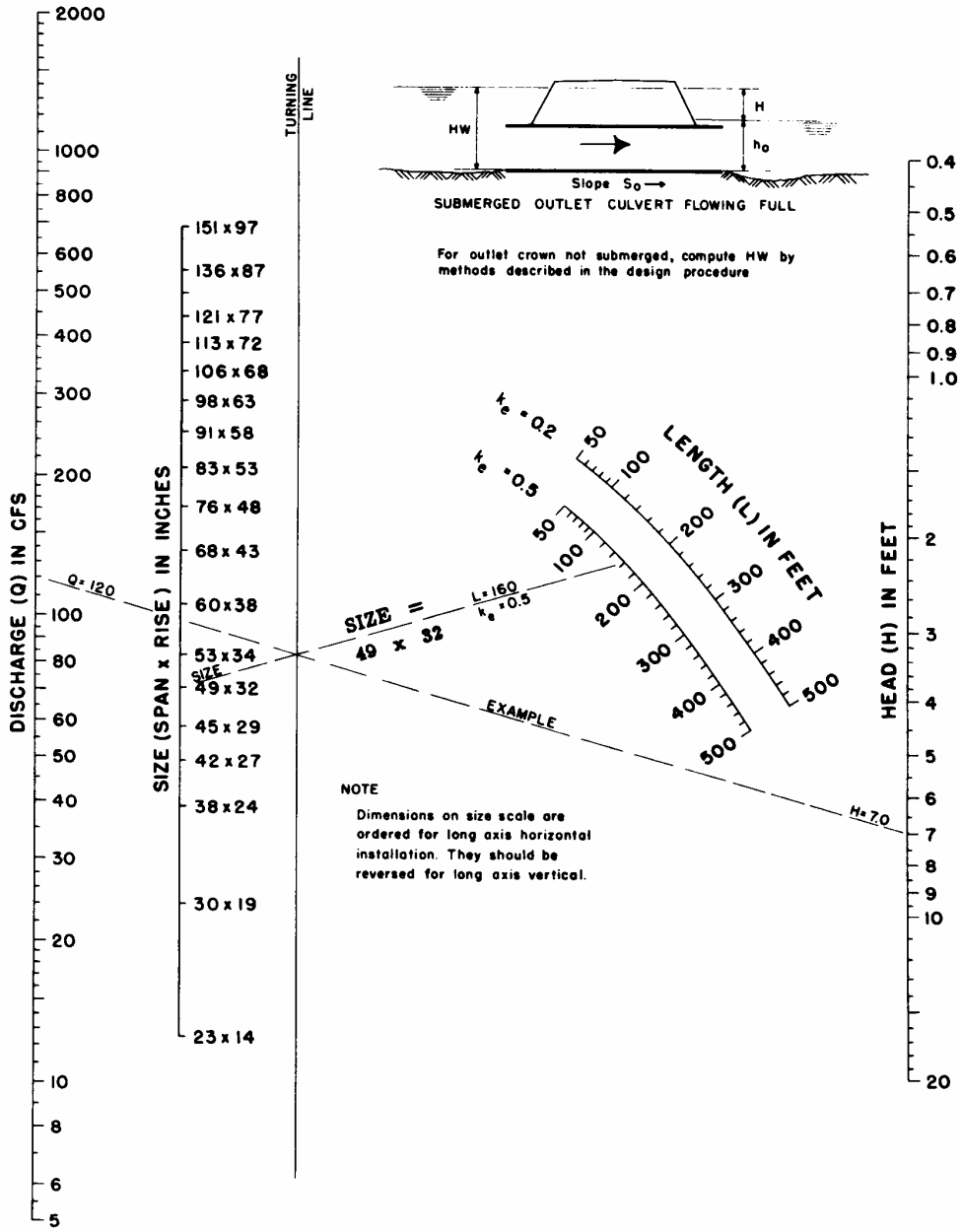
CHART 29B



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 33B



HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL

n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

Figure 6-77
Allowable Fill Heights for
Reinforced Circular Concrete Pipe

Pipe Diameter (Inches)	Allowable Fill height (Ft.)					
	Class III		Class IV		Class V	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
12	1.50	10	1.00	18	0.50	25
15	1.50	10	1.00	18	0.50	25
18	1.50	10	1.00	18	0.50	25
21	1.50	10	1.00	18	0.50	25
24	1.50	10	1.00	18	0.50	25
27	1.50	10	1.00	18	0.50	25
30	1.50	10	1.00	18	0.50	25
36	1.50	12	1.00	20	0.50	30
42	1.50	12	1.00	20	0.50	30
48	1.50	12	1.00	20	0.50	30
54	1.50	12	1.00	20	0.50	30
60	1.50	12	1.00	20	0.50	30
72	1.50	12	1.00	20	0.50	30

NOTES

1. Beddings: Class C for Class III pipes
   Class B for Class IV and Class V pipes

2. Minimum fill height represents the minimum cover from top of the pipe to the subgrade top elevation. When the cover is further restricted, ductile iron pipe with Class A bedding may be used with a minimum cover of 3 inches to top of subgrade.

3. Maximum fill height is measured from the finished grade.

Figure 6-78
Allowable Fill Heights for Round Corrugated Steel Pipe
Plain Galvanized

EQUIVALENT ROUND PIPE DIAMETER (IN.)	AREA (FT.^{2})	MINIMUM COVER (FT.)	MINIMUM METAL THICKNESS (IN.)	MINIMUM METAL THICKNESS (GAGE)	MAXIMUM FILL HEIGHTS (FT.)
2 2/3" x 1/2" CORRUGATIONS					
12	0.8	1.0	0.064	16	124
15	1.2				100
18	1.8				83
24	3.1				62
30	4.8		0.079	14	62
36	7.1				51
42	9.6				44
48	12.6		0.109	12	54
54	15.9				46
60	19.6		0.138	10	51
66	23.8				43
72	28.3		0.168	8	44
78	33.2				~~58~~ 37
84	38.5				~~44~~ 30
90	44.2				~~33~~ 25
96	50.3				~~27~~ 20
3" x 1" AND 5" x 1" CORRUGATIONS					
36	7.1	1.0	0.079	14	60
42	9.6				51
48	12.6				45
54	15.9				40
60	19.6				36
66	23.8				33
72	28.3				30
78	33.2		0.109	12	38
84	38.5				35
90	44.2				33
96	50.3				31

NOTE: THE GAGE SHOWN ABOVE ARE INDUSTRY STANDARD. ONLY THE GAGE SHOWN FOR EACH PIPE SIZE MAY BE USED.

Figure 6-79
Allowable Fill Heights for Corrugated Steel Pipe Arches
Plain Galvanized

EQUIVALENT ROUND PIPE DIAMETER (IN.)	SPAN AND RISE (IN.)	AREA (FT.^{2})	MINIMUM COVER (FT.)	MINIMUM METAL THICKNESS (IN.)	MINIMUM METAL THICKNESS (GAGE)	MAXIMUM FILL HEIGHTS (FT.)
2 2/3" x 1/2" CORRUGATIONS						
15	17 x 13	1.2	1.0	0.079	14	22
18	21 x 15	1.7				21
21	24 x 18	2.3				19
24	28 x 20	2.9				18
30	35 x 24	4.4				18
36	42 x 29	6.5				15
42	49 x 33	8.4		0.109	12	14
48	57 x 38	11.3		0.138	10	14
54	64 x 43	14.4				14
60	71 x 47	17.4		0.168	8	14
66	77 x 52	21.3				14
72	83 x 57	25.6				15
3" x 1" AND 5" x 1" CORRUGATIONS						
36	40 x 31	7.0	1.0	0.079	14	23
42	46 x 31	9.4				22
54	53 x 41	12.3		0.109	12	~~21~~
60	60 x 46	15.6				~~24~~ 31
66	66 x 51	19.3				28
72	73 x 55	23.2				26
78	81 x 59	27.4				24
84	87 x 63	32.1				22
90	95 x 67	37.0				20
96	103 x 71	42.4	1.5			18

NOTE: THE GAGE SHOWN ABOVE ARE INDUSTRY STANDARD. ONLY THE GAGE SHOWN FOR EACH PIPE SIZE MAY BE USED.

Figure 6-80
Allowable Fill Heights for Round Corrugated Aluminum Pipe

EQUIVALENT ROUND PIPE DIAMETER (IN.)	AREA (FT.^{2})	MINIMUM COVER (FT.)	MINIMUM METAL THICKNESS (IN.)	MINIMUM METAL THICKNESS (GAGE)	MAXIMUM FILL HEIGHTS (FT.)
2 2/3" x 1/2" CORRUGATIONS					
12	0.8	1.0	0.075	14	116
15	1.2				90
18	1.8				75
24	3.1				56
30	4.8		0.105	12	63
36	7.1				52
42	9.6		0.135	10	58
48	12.6				50
54	15.9	1.5			45
60	19.6		0.164	8	49
66	23.8				44
72	28.3				40
78	33.2	2.0			37
84	38.5				34
90	44.2				N.A.
96	50.3				N.A.
3" x 1" AND 5" x 1" CORRUGATIONS					
36	7.1	1.0	0.075	14	42
42	9.6				36
48	12.6				31
54	15.9	1.5	0.105	12	39
60	19.6				35
66	23.8				32
72	28.3		0.135	10	38
78	33.2				35
84	38.5				32
90	44.2	2.0			30
96	50.3		0.164	8	34

NOTE: THE GAGE SHOWN ABOVE ARE INDUSTRY STANDARD. ONLY THE GAGE SHOWN FOR EACH PIPE SIZE MAY BE USED.

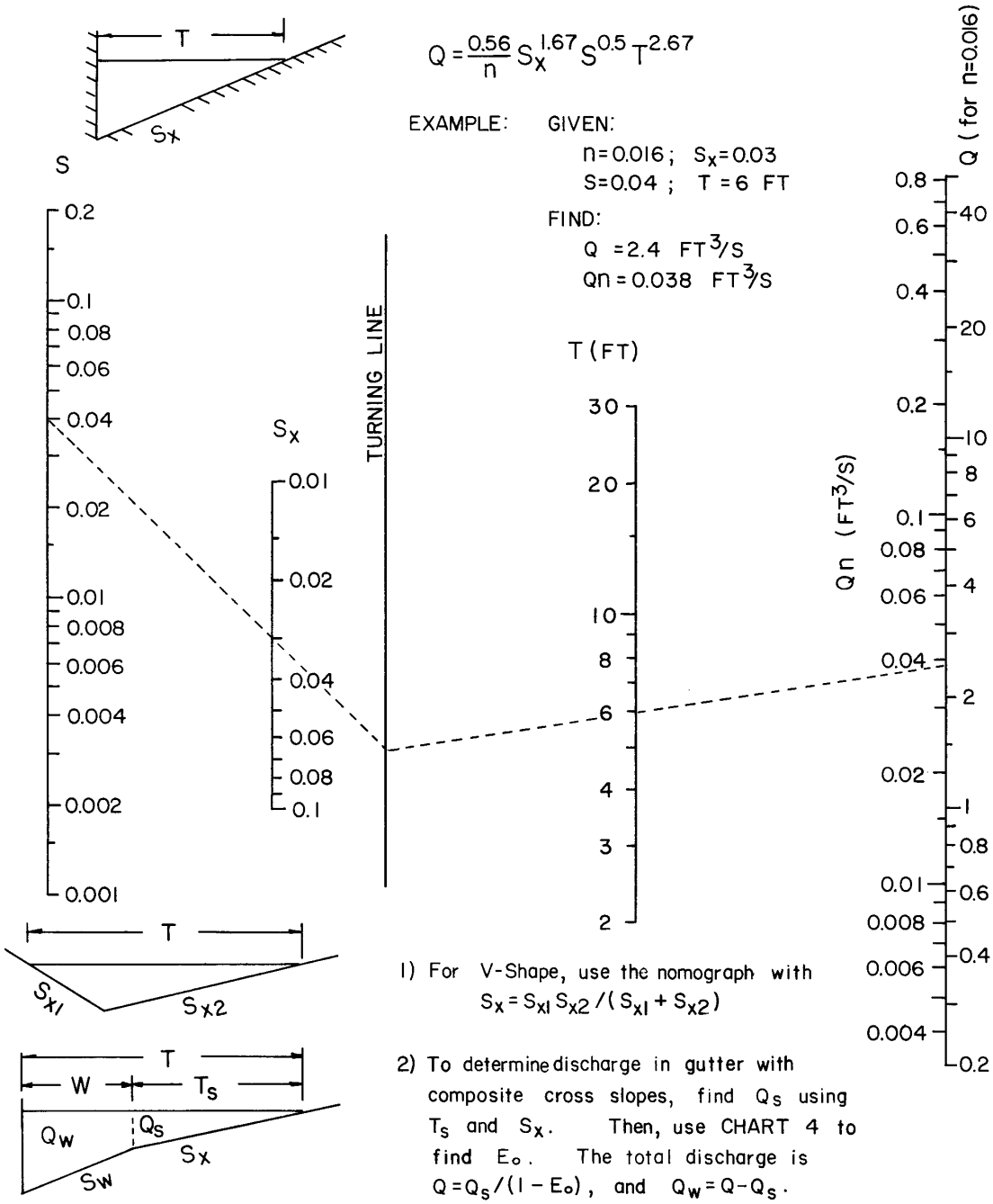
Figure 6-81
Allowable Fill Heights for Corrugated Aluminum Pipe Arches

EQUIVALENT ROUND PIPE DIAMETER (IN.)	SPAN AND RISE (IN.)	AREA (FT.^{2})	MINIMUM COVER (FT.)	MINIMUM METAL THICKNESS (IN.)	MINIMUM METAL THICKNESS (GAGE)	MAXIMUM FILL HEIGHTS (FT.)
2 2/3" x 1/2" CORRUGATIONS						
15	17 x 13	1.2	1.25	0.105	12	15
18	21 x 15	1.7				15
21	24 x 18	2.3				14
24	28 x 20	2.9		0.134	10	14
30	35 x 24	4.4	1.50			13
36	42 x 29	6.5		0.164	8	13
42	49 x 33	8.4				12
48	57 x 38	11.3				8
54	64 x 43	14.4	2.00			7
60	71 x 47	17.4				7
66	77 x 52	21.3				7
72	83 x 57	25.6				7
3" x 1" AND 5" x 1" CORRUGATIONS						
36	40 x 31	7.0	1.50	0.105	12	29
42	46 x 31	9.4				25
54	53 x 41	12.3		0.135	10	29
60	60 x 46	15.6	1.75			29
66	66 x 51	19.3				25
72	73 x 55	23.2				22
78	81 x 59	27.4	2.00			29
84	87 x 63	32.1				26
90	95 x 67	37.0				24
96	103 x 71	42.4	2.25			34

NOTE: THE GAGE SHOWN ABOVE ARE INDUSTRY STANDARD. ONLY THE GAGE SHOWN FOR EACH PIPE SIZE MAY BE USED.

Sequence ?

CHART 1B



$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

EXAMPLE: GIVEN:

$n = 0.016$; $S_x = 0.03$

$S = 0.04$; $T = 6$ FT

FIND:

$Q = 2.4$ FT^3/S

$Qn = 0.038$ FT^3/S

1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use CHART 4 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.

Flow in Triangular Gutter Sections - English Units

A - 3

Figure 6-96
Average Width of Water Spread in a Triangular Gutter Section

T_1/T_2	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
T_d/T_2	0.65	0.66	0.68	0.70	0.74	0.77	0.82	0.86	0.90

Figure 6-98
Curb, Gutter and Inlet Selection

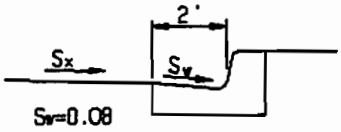
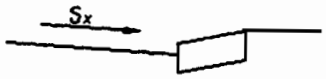
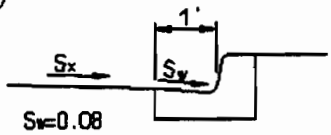
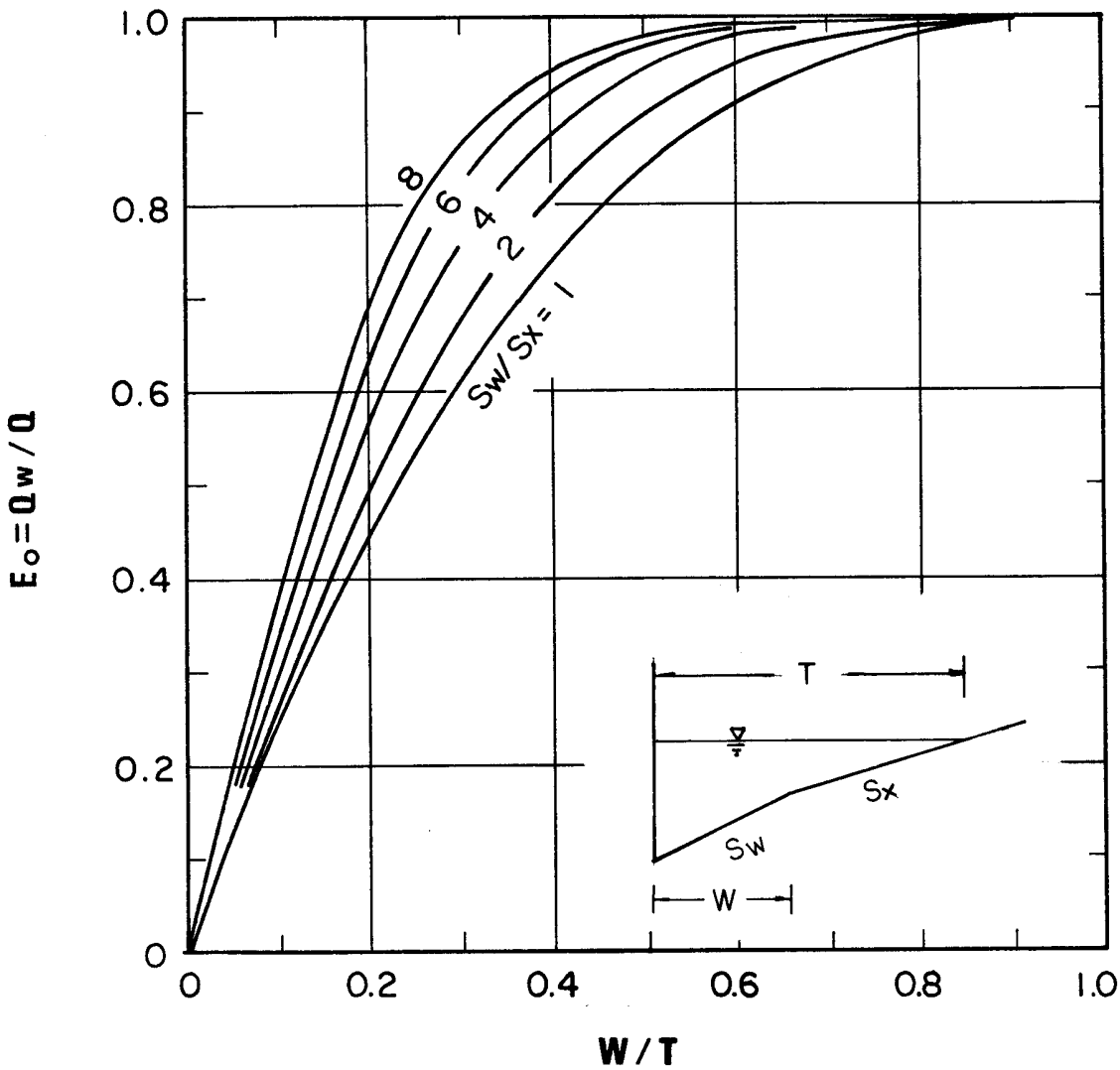
UNIFORM CROSS SLOPE		COMPOSITE CROSS SLOPE	
TYPICAL SECTION	1	TYPICAL SECTION	3 
CURB TYPE	P.C.C. CURB, TYPE 1, TYPE 2	CURB & GUTTER TYPE	INT. P.C.C. CURB & GUTTER, TYPE 1
INLET TYPE	CATCH BASIN A, A-A	INLET TYPE	CATCH BASIN J, J MODIFIED
TYPICAL SECTION	2 	TYPICAL SECTION	4 
CURB TYPE	P.C.C. PARKWAY CURB, TYPE 1, TYPE 2	CURB & GUTTER TYPE	INT. P.C.C. CURB & GUTTER, TYPE 3
INLET TYPE	CATCH BASIN B, P.W.B.D.	INLET TYPE	CURB OPENING INLET

CHART 2A



Ratio of frontal flow to total gutter flow.

A - 4

Figure 6-101
Flow in Composite Gutter Sections

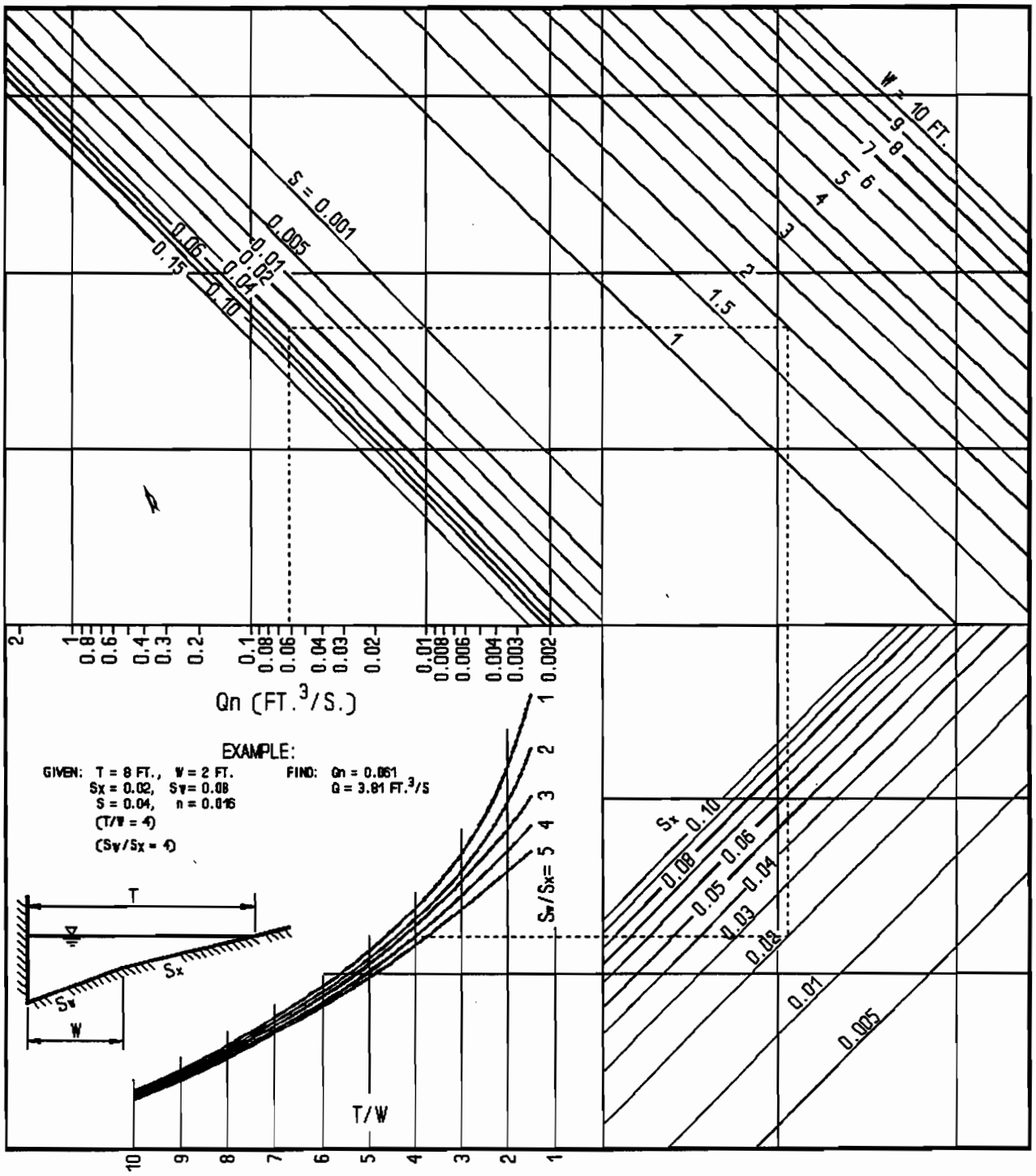


Figure 6-102
Gutter Flow Rate Q on Uniform Cross Slope (cfs)
n = 0.016 & S_x = 0.02

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.01	0.02	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.06	0.07	0.07	0.08	0.08	0.08
3	0.05	0.06	0.09	0.11	0.13	0.15	0.16	0.18	0.19	0.20	0.21	0.22	0.23	0.24	0.25
4	0.11	0.14	0.20	0.25	0.29	0.32	0.36	0.38	0.41	0.44	0.46	0.48	0.50	0.52	0.55
5	0.20	0.26	0.37	0.46	0.52	0.58	0.65	0.70	0.75	0.79	0.84	0.88	0.92	0.96	0.99
6	0.33	0.43	0.61	0.75	0.86	0.96	1.05	1.14	1.22	1.30	1.37	1.43	1.50	1.56	1.62
7	0.50	0.65	0.92	1.13	1.30	1.46	1.60	1.73	1.84	1.96	2.06	2.16	2.26	2.34	2.44
8	0.72	0.93	1.32	1.61	1.86	2.08	2.28	2.47	2.64	2.80	2.95	3.09	3.23	3.36	3.49
9	0.99	1.27	1.80	2.21	2.55	2.85	3.13	3.38	3.61	3.83	4.04	4.23	4.42	4.60	4.78
10	1.31	1.69	2.39	2.93	3.38	3.78	4.14	4.46	4.78	5.07	5.35	5.61	5.86	6.10	6.33

Figure 6-103
Gutter Flow Rate Q on Uniform Cross Slope (cfs)
n = 0.016 & S_x = 0.04

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.06	0.07	0.10	0.12	0.14	0.16	0.18	0.19	0.21	0.22	0.22	0.24	0.25	0.26	0.27
3	0.16	0.22	0.30	0.38	0.43	0.48	0.53	0.57	0.61	0.65	0.68	0.71	0.75	0.78	0.81
4	0.36	0.46	0.66	0.81	0.93	1.04	1.13	1.23	1.32	1.40	1.47	1.55	1.61	1.68	1.74
5	0.65	0.84	1.19	1.46	1.69	1.88	2.07	2.23	2.39	2.53	2.67	2.80	2.93	3.05	3.16
6	1.06	1.37	1.94	2.38	2.75	3.06	3.37	3.64	3.89	4.13	4.35	4.56	4.76	4.96	5.15
7	1.60	2.07	2.93	3.59	4.14	4.64	5.08	5.48	5.87	6.23	6.56	6.88	7.19	7.48	7.76
8	2.29	2.96	4.19	5.13	5.93	6.62	7.26	7.84	8.38	8.89	9.37	9.83	10.26	10.68	11.09
9	3.14	4.05	5.74	7.02	8.11	9.07	9.94	10.73	11.48	12.17	12.83	13.45	14.05	14.63	15.18
10	4.16	5.37	7.60	9.30	10.74	12.00	13.16	14.21	15.20	16.11	16.99	17.82	18.61	19.37	20.1

Figure 6-104
Gutter Flow Rate Q on Uniform Cross Slope (cfs)
n = 0.016 & S_x = 0.06

	Longitudinal Slopes x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.11	0.14	0.20	0.25	0.28	0.32	0.35	0.38	0.40	0.43	0.45	0.47	0.50	0.52	0.54
3	0.33	0.42	0.60	0.73	0.85	0.95	1.04	1.12	1.20	1.27	1.34	1.47	1.47	1.53	1.59
4	0.71	0.91	1.29	1.58	1.83	2.05	2.24	2.42	2.59	2.75	2.90	3.04	3.17	3.30	3.43
5	1.28	1.66	2.34	2.88	3.32	3.72	4.07	4.40	4.69	4.98	5.26	5.51	5.76	5.99	6.22
6	2.08	2.70	3.82	4.67	5.41	6.04	6.62	7.15	7.65	8.10	8.55	8.97	9.37	9.75	10.12
7	3.16	4.08	5.77	7.06	8.16	9.12	9.99	10.79	11.54	12.24	12.90	13.53	14.13	14.71	15.26
8	4.51	5.82	8.22	10.09	11.6	13.0	14.27	15.41	16.46	17.46	18.42	19.32	20.18	21.00	21.79
9	6.17	7.97	11.27	13.81	15.9	17.8	19.53	21.10	22.55	23.92	25.22	26.45	27.62	28.75	29.84
10	8.18	10.56	14.93	18.29	21.1	23.6	25.87	27.94	29.87	31.69	33.40	35.03	36.59	38.08	39.5

Figure 6-105
Gutter Flow Rate Q on Composite Section (cfs)
n = 0.016, S_x = 0.02, S_w = 0.08 & W = 2 Feet

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.18	0.23	0.33	0.40	0.46	0.52	0.57	0.61	0.66	0.70	0.73	0.77	0.80	0.84	0.87
3	0.24	0.32	0.45	0.55	0.64	0.72	0.78	0.85	0.91	0.96	1.01	1.05	1.11	1.16	1.20
4	0.34	0.44	0.62	0.76	0.88	0.98	1.07	1.16	1.24	1.32	1.39	1.46	1.52	1.58	1.64
5	0.46	0.58	0.84	1.03	1.19	1.33	1.46	1.57	1.68	1.79	1.88	1.97	2.06	2.15	2.23
6	0.61	0.79	1.12	1.38	1.59	1.78	1.95	2.10	2.25	2.39	2.52	2.64	2.76	2.87	2.98
7	0.81	1.04	1.48	1.82	2.09	2.34	2.57	2.78	2.97	3.15	3.32	3.48	3.64	3.79	3.93
8	1.04	1.36	1.92	2.35	2.72	3.04	3.33	3.60	3.85	4.08	4.31	4.52	4.71	4.91	5.10
9	1.06	1.73	2.45	3.01	3.47	3.88	4.25	4.59	4.91	5.21	5.49	5.76	6.02	6.26	6.50
10	1.69	2.18	3.08	3.77	4.36	4.78	5.34	5.77	6.17	6.54	6.90	7.23	7.55	7.86	8.16

Figure 6-106
Gutter Flow Rate Q on Composite Section (cfs)
n = 0.016, S_x = 0.04, S_w = 0.08 & W = 2 Feet

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.18	0.23	0.33	0.40	0.46	0.52	0.56	0.61	0.66	0.69	0.73	0.77	0.81	0.83	0.87
3	0.33	0.42	0.60	0.74	0.85	0.96	1.05	1.13	1.21	1.28	1.35	1.42	1.48	1.54	1.60
4	0.55	0.72	1.02	1.25	1.44	1.61	1.77	1.91	2.04	2.16	2.29	2.39	2.50	2.61	2.71
5	0.88	1.14	1.61	1.98	2.29	2.56	2.80	3.03	3.23	3.43	3.62	3.80	3.97	4.13	4.28
6	1.32	1.71	2.42	2.96	3.42	3.83	4.20	4.54	4.87	5.14	5.42	5.68	5.93	6.18	6.41
7	1.89	2.45	3.46	4.25	4.90	5.47	5.72	6.49	6.93	7.35	7.75	8.13	8.50	8.84	9.16
8	2.61	3.38	4.77	5.84	6.75	7.55	8.26	8.93	9.54	10.13	10.67	11.19	11.69	12.17	12.63
9	3.49	4.50	6.36	7.79	9.00	10.06	11.02	11.91	12.73	13.5	14.23	14.93	15.59	16.23	16.85
10	4.53	5.85	8.27	10.13	11.70	13.08	14.33	15.48	16.55	17.55	18.50	19.41	20.27	21.10	21.89

Figure 6-107

Gutter Flow Rate Q on Composite Section (cfs)

n = 0.016, S_x = 0.06, S_w = 0.08 & W = 2 Feet

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.18	0.23	0.33	0.40	0.46	0.52	0.57	0.61	0.66	0.70	0.73	0.77	0.80	0.84	0.87
3	0.42	0.55	0.77	0.95	1.10	1.23	1.35	1.45	1.55	1.65	1.74	1.82	1.90	1.98	2.06
4	0.83	1.07	1.51	1.86	2.14	2.40	2.63	2.84	3.03	3.22	3.39	3.56	3.72	3.87	4.01
5	1.43	1.84	2.61	3.20	3.69	4.13	4.52	4.89	5.22	5.54	5.84	6.13	6.40	6.66	6.91
6	2.25	2.91	4.12	5.05	5.83	6.52	7.14	7.71	8.24	8.74	9.22	9.67	10.10	10.51	10.91
7	3.34	4.31	6.10	7.47	8.63	9.64	10.57	11.4	12.2	12.95	13.65	14.31	14.95	15.56	16.15
8	4.71	6.08	8.60	10.54	12.17	13.60	14.90	16.1	17.2	18.25	19.24	20.18	21.08	21.94	22.77
9	6.39	8.25	11.67	14.30	16.51	18.45	20.21	21.8	23.3	24.77	26.11	27.38	28.60	29.77	30.84
10	8.41	10.86	15.36	18.82	21.73	24.29	26.61	28.5	30.7	32.59	34.36	36.04	37.64	39.18	40.66

Figure 6-108
Average Gutter Flow Velocity V ~~(cfs)~~ (fps)
Uniform Cross Slope, S_x = 0.02, n = 0.016

	Longitudinal Slope, S x 100 (100%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.01	0.02	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.06	0.07	0.07	0.08	0.08	0.08
3	0.04	0.05	0.08	0.10	0.12	0.14	0.15	0.17	0.18	0.19	0.20	0.21	0.22	0.22	0.23
4	0.10	0.13	0.18	0.22	0.26	0.28	0.31	0.33	0.36	0.38	0.40	0.41	0.43	0.45	0.47
5	0.18	0.23	0.31	0.38	0.43	0.47	0.52	0.55	0.59	0.62	0.65	0.68	0.71	0.74	0.76
6	0.29	0.36	0.48	0.57	0.64	0.70	0.76	0.81	0.86	0.92	0.96	1.00	1.04	1.08	1.12
7	0.42	0.51	0.67	0.79	0.88	0.97	1.05	1.12	1.18	1.25	1.31	1.36	1.42	1.47	1.52
8	0.57	0.69	0.89	1.04	1.16	1.27	1.37	1.46	1.55	1.63	1.71	1.78	1.85	1.91	1.98
9	0.75	0.89	1.13	1.32	1.47	1.60	1.73	1.84	1.94	2.05	2.14	2.23	2.32	2.40	2.48
10	0.95	1.12	1.41	1.62	1.80	1.96	2.11	2.25	2.38	2.50	2.61	2.72	2.83	2.93	2.98

Figure 6-109

Average Gutter Flow Velocity V ~~(cfs)~~ (fps)

Uniform Cross Slope, S_x = 0.04, n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.70	0.92	1.29	1.58	1.84	2.05	2.25	2.43	2.60	2.76	2.90	3.05	3.18	3.31	3.43
3	0.93	1.20	1.70	2.08	2.40	2.69	2.94	3.19	3.40	3.61	3.80	3.99	4.16	4.34	4.50
4	1.13	1.45	2.06	2.52	2.92	3.25	3.57	3.85	4.12	4.37	4.61	4.84	5.05	5.26	5.46
5	1.30	1.69	2.39	2.93	3.38	3.79	4.14	4.47	4.78	5.07	5.34	5.60	5.86	6.10	6.33
6	1.47	1.90	2.70	3.31	3.82	4.27	4.68	5.03	5.40	5.73	6.04	6.34	6.62	6.89	7.14
7	1.64	2.11	2.99	3.66	4.23	4.73	5.18	5.60	5.99	6.35	6.69	7.02	7.33	7.63	7.92
8	1.79	2.32	3.27	4.00	4.63	5.18	5.67	6.12	6.54	6.95	7.32	7.68	8.02	8.34	8.66
9	1.93	2.50	3.53	4.33	5.01	5.43	6.13	6.62	7.08	7.51	7.91	8.31	8.67	9.03	9.37
10	2.08	2.68	3.80	4.65	5.37	6.00	6.58	7.11	7.59	8.05	8.49	8.91	9.30	9.68	10.05

(fps)

Figure 6-110
Average Gutter Flow Velocity V ~~(cfs)~~
Uniform Cross Slope, S_x = 0.06, n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.93	1.20	1.70	2.08	2.40	2.69	2.94	3.18	3.40	3.61	3.80	3.99	4.17	4.34	4.50
3	1.22	1.57	2.23	2.73	3.15	3.52	3.86	4.17	4.46	4.73	4.98	5.23	5.46	5.68	5.90
4	1.48	1.91	2.70	3.31	3.82	4.27	4.67	5.05	5.40	5.73	6.04	6.33	6.62	6.89	7.15
5	1.71	2.21	3.13	3.84	4.42	4.96	5.43	5.86	6.27	6.65	7.01	7.35	7.68	7.99	8.29
6	1.94	2.50	3.54	4.33	5.00	5.60	6.13	6.62	7.08	7.51	7.92	8.30	8.67	9.03	9.37
7	2.15	2.77	3.92	4.80	5.55	6.20	6.79	7.34	7.85	8.32	8.77	9.20	9.60	10.00	10.38
8	2.34	3.03	4.29	5.25	6.06	6.78	7.43	8.02	8.58	9.10	9.59	10.06	10.51	10.94	11.35
9	2.54	3.28	4.64	5.68	6.56	7.33	8.03	8.68	9.28	9.84	10.37	10.88	11.37	11.83	12.28
10	2.72	3.52	4.96	6.09	7.04	7.87	8.62	9.31	9.95	10.56	11.13	11.67	12.19	12.69	13.17

Figure 6-111

Average Gutter Flow Velocity V ~~(cfs)~~ (fps)

Composite Section, S_x = 0.02, n = 0.016, s_w = 0.08 & W = 2 Feet

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	1.12	1.45	2.06	2.52	2.91	3.26	3.57	3.85	4.12	4.37	4.61	4.83	5.05	5.25	5.45
3	1.17	1.53	2.17	2.65	3.06	3.43	3.75	4.06	4.34	4.60	4.85	5.08	5.31	5.53	5.74
4	1.21	1.57	2.22	2.72	3.14	3.51	3.85	4.16	4.44	4.71	4.96	5.21	5.45	5.67	5.88
5	1.24	1.61	2.28	2.79	3.22	3.60	3.95	4.26	4.56	4.83	5.10	5.35	5.58	5.81	6.03
6	1.28	1.66	2.34	2.88	3.32	3.71	4.07	4.40	4.69	4.98	5.26	58.51	5.76	5.99	6.22
7	1.33	1.72	2.43	2.98	3.44	3.85	4.21	4.56	4.87	5.17	5.45	5.71	5.97	6.21	6.45
8	1.38	1.79	2.53	3.10	3.58	4.01	4.39	4.73	5.07	5.38	5.67	5.94	6.21	6.46	6.71
9	1.44	1.86	2.64	3.23	3.73	4.17	5.47	4.94	5.28	5.60	5.91	6.19	6.47	6.73	6.99
10	1.50	1.94	2.75	3.37	3.89	4.35	4.77	5.15	5.51	5.84	6.16	6.46	6.74	7.02	7.29

(fps)

Figure 6-112

Average Gutter Flow Velocity V ~~(cfs)~~

Composite Section, S_x = 0.04, n = 0.016, s_w = 0.08 & W = 2 Feet

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	1.12	1.45	2.06	2.52	2.92	3.25	3.57	3.85	4.13	4.37	4.61	4.84	5.05	5.26	5.46
3	1.28	1.64	2.33	2.85	3.29	3.68	4.04	4.36	4.66	4.94	5.21	5.47	5.71	5.94	6.17
4	1.40	1.81	2.56	3.13	3.62	4.05	4.43	4.79	5.12	5.43	5.73	6.01	6.27	6.53	6.78
5	1.53	1.97	2.79	3.42	3.95	4.42	4.84	5.22	5.59	5.93	6.24	6.55	6.84	7.13	7.39
6	1.66	2.14	3.03	3.72	4.28	4.79	5.25	5.67	6.06	6.43	6.78	7.11	7.43	7.13	8.02
7	1.79	2.31	3.26	4.00	4.62	5.17	5.66	6.11	6.53	6.94	7.31	7.67	8.01	8.33	8.65
8	1.92	2.48	3.51	4.29	4.96	5.54	6.08	6.56	7.01	7.44	7.85	8.23	8.60	8.94	9.28
9	2.04	2.64	3.74	4.58	5.30	5.92	6.49	7.00	7.49	7.94	8.37	8.78	9.17	9.54	9.91
10	3.11	2.83	3.98	4.87	5.63	6.29	6.89	7.43	7.95	8.44	8.89	9.33	9.74	10.13	10.52

Figure 6-113
Average Gutter Flow Velocity V ~~(cfs)~~ (fps)
Composite Section, S_x = 0.06, n = 0.016, s_w = 0.08 & W = 2 Feet

	Longitudinal Slope, S x 100 (%)														
Spread T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	1.12	1.45	2.06	2.52	2.91	3.26	3.57	3.85	4.12	4.37	4.61	4.83	5.06	5.25	5.45
3	1.37	1.77	2.51	3.08	3.55	3.97	4.35	4.69	5.03	5.33	5.62	5.89	6.16	6.41	6.65
4	1.60	2.06	2.92	3.57	4.13	4.61	5.05	5.46	5.84	6.19	6.53	6.85	7.15	7.44	7.72
5	1.81	2.33	3.30	4.05	4.67	5.23	5.73	6.19	6.61	7.02	7.39	7.76	8.10	8.43	8.75
6	2.01	2.60	3.68	4.51	5.20	5.82	6.37	6.88	7.36	7.81	8.22	8.63	9.02	9.38	9.74
7	2.21	2.85	4.04	4.94	5.71	6.39	7.00	7.56	8.08	8.57	9.04	9.47	9.89	10.30	10.69
8	2.40	3.10	4.39	5.37	6.20	6.94	7.60	8.21	8.78	9.31	9.81	10.29	10.75	11.19	11.61
9	2.58	3.34	4.71	5.79	6.68	7.47	8.18	8.84	9.45	10.02	10.57	11.08	11.58	12.05	12.50
10	2.76	3.57	5.05	6.19	7.14	7.99	8.75	9.45	10.10	10.72	11.30	11.85	12.38	12.88	13.37

Figure 6-114
Frontal Flow Interception Factor, R_f

V (fps)	6.8 or Less	7	8	9	10	11	12	13	14	15
R_f	1.00	0.92	0.84	0.74	0.65	0.56	0.47	0.39	0.30	0.20

Figure 6-115
Side Flow Interception Factor, R_s

V (fps)	S_x (Feet/foot)				
	0.02	0.03	0.04	0.05	0.06
1	0.63	0.71	0.77	0.81	0.83
2	0.32	0.42	0.49	0.55	0.59
3	0.19	0.26	0.32	0.37	0.41
4	0.12	0.17	0.22	0.26	0.29
5	0.08	0.12	0.16	0.19	0.22
6	0.06	0.09	0.12	0.14	0.17
7	0.05	0.07	0.09	0.11	0.13
8	0.04	0.06	0.07	0.09	0.11
9	0.03	0.05	0.06	0.07	0.09
10	0.03	0.04	0.05	0.06	0.07
11	0.02	0.03	0.04	0.05	0.06
12	0.02	0.03	0.04	0.05	0.05
13	0.02	0.02	0.03	0.04	0.05
14	0.01	0.02	0.03	0.03	0.04
15	0.01	0.02	0.02	0.03	0.04

Figure 6-116
Gutter Flow Intercepted by Standard Grate, Q_i (cfs)
Uniform Cross Slope S_x = 0.02 & n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.01	0.02	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.06	0.07	0.07	0.08	0.08	0.08
3	0.04	0.05	0.08	0.10	0.12	0.14	0.15	0.17	0.18	0.19	0.20	0.21	0.22	0.22	0.23
4	0.10	0.13	0.18	0.22	0.26	0.28	0.31	0.33	0.36	0.38	0.40	0.41	0.43	0.45	0.47
5	0.18	0.23	0.31	0.38	0.43	0.47	0.52	0.55	0.59	0.62	0.65	0.68	0.71	0.74	0.76
6	0.29	0.36	0.48	0.57	0.64	0.70	0.76	0.81	0.86	0.92	0.96	1.00	1.04	1.08	1.12
7	0.42	0.51	0.67	0.79	0.88	0.97	1.05	1.12	1.18	1.25	1.31	1.36	1.42	1.47	1.52
8	0.57	0.69	0.89	1.04	1.16	1.27	1.37	1.46	1.55	1.63	1.71	1.78	1.85	1.91	1.98
9	0.75	0.89	1.13	1.32	1.47	1.60	1.73	1.84	1.94	2.05	2.14	2.23	2.32	2.40	2.48
10	0.95	1.12	1.41	1.62	1.80	1.96	2.11	2.25	2.38	2.50	2.61	2.72	2.83	2.93	2.98

Figure 6-117
Gutter Flow Intercepted by Standard Grate, Q_i (cfs)
Uniform Cross Slope S_x = 0.04 & n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.06	0.07	0.11	0.13	0.15	0.17	0.19	0.20	0.22	0.23	0.24	0.26	0.27	0.28	0.29
3	0.15	0.21	0.29	0.37	0.43	0.47	0.52	0.57	0.61	0.64	0.68	0.71	0.75	0.78	0.81
4	0.33	0.43	0.61	0.75	0.85	0.96	1.04	1.12	1.20	1.27	1.34	1.41	1.47	1.53	1.58
5	0.57	0.73	1.02	1.24	1.41	1.57	1.72	1.84	1.97	2.08	2.19	2.29	2.36	2.40	2.43
6	0.87	1.11	1.53	1.84	2.09	2.32	2.52	2.71	2.88	3.04	3.11	3.16	3.21	3.24	3.26
7	1.24	1.57	2.13	2.54	2.87	3.17	3.44	3.69	3.84	3.92	3.98	4.02	4.05	4.07	4.07
8	1.66	2.08	2.80	3.32	3.76	4.14	4.49	4.65	4.75	4.81	4.85	4.87	4.88	4.86	4.84
9	2.16	2.67	3.56	4.20	4.74	5.22	5.46	5.58	5.66	5.70	5.71	5.69	5.66	5.60	5.54
10	2.72	3.34	4.39	5.16	5.82	6.26	6.41	6.51	6.56	6.56	6.52	6.46	6.37	6.26	6.17

Figure 6-118

Gutter Flow Intercepted by Standard Grate, Q_i (cfs)

Uniform Cross Slope S_x = 0.06 & n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.11	0.14	0.20	0.25	0.28	0.32	0.35	0.38	0.40	0.43	0.45	0.47	0.50	0.52	0.54
3	0.32	0.41	0.58	0.70	0.82	0.91	1.00	1.07	1.15	1.21	1.28	1.34	1.40	1.46	1.51
4	0.67	0.85	1.17	1.42	1.63	1.81	1.97	2.11	2.26	2.39	2.51	2.58	2.62	2.66	2.69
5	1.16	1.46	1.98	2.36	2.68	2.97	3.22	3.46	3.63	3.71	3.77	3.82	3.85	3.87	3.88
6	1.80	2.23	2.96	3.50	3.96	4.36	4.73	4.86	4.92	5.01	5.05	5.07	5.07	5.05	5.02
7	2.59	3.15	4.12	4.83	5.45	5.95	6.12	6.23	6.29	6.32	6.31	6.27	6.20	6.12	6.01
8	3.49	4.20	5.42	6.34	7.11	7.39	7.54	7.60	7.60	7.55	7.47	7.34	7.18	7.00	6.79
9	4.53	5.40	6.88	8.01	8.67	8.88	8.96	8.94	8.85	8.71	8.51	8.26	7.97	7.66	7.31
10	5.70	6.72	8.48	9.83	10.21	10.35	10.34	10.22	10.01	9.72	9.37	8.98	8.53	8.03	7.51

Figure 6-119

Gutter Flow Intercepted by Standard Grate, Q_i (cfs)

Composite Section S_x = 0.02, S_w = 0.08, W = 2 Feet & n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.18	0.23	0.33	0.40	0.46	0.52	0.57	0.61	0.66	0.70	0.73	0.77	0.80	0.84	0.87
3	0.23	0.31	0.44	0.54	0.63	0.71	0.77	0.84	0.90	1.94	1.01	1.04	1.09	1.14	1.18
4	0.33	0.42	0.59	0.72	0.84	0.93	1.02	1.10	1.18	1.25	1.32	1.38	1.44	1.50	1.55
5	0.43	0.55	0.76	0.93	1.07	1.19	1.31	1.40	1.50	1.60	1.68	1.76	1.83	1.91	1.98
6	0.55	0.70	0.96	1.17	1.34	1.49	1.63	1.76	1.87	1.98	2.52	2.18	2.28	2.37	2.44
7	0.70	0.88	1.19	1.44	1.64	1.82	1.98	2.14	2.28	2.41	2.53	2.65	2.77	2.86	2.90
8	0.87	1.08	1.45	1.74	1.97	2.18	2.37	2.55	2.72	2.87	3.02	3.16	3.28	3.33	3.37
9	1.06	1.30	1.73	2.06	2.33	2.58	2.80	3.00	3.20	3.38	3.55	3.70	3.76	3.81	3.85
10	1.28	1.56	2.04	2.41	2.72	3.00	3.26	3.49	3.71	3.92	4.11	4.18	4.25	4.30	4.34

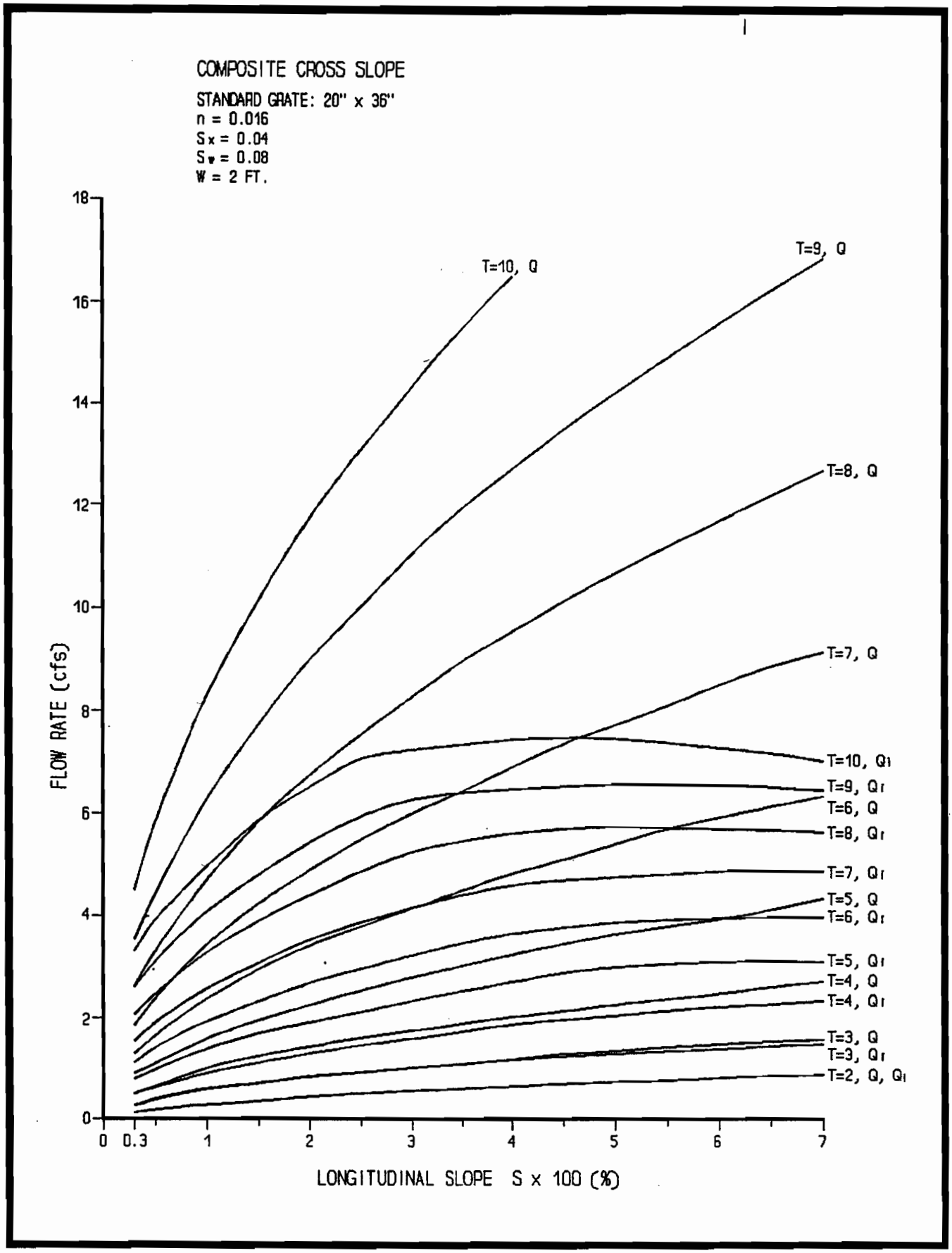
Figure 6-120
Gutter Flow Intercepted by Standard Grate, Q_i (cfs)
Composite Section S_x = 0.04, S_w = 0.08, W = 2 Feet & n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.18	0.23	0.33	0.40	0.46	0.52	0.57	0.61	0.66	0.70	0.73	0.77	0.80	0.84	0.87
3	0.32	0.41	0.59	0.72	0.83	0.93	1.02	1.10	1.18	1.25	1.31	1.38	1.44	1.50	1.55
4	0.53	0.68	0.95	1.16	1.32	1.48	1.62	1.74	1.87	1.97	2.08	2.18	2.25	2.28	2.32
5	0.81	1.03	1.41	1.70	1.95	2.16	2.35	2.53	2.70	2.85	2.97	3.03	3.08	3.12	3.14
6	1.16	1.45	1.96	2.35	2.67	2.95	3.21	3.45	3.67	3.78	3.85	3.90	3.95	3.98	3.99
7	1.58	1.96	2.60	3.08	3.50	3.86	4.19	4.49	4.61	4.69	4.76	4.80	4.83	4.84	4.84
8	2.08	2.53	3.32	3.92	4.42	4.87	5.27	5.47	5.56	5.63	5.68	5.69	5.70	5.68	5.65
9	2.64	3.18	4.11	4.82	5.43	5.96	6.27	6.41	6.49	6.54	6.56	6.55	6.52	6.46	6.38
10	3.27	3.89	4.99	5.82	6.53	7.08	7.26	7.38	7.45	7.46	7.43	7.37	7.29	7.18	7.05

Figure 6-121
Gutter Flow Intercepted by Standard Grate, Q_i (cfs)
Composite Section S_x = 0.06, S_w = 0.08, W = 2 Feet & n = 0.016

	Longitudinal Slope, S x 100 (%)														
Spread, T (Feet)	**0.30**	**0.50**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**
2	0.18	0.23	0.33	0.40	0.46	0.52	0.57	0.61	0.66	0.70	0.73	0.77	0.80	0.84	0.87
3	0.41	0.54	0.75	0.92	1.06	1.19	1.30	1.40	1.49	1.65	1.68	1.82	1.82	1.86	1.89
4	0.79	1.00	1.39	1.69	1.93	2.15	2.35	2.52	2.69	2.84	2.89	2.94	2.98	3.01	3.03
5	1.30	1.63	2.22	2.67	3.03	3.36	3.66	3.92	4.01	4.09	4.14	4.18	4.20	4.21	4.21
6	1.95	2.42	3.24	3.85	4.36	4.82	5.13	5.25	5.33	5.39	5.42	5.42	5.40	5.37	5.31
7	2.75	3.36	4.42	5.21	5.89	6.35	6.52	6.62	6.67	6.68	6.65	6.58	6.50	6.39	6.26
8	3.68	4.44	5.76	6.76	7.57	7.80	7.94	8.00	7.89	7.91	7.80	7.64	7.45	7.23	6.99
9	4.73	5.65	7.24	8.46	9.08	9.28	9.36	9.32	9.21	9.04	8.79	8.51	8.18	7.82	7.44
10	5.90	6.98	8.87	10.27	10.63	10.76	10.73	10.59	10.33	10.01	9.62	9.18	8.67	8.14	7.55

Figure 6-122
Interception Capacity of Standard Grate
On Continuous Grade



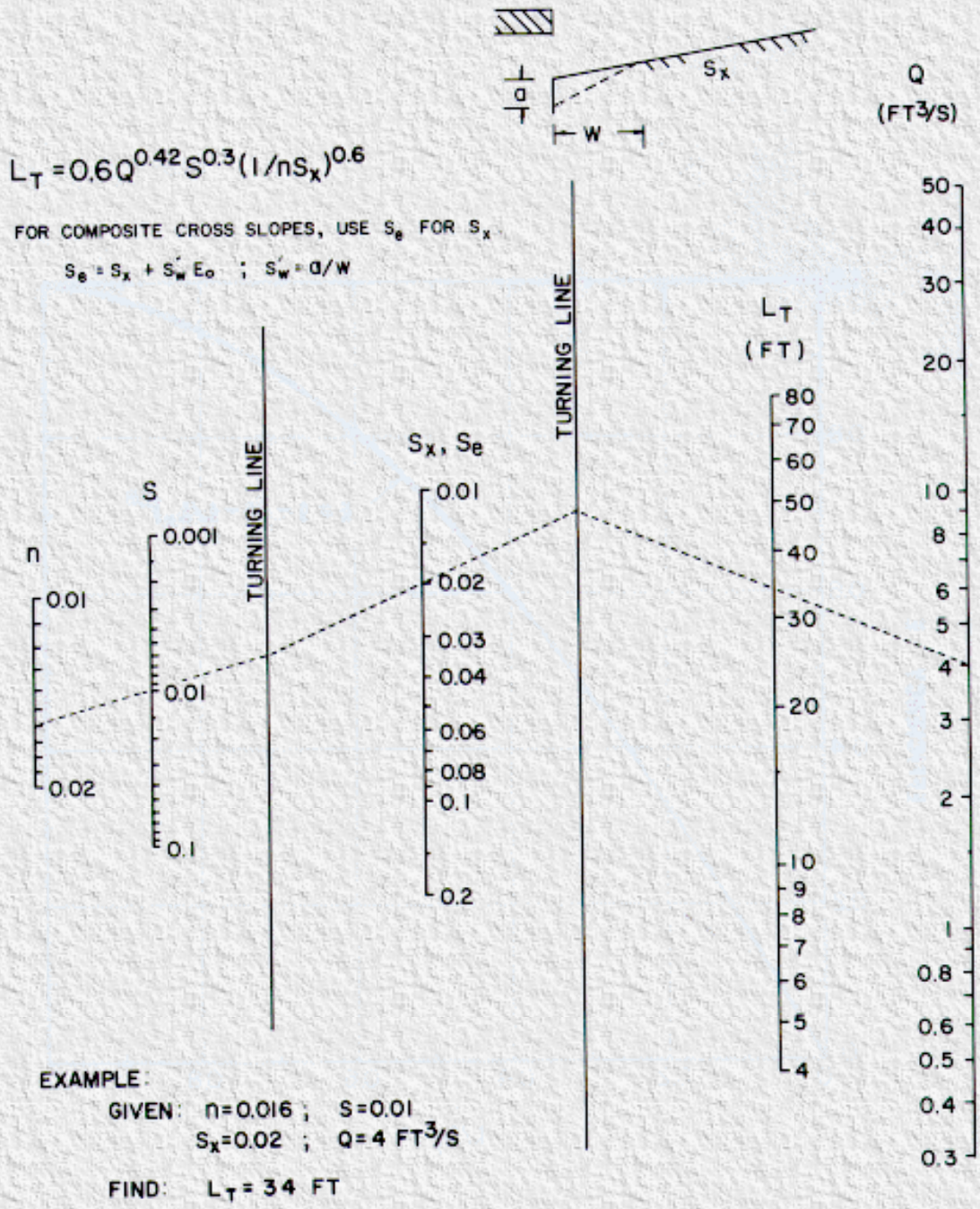
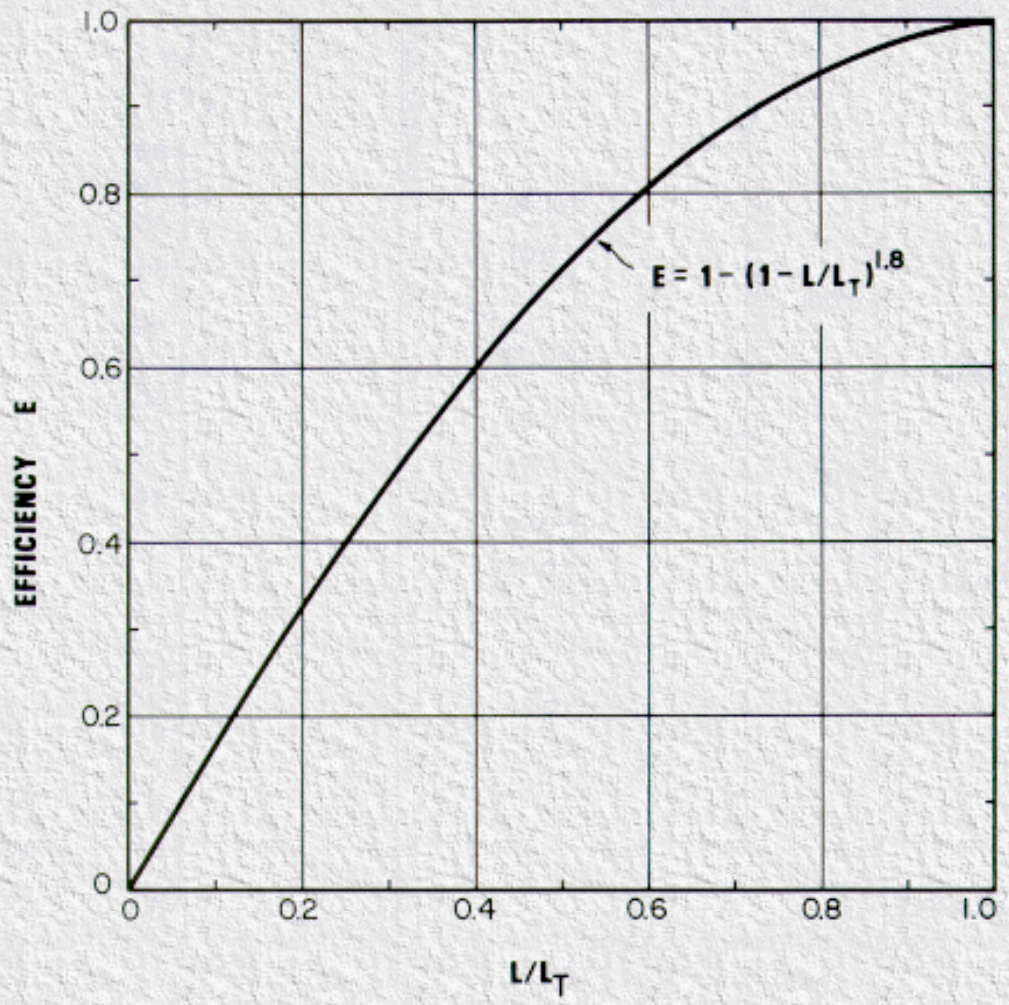
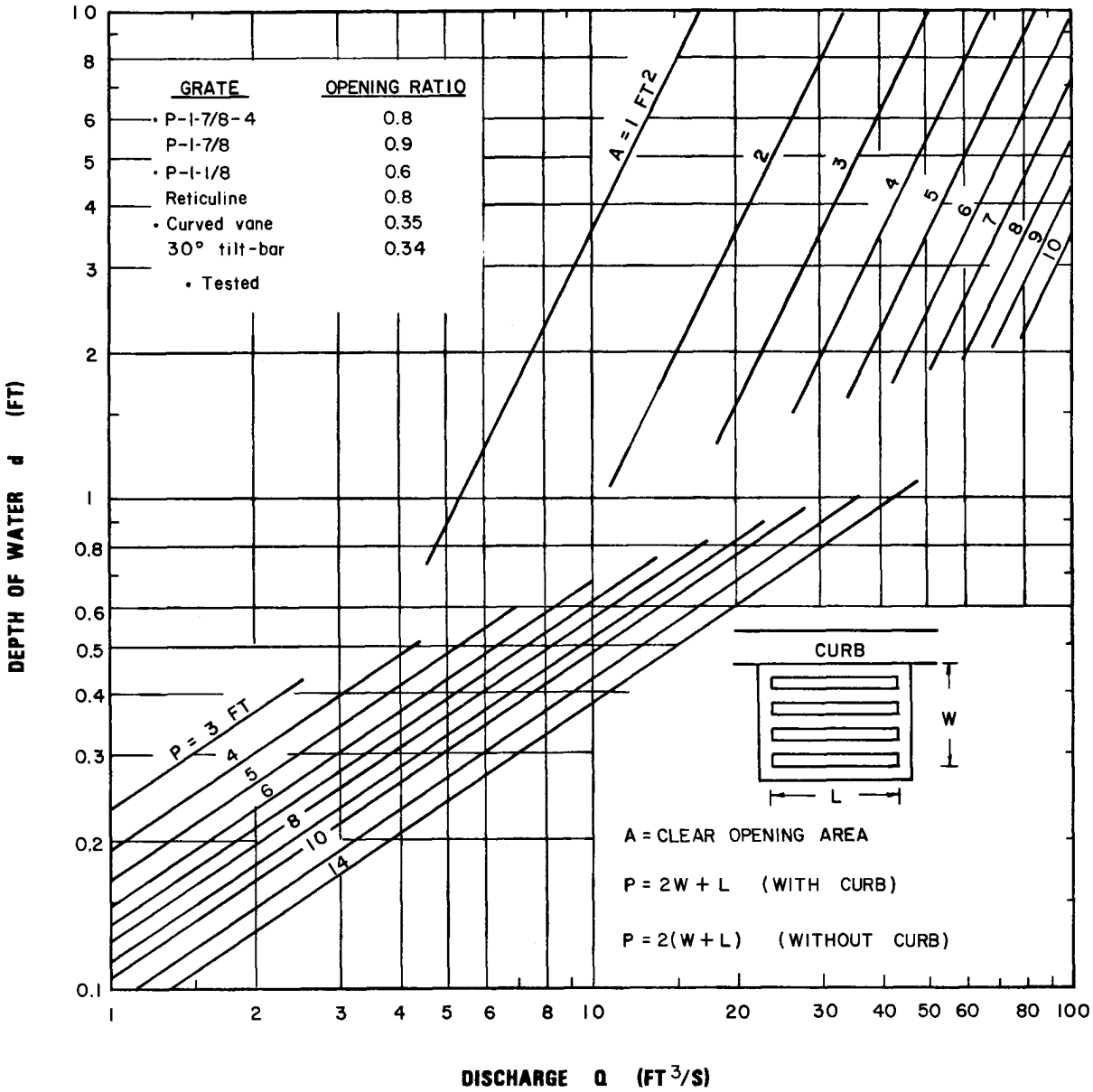


Chart 9. Curb-opening and slotted drain inlet length for total interception.



Curb-Opening and Slotted Drain Inlet Interception Efficiency

CHART 9B



Grate Inlet Capacity in Sump Conditions - English Units

A - 19

Figure 6-126
Depressed Curb-Opening Inlet Capacity in Sump Conditions

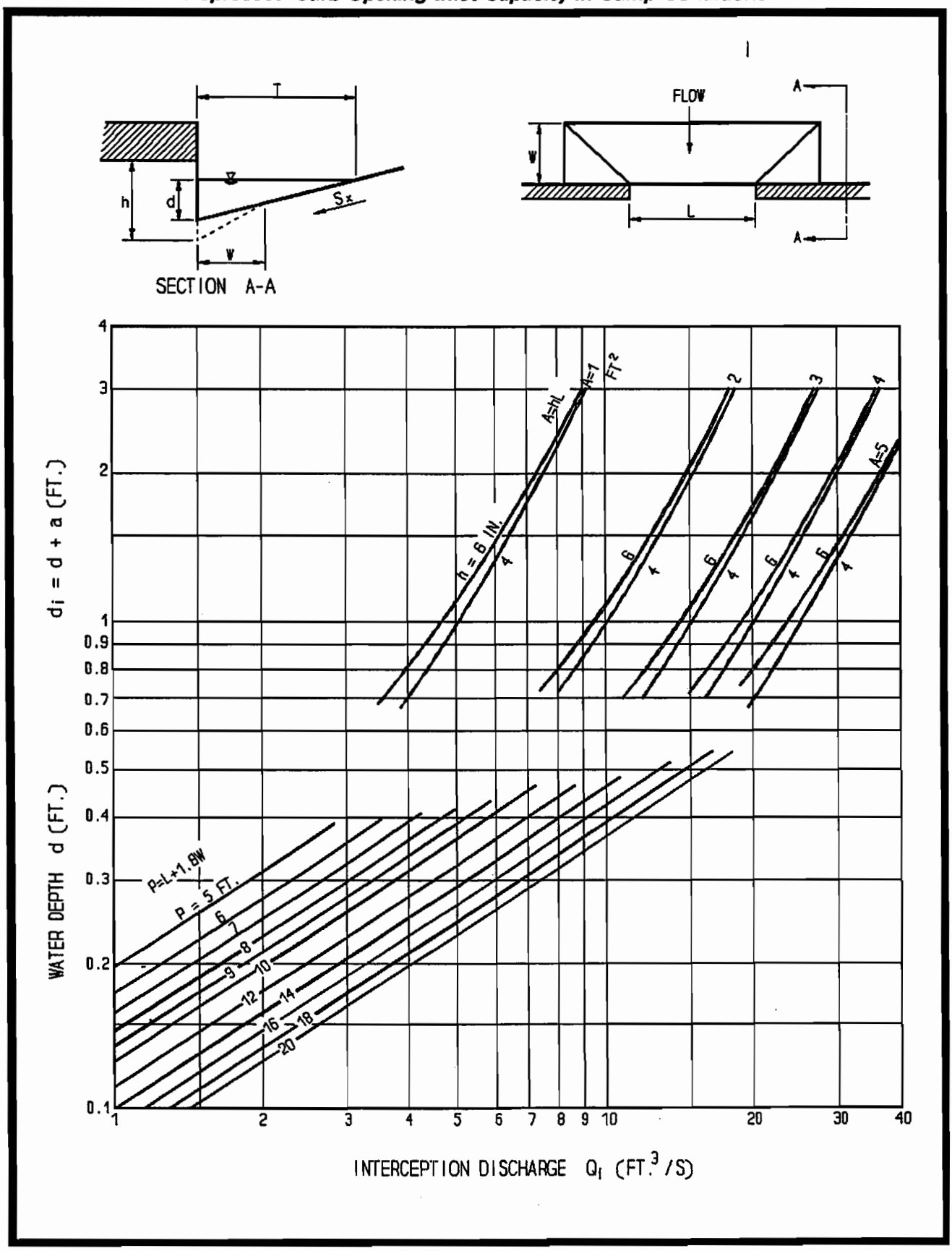
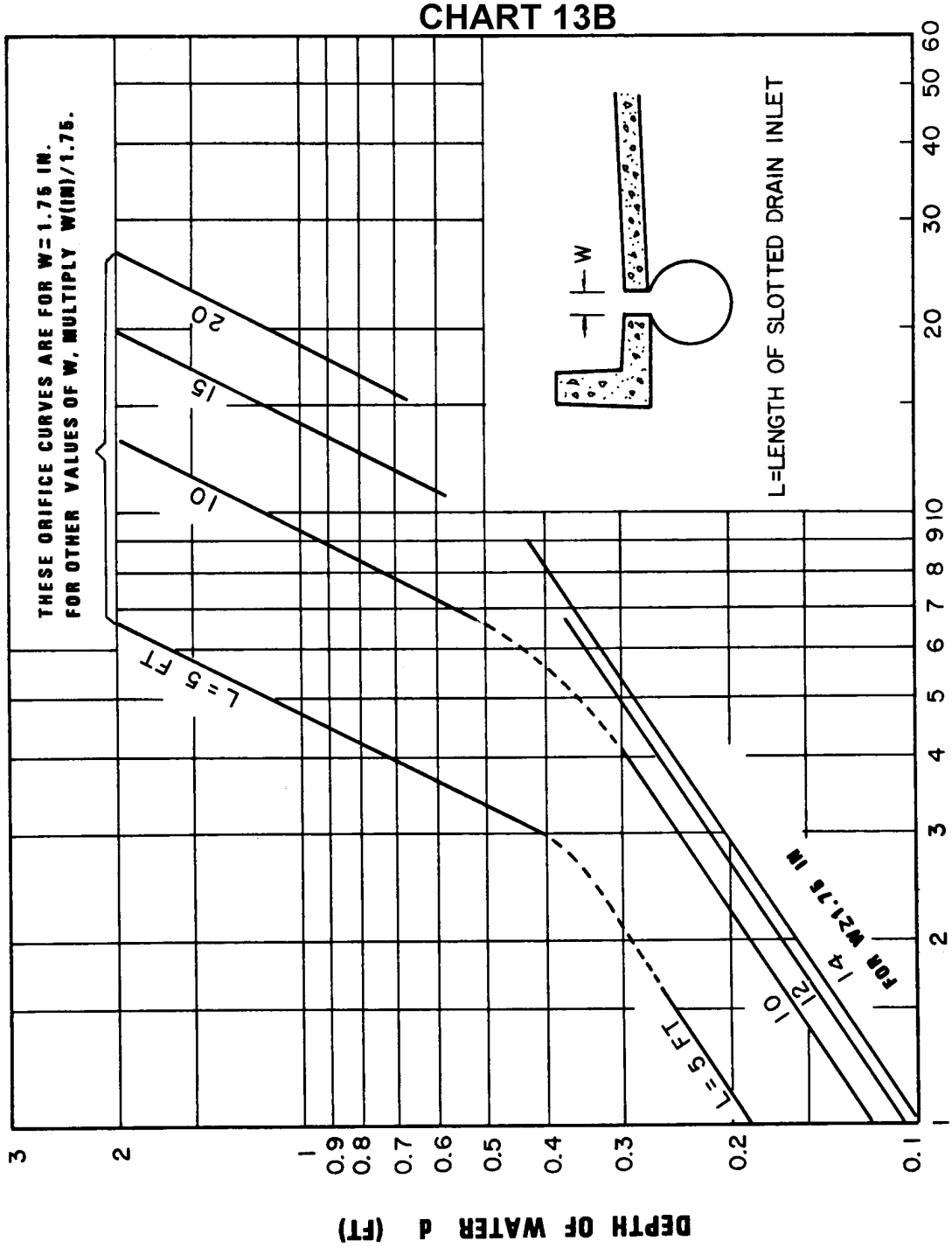
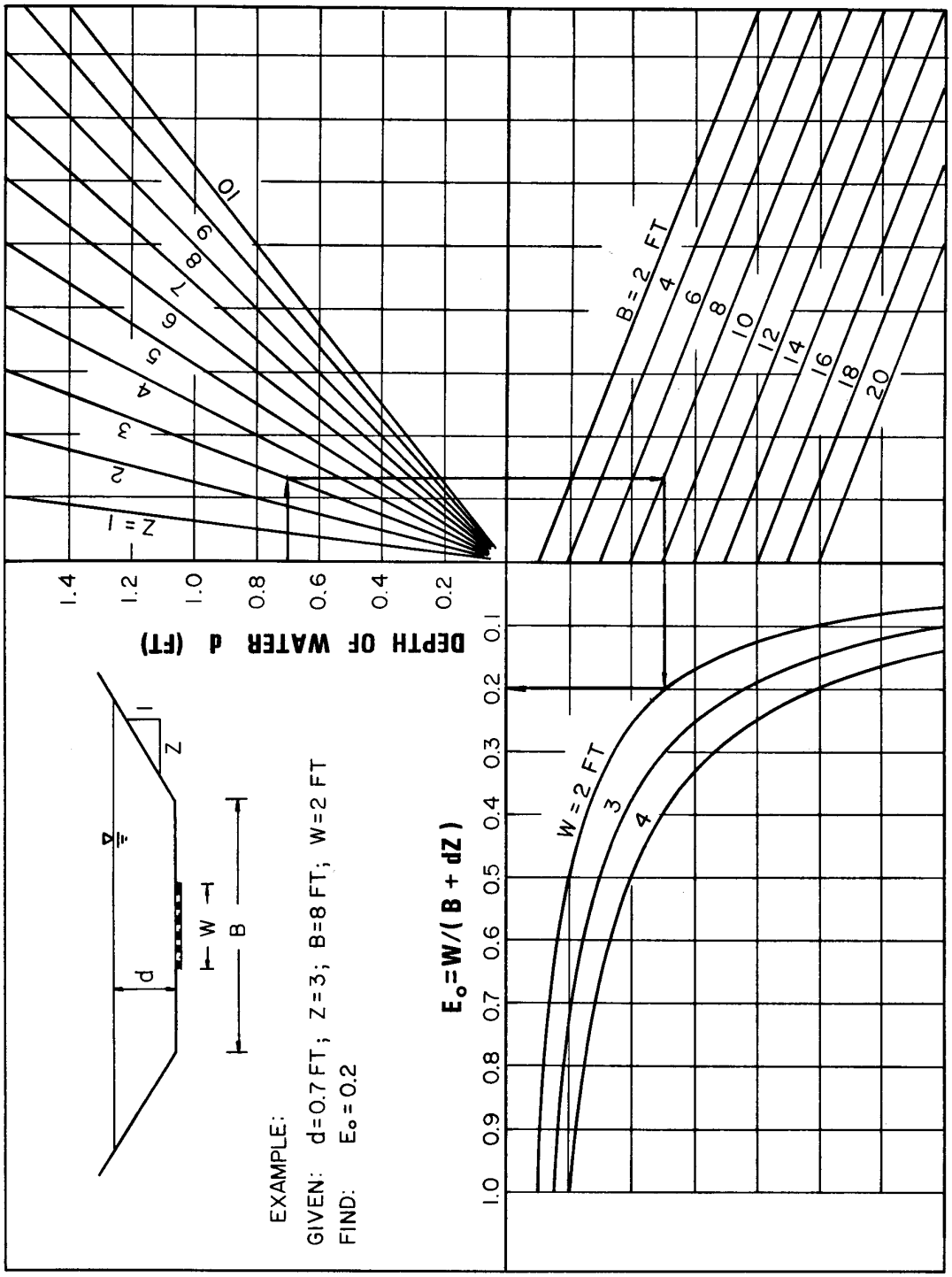


CHART 13B



Slotted Drain Inlet Capacity in Sump Locations - English Units

CHART 15B



Ratio of Frontal Flow to Totl Flow in a Trapezoidal Channel - English Units

Figure 6-130
Safety Factors for Grate Inlets

Condition	Factor of Safety
Curb on one side	1.50
No curb, i.e.: the interception takes place from all four sides	2.00

Chapter Seven

Intersections

The intersection of two or more roads presents an opportunity for conflict among vehicles. For freeways, the potential for conflict is significantly reduced through the use of interchanges. But interchanges usually are not feasible for the vast majority of intersections on arterials and collectors. This chapter is a general discussion of intersection design with those elements of particular application to this state. The details on intersection design are found in Chapter 9 of AASHTO's Green Book.

The principal objectives in the design of at-grade intersections are:

- To minimize the potential for and severity of conflicts,
- To provide adequate capacity, and
- To assure the convenience and ease of drivers in making the necessary maneuvers.

In the design of intersections there are three elements to consider:

(1) Perception-reaction distance,

(2) Maneuver distance, and

(3) Queue-storage distance.

The distance traveled during the perception-reaction time varies with vehicle speed, driver alertness, and driver familiarity with the location. Where left-turn lanes are introduced, this distance includes that to brake and change lanes. Where no turn lanes are provided, the distance needed is for the driver to brake comfortably. The storage length should be sufficient to accommodate the longest queue most commonly experienced.

An important consideration in the design of intersections is the treatment of right-turn lanes. Right turns can be free flowing, yield or stop controlled. In order to operate properly, free flowing right-turn lanes need to have an adequate acceleration distance free of access points for drivers to safely merge into the through traffic. Some drivers, particularly older drivers, are apprehensive when entering another leg of an intersection and may stop or slow down in the merge lane until the lane is clear of traffic. However, when properly designed, the majority of drivers will use the lane as proposed.

7.1 GENERAL CONSIDERATIONS

This section describes the various types of intersections and the general criteria that must be considered during design. Project intersection design configurations are developed during the project development phase based upon capacity analysis, accident studies, pedestrian use, bicycle use and transit options. In addition, design-hour turning movements, size and operating characteristics of the predominant vehicles, types of movements that must be provided, vehicle speeds, and existing and proposed adjacent land-use are considered.

Intersection designs range from a simple residential driveway to a complicated convergence of several high-volume multi-lane roadways. They all have the same fundamental design elements: (1) level of service, (2) alignment, (3) profile, (4) roadway cross section(s), and (5) sight distance. However, other

elements are introduced in intersection designs such as: speed-change lanes, turning lanes, auxiliary lanes, traffic islands, medians (flush and raised), channelization, pedestrian and bicycle accessibility, and traffic signalization.

When identified in the project scope of work, traffic calming measures may be a part of an intersection design. The Department's *Traffic Calming Design Manual* gives details on the alternative treatments and general design guidance including those for roundabouts. Additional information on the design of roundabouts can be found in FHWA's publication *Roundabouts: An Informational Guide.*

7.1.1 TYPES OF INTERSECTIONS

The three basic types of intersections are the three-leg or T-intersection (with variations in the angle of approach), the four-leg intersection, and the multi-leg intersection. Each intersection can vary greatly in scope, shape, use of channelization and other types of traffic control devices. The simplest and most common T-intersection is the private entrance or driveway. At the other extreme, a major highway intersecting another major highway usually requires a rather complex design.

7.1.2 LEVELS OF SERVICE

Levels of service for highway facilities were discussed in Chapter Two. The relationships between traffic volumes and highway capacity, together with operating speeds, provide a measure of the level of service. The characteristics of at-grade intersections can have a dramatic effect on capacity and the level of service.

Capacity analysis is one of the most important considerations in the design of intersections. Optimum capacities and improved conditions can be obtained when at-grade intersections include auxiliary lanes, proper use of channelization, and traffic control devices. The *Highway Capacity Manual* provides the procedures for analyzing the capacity of signalized and unsignalized intersections.

7.1.3 ALIGNMENT

Ideally, intersecting roads should meet at, or nearly at, right angles. Roads intersecting at acute angles require extensive turning roadway areas and tend to limit visibility, particularly for drivers of trucks. Acute-angle intersections increase the exposure time of vehicles crossing the main traffic flow and may increase the accident potential. Although a right angle crossing normally is desired, some deviation is permissible. Angles above approximately 60 degrees produce only a small reduction in visibility, which often does not warrant realignment closer to 90 degrees.

Intersections on sharp curves should be avoided wherever possible because the superelevation of pavements on curves complicates the design of the intersection. Also, this situation often leads to sight distance problems because of the sharp curve. It may be desirable to flatten the curve, or to introduce two curves separated by a tangent through the intersection. If either of these options is used, a substantial change in alignment may be necessary.

7.1.4 PROFILE

Combinations of grade lines that make vehicle control difficult should be avoided at intersections. The grades of intersecting highways should be as flat as practical on those sections that are to be used as storage space for stopped vehicles. Most vehicles must have the brakes applied to stand still unless they are stopped on a gradient flatter than 1 percent. Grades in excess of 3 percent generally should be avoided in the vicinity of intersections.

The profile grade lines and cross section on the legs of an intersection should be adjusted for a distance back from the intersection to provide a smooth junction and adequate drainage. Normally, the grade line of the major highway should be carried through the intersection, and that of the crossroad should be adjusted to it. This design requires transition of the crown of the minor highway to an inclined cross section at its junction with the major highway. For intersections with traffic

signals, or where signals may be warranted in the near future, it may be desirable to warp the crowns of both roads to avoid a pronounced hump or dip in the grade line of the minor highway. Intersections in superelevation areas are difficult to provide smooth grades or adequate drainage for and should be avoided.

7.1.5 FRONTAGE ROAD INTERSECTIONS

When a divided arterial highway is flanked by a frontage road, the problems of design and traffic control are more complex. Four separate intersections actually exist at each cross street.

The problem becomes more severe when the distance between the arterial and frontage road is relatively small. Generally, the outer separation between the two roadways should be 150 ft [50 m] or more.

Quite often, right-of-way considerations make it impractical to provide the full desired outer separation width. The alternative is to accept a narrow outer separation between cross roads and design a bulb-shaped separation in the immediate vicinity of each cross road.

7.1.6 DISTANCE BETWEEN INTERSECTIONS

Criteria for location, frequency and layout of private entrances and driveways are documented in DelDOT's *Standards and Regulations for Access to State Highways*. Illustrative sketches are shown for typical entrance and driveway designs for various conditions. For other types of public intersections, there are no fixed criteria as to frequency or distance between intersections. However, intersection spacing should provide sufficient distance to allow the proper development of all necessary turning lanes, bypass lanes, and, if signalized, proper signal coordination. Ideally this distance should be at least 350 ft [110 m] or more. Where intersections are closely spaced, several considerations should be kept in mind. It may be necessary to impose turn restrictions at some locations, prohibit pedestrian crossings, or provide frontage roads for access to intersecting roads. Where crossroads are widely spaced each at-grade intersection must necessarily accommodate all cross, turning and pedestrian movements.

7.2 TURNING MOVEMENTS

All intersections involve some degree of vehicular turning movements. There are various factors that influence the geometric design of turning lanes. The design controls for turning roadways are the traffic volume and types of vehicles making the turning movement. The roadway of primary concern is that used by right-turning traffic but may also be used for other roadways within the intersection. Figure 7-1 shows the terminology used when designing turning movements. The outer trace of the front bumper overhang and the path of the inner rear wheel establish the boundaries of the turning paths of a design vehicle.

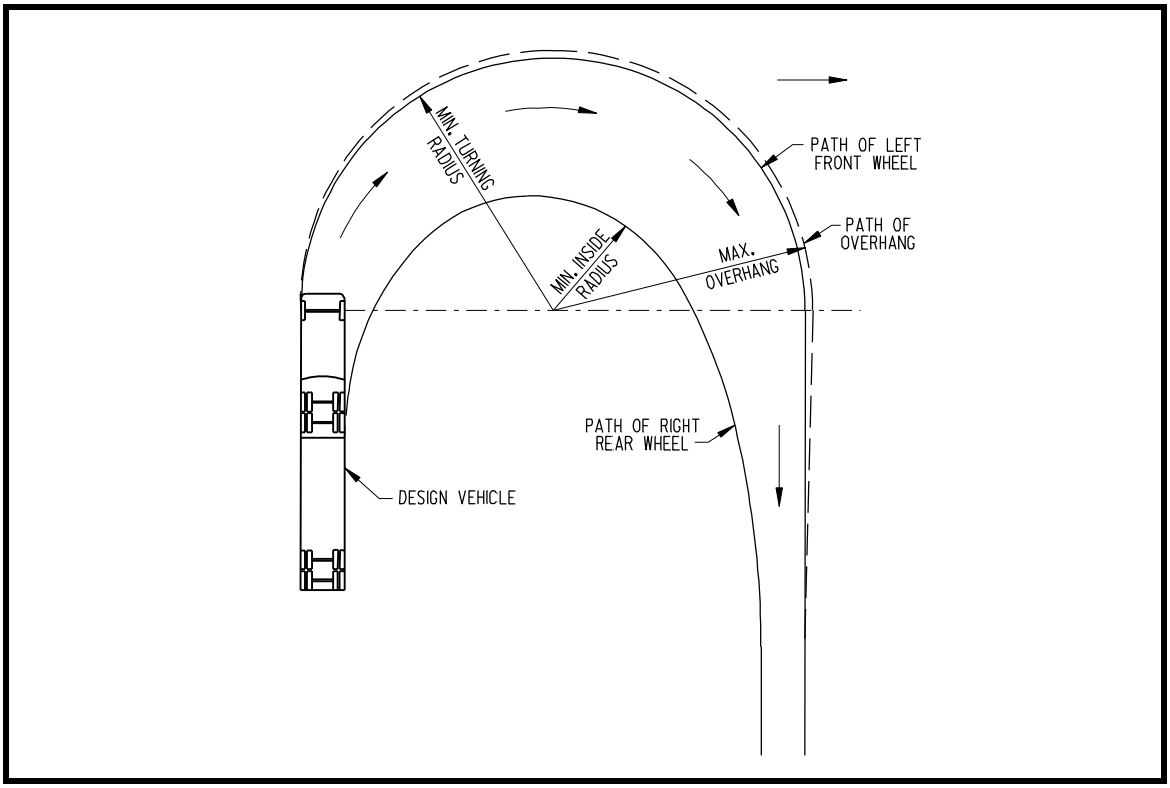
The three typical types of designs for right-turning roadways in intersections are:

(1) A minimum edge-of-traveled-way design (Green Book, pages 583 to 621),

(2) A design with a corner triangular island (Green Book, pages 634 to 639), and

(3) A free-flow design using simple radius or compound radii (Green Book, pages 639 to 649). The turning radii and pavement cross slopes for free-flow right turns are functions of design speed and type of vehicle.

Figure 7-1
Design Vehicle Turning Terminology



7.2.1 DESIGN VEHICLES

Design vehicles were briefly discussed in Chapter Two as a design consideration. The physical characteristics of vehicles using highways are positive controls in geometric design. The Division of Planning will provide the designer with information on existing and anticipated vehicle types and truck patterns. It is the designer's responsibility to examine this information and establish a representatively sized vehicle within each class for design use. Design vehicles are selected motor vehicles with the weight, dimensions and operating characteristics used to establish highway design controls for accommodating vehicles of designated classes.

The Division of Planning identifies each road section in terms of functional classification, as well as Traffic Pattern Group (TPG).

The functional classification system was discussed in Chapter Two. Each roadway segment on the state roadway system has been further classified into eight groups to represent the traffic characteristics (TPG) of vehicles using roads in the network. Visual observation in the field and permanent Automatic Vehicle Classifier (AVC) stations are used to classify vehicles resulting in the traffic composition for each road segment. A master summary table (Vehicle Classification Data Composition of Vehicle Percentages) is developed and available for designers to determine the percentage of the various types of vehicles using the facility under design. This table is directly related to the facility as defined by the Functional System.

During the classification process, vehicles are actually classified into thirteen vehicle classes ranging from motorcycles to seven or more axle multi-trailers. For design purposes these thirteen classes are combined into four

general classes of vehicles: (1) passenger cars, (2) buses, (3) trucks, and (4) recreational vehicles. The passenger car class includes passenger cars of all size, sport/utility vehicles, minivans, vans and pickup trucks. The bus class includes inter-city buses (motor coaches), city transit, school, and articulated buses. The truck class includes single-unit trucks, truck tractor-semitrailer combinations, and truck tractors with semitrailers in combination with full trailers. The recreational vehicle class includes motor homes, passenger cars with campers, cars with boat trailers, motor homes with boat trailers, and motor homes pulling cars. In addition, the bicycle should also be considered a design vehicle where applicable.

The dimensions for the design vehicles within these classes are listed in the Green Book. In the design process, the largest design vehicle likely to use that facility and its turning roadways with considerable frequency, or a design vehicle with special characteristics, is used to determine the design of such critical features as radii at intersections and radii of turning roadways.

A general guide to selecting a design vehicle is as follows:

- **P design vehicle** would be used for residential driveways, roadways with restricted truck use, local road intersections with a major roadway where use is infrequent, and low volume-minor road intersections. In most cases the selection of the SU design vehicle is preferred in all of these cases. The radii and widths permitted for the P design vehicle are very awkward for a single-unit delivery or service truck to safely maneuver.
- **SU design vehicle** provides the most economical minimum edge-of-traveled way design for rural roadways and other light truck use intersections. However, current and projected truck use on these roadways needs to be considered before a final design vehicle selection, particularly if any channelization is proposed.
- **Semitrailer combinations design vehicles** should be used where truck combinations will turn repeatedly, particularly heavily industrialized or commercial areas. Providing for these vehicles increases the paved areas, radii and other design parameters. Even in rural areas the local economy may be based on frequent semitrailer usage. Project development and scoping should identify these areas.

A project may have several design vehicles depending upon the predominant type of vehicle using the turning roadways being designed. A residential driveway would only need to consider a passenger car with an occasional single unit delivery truck. Industrial entrances would consider the predominant semi-trailer unit as the design vehicle. Other turning roadways and intersections would have to be similarly analyzed and an appropriate design vehicle selected. The purpose of this analysis is to assure the physical features are placed in positions that allow for the movement without making unsafe maneuvers, particularly within the through travel lanes, or destroying roadway features (curbs, signs, light poles, etc.).

Figure 7-2 shows selected minimum radii for the more commonly used design vehicles. Figure 7-3 (also see the Green Book, pages 216 to 224) shows the minimum design radii at the inner edge of the traveled way for roadway curves within an intersection based on a superelevation rate of 8.0 percent and free flow. For design speeds above 45 mph [70 km/h], the values are based on open road conditions. (See the Green Book, page 147.) At intersections controlled by stop signs, lower rates of superelevation are usually more appropriate. See the Green Book, pages 150 to 151, for urban streets.

Figure 7-2
Minimum Turning Radii for Selected Design Vehicles

Design Vehicle Type	Symbol	US Customary		Metric	
		Minimum Design Radius (ft)	Minimum Inside Radius (ft)	Minimum Design Radius [m]	Minimum Inside Radius [m]
Passenger Car	P	24	14.4	7.3	4.4
Single Unit Truck	SU	42	28.3	12.8	8.6
Intercity Bus	BUS-40 [BUS-12]	45	27.6	13.7	8.4
City Transit Bus	City-Bus	42	24.5	12.8	7.5
Conventional School Bus (65 pass.)	S-BUS36 [S-BUS11]	38.9	23.8	11.9	7.3
Large School Bus (84 pass.)	S-BUS40 [S-BUS12]	39.4	25.4	12.0	7.7
Intermediate Semi-trailer	WB-40 [WB-12]	40	19.3	12.2	5.9
Intermediate Semi-trailer	WB-50 [WB-15]	45	17.0	13.7	5.2

Figure 7-3
Minimum Radii at Inner Edge of Traveled Way for Intersection Curves–Free Flow

US Customary	
Design Speed (turning) (mph)	Minimum Radius (ft)
10	25
15	50
20	90
25	150
30	230
35	310
40	430
45	540
Metric	
Design Speed (turning) [km/h]	Minimum Radius [m]
10	7
20	10
30	25
40	50
50	80
60	115
70	160

7.2.2 EDGE-OF-TRAVELED-WAY DESIGNS

In the design of the edge of pavement for the minimum path of a given design vehicle, it is assumed that the vehicle is properly positioned within the traffic lane at the beginning and end of the turn. This position is 2 ft [0.6 m] from the edge of 12 ft [3.6 m] wide pavements on the tangents approaching and leaving the intersection curve.

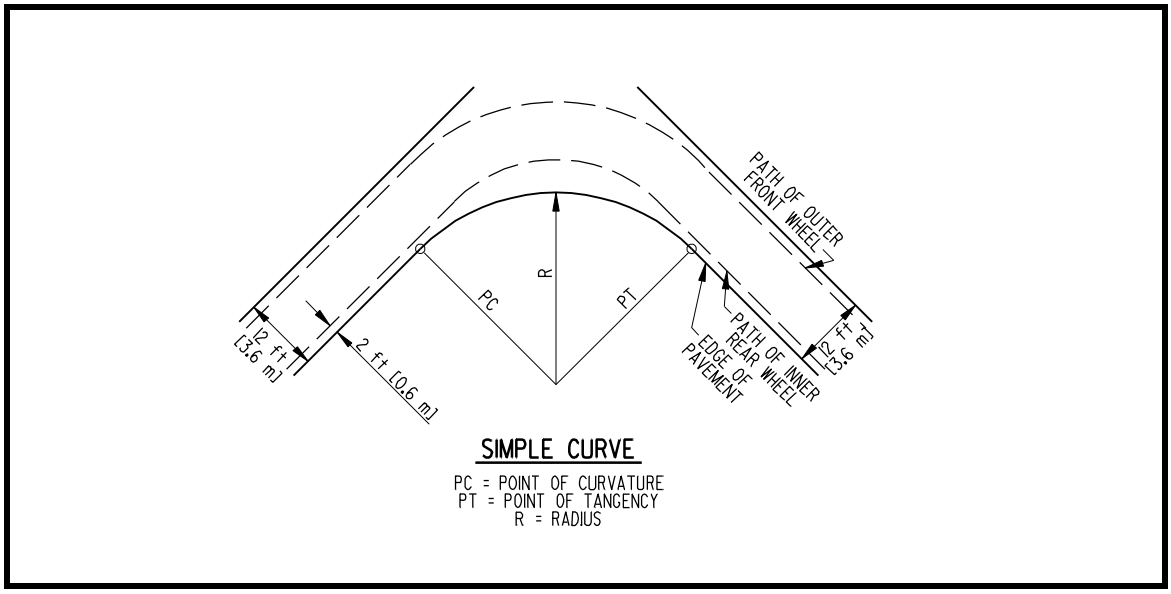
Three types of curves commonly are used for the design of pavement edges at intersections:

- simple curve,
- 3-centered symmetric compound curve, and
- 3-centered asymmetric compound curve.

Use of the simple curve usually is limited to residential driveways and low traffic volume intersections where there is little truck traffic. The 3-centered curve should be used

for edge-of-traveled-way design at all major intersections. Figures 7-4 and 7-5 illustrate the three types of edge-of-traveled-way designs. The Green Book, Exhibits 9-19 and 9-20 on pages 584 through 591 tabulate values for application of simple and three-centered compound curve applications for various angles of turning roadways. The angle of turn that is the next highest to the angle of turn of the intersection being designed should be selected.

Figure 7-4
Intersection Edge-of-Traveled-Way Design Layout
Using Simple Curves



7.2.3 PAVEMENT WIDTHS FOR TURNING ROADWAYS

The pavement and roadway widths of turning roadways at intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated, and may be designed for one-way or two-way operation, depending on the geometry of the intersection. Widths determined for turning roadways may also apply on through roadways within an intersection, such as channelizing islands.

Pavement widths for turning roadways are classified for the following types of operations:

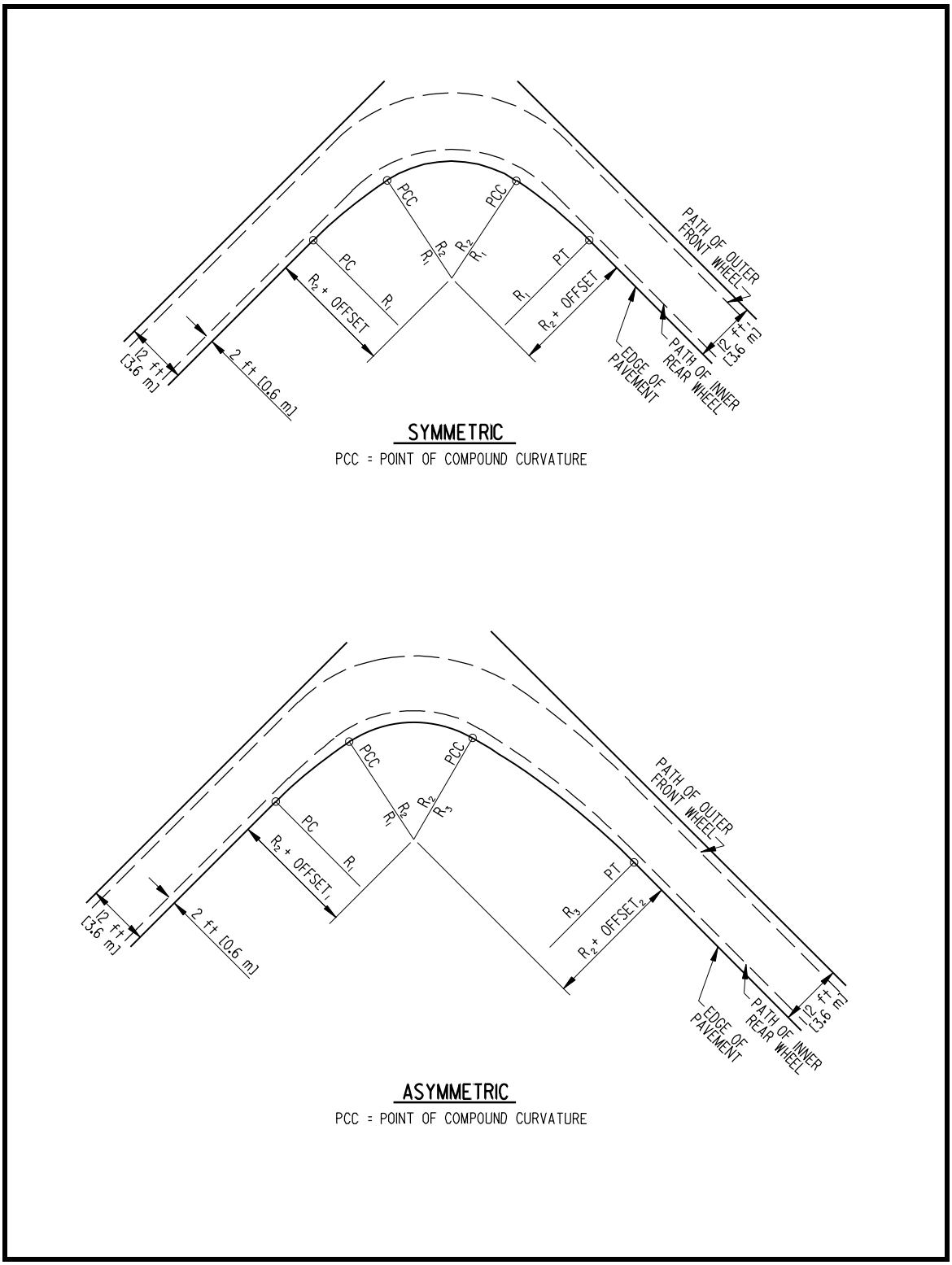
Case I – one-lane, one-way operation with no provision for passing a stalled vehicle;

Case II – one-lane, one-way operation with provision for passing a stalled vehicle; and

Case III – two-lane operation, either one-way or two-way.

Case I widths are normally used for minor turning movements and for moderate turning volumes where the connecting roadway is relatively short. The chance of vehicle breakdown is remote under these conditions, but one of the edges of pavement should be available for passing a stalled vehicle, i.e. mountable curb and clear of obstructions.

Figure 7-5
Intersection Edge-of-Traveled-Way Design Layout
Using 3-Centered Compound Curves



Case II widths are determined to allow operation at low speed and with restricted clearance past a stalled vehicle. These widths are applicable to all turning movements of moderate to heavy volumes that do not exceed the capacity of a single-lane connection. In the event of a breakdown, traffic flow can be maintained at somewhat reduced speed. Many ramps and connections at channelized intersections are in this category.

Case III widths apply where operation is two-way, or one-way with two lanes needed to handle the traffic volume. In the latter case, downstream lanes must be able to accommodate the two-lane volume. In each category the required pavement width depends jointly on the size of the design vehicle and the curvature of the turning roadway. Selection of the design vehicle is based on the size and frequency of vehicle types. The pavement width increases with both the size of the design vehicle and the sharpness of curvature. See Figures 7-6 and 7-7 for the recommended design widths of pavements for turning roadways at intersections for three types of operations and for three conditions of traffic mixes.

The designer should refer to the Green Book, pages 199 to 229, for further details on designing turning roadways within intersections.

7.3 CHANNELIZATION

Channelization is the separation or regulation of conflicting-traffic movements into definite paths of travel by traffic islands or pavement markings to facilitate the safe and orderly movement of both vehicles and pedestrians. Proper channelization increases capacity, improves safety, provides maximum convenience, and instills driver confidence. Improper channelization has the opposite effect and may be worse than none at all. Over channelization should be avoided because it could create confusion and deteriorate operations.

7.3.1 PURPOSE

Channelization of at-grade intersections is generally warranted for one or more of the following factors:

- The paths of vehicles are confined by channelization so that not more than two paths cross at any one point.
- The angle and location at which vehicles merge, diverge or cross are controlled.
- The paved area is reduced, thereby narrowing the area of conflict between vehicles and decreasing the tendency of drivers to wander.
- Clearer indications are provided for the proper path in which movements are to be made.
- The predominant movements are given priority.
- Areas provide for pedestrian refuge.
- Separate storage lanes permit turning vehicles to wait clear of through-traffic lanes.
- Space is provided for traffic control devices so they can be more readily perceived.
- Prohibited turns are controlled.

7.3.2 DESIGN PRINCIPLES

Design of a channelized intersection usually involves the following significant controls—the type of design vehicle, the cross sections on the crossroads, the projected traffic volumes in relation to capacity, the number of pedestrians, the speed of vehicles, and the type and location of traffic control devices. Furthermore, physical controls such as right-of-way and terrain have an effect on the extent of channelization that is economically feasible.

Figure 7-6
Design Widths for Turning Roadways (US Customary)

Radius on Inner Edge R (ft)	Pavement Width (ft)								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	14	17	18	20	24	26	26
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 1 ft Add 2 ft			None Add 1 ft			Add 1 ft Add 2 ft		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 12 ft where shoulder is 4 ft or wider			Deduct shoulder width; minimum width as under Case I			Deduct 2 ft where shoulder is 4 ft or wider		

Note:

Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination vehicles.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Figure 7-7
Design Widths for Turning Roadways [Metric]

Radius on Inner Edge R [m]	Pavement width [m]								
	Case I One-Lane, One-way Operation—No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-way Operation—With Provision for Passing a Stalled Vehicle			Case III Two Lane Operation Either One Way Or Two Way		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification Regarding Edge Treatment:									
No stabilized Shoulder	None			None			None		
Mountable curb	None			None			None		
Barrier curb: ** one side two sides	Add 0.3 m Add 0.6 m			None Add 0.3 m			Add 0.3 m Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B and C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width; minimum width as under Case I			Deduct 0.6 m where shoulder is 1.2 m or wider		

Note:

Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.

Traffic Condition B = sufficient SU vehicles to govern design, but some consideration for semi-trailer combination trucks.

Traffic Condition C = sufficient bus and combination-trucks to govern design.

** Dimension to face of curb; gutter pan included with surface width.

Certain principles should be followed in the design of a channelized intersection, but the extent to which they are applied will depend on the characteristics of the total design plan. These principles are:

- Motorists should not be confronted with more than one decision at a time.
- Unnatural paths that require turns greater than 90 degrees or sudden and sharp reverse curves should be avoided.
- Areas of vehicle conflict should be reduced as much as possible. Channelization should be used to keep vehicles within well-defined paths that minimize the area of conflict.
- The points of crossing or conflict should be studied carefully to determine if such conditions would be better separated or consolidated to simplify design with appropriate control devices added to ensure safe operation.
- Refuge areas for turning vehicles should be provided clear of through traffic.
- Prohibited turns should be blocked wherever possible.
- Location of essential control devices should be established as a part of the design of a channelized intersection.
- Channelization may be desirable to separate the various traffic movements where multiple-phase signals are used.

7.3.3 ISLANDS

Design of islands is the principal concern in channelization. An island is a defined area between traffic lanes for control of vehicle movements. It may range from an area delineated by barrier curbs to a pavement area marked with paint.

Islands provide three major functions:

- **Channelizing islands**–designed to control and direct traffic movement, usually turning;
- **Divisional islands**–designed to divide opposing or same-direction traffic streams, usually through movements; and
- **Refuge islands**–to provide refuge for pedestrians and bicyclists.

Most islands combine two or all of these functions. Islands may be delineated or outlined by a variety of treatments, depending on their size, location and function. Types of delineators include: (1) raised islands outlined by curbs, (2) islands delineated by pavement markings, and (3) non-paved areas formed by the pavement edges–possibly supplemented by delineators on posts or other guide posts.

Islands should be sufficiently large to command attention, and to accommodate pedestrian refuge and pedestrian signal poles where they are needed. Curbed islands normally should be no smaller than 50 ft^2 [5 m^2] for urban streets and about 75 ft^2 [7 m^2] for rural intersections; however, 100 ft^2 [9 m^2] minimum is preferable for both. Triangular islands should not be less than 12 ft [3.6 m], preferably 15 feet [4.5 m], on a side before rounding the corners; those with five foot wide curb ramps, pedestrian refuge and pedestrian signal poles should have sides that are at least 15 ft [4.5 m] and preferably 20 feet [6.0 m] on a tangent side resulting in a minimum island area of 175 ft^2 [16 m^2]. Islands with pedestrian signals and curb ramps wider than five feet will have to be larger accordingly. Median islands narrower than 8 ft [2.4 m] from back of curb to back of curb cannot be mowed effectively; therefore they should be paved. Elongated or divisional islands should not be less than 4 ft [1.2 m] wide, 6 ft [1.8 m] if pedestrians are anticipated and 20 to 25 ft [6.0 to 8.0 m] long.

DelDOT has adopted general rules for the placement of islands. The first preference is to design the intersection with radii that accommodate the selected design vehicle path with-

out creating large open paved areas requiring the need for islands to direct traffic. The overriding criteria are operational efficiency and safety. In many locations the best way to accomplish this is by the use of islands. Curbed islands usually provide the better alternative to delineate the location, minimize maintenance and control drainage. Due to the higher speeds associated with rural areas, using the proper type of curb and offsets are important design considerations. Full shoulder width offset and P.C.C. Type 2 are the preferred design elements. In locations where it is necessary to place the island closer to the traveled way, a 5 ft [1.5 m] offset is to be maintained to accommodate through movement of bicyclists.

Urban intersections will normally have curbed shoulders, parking lanes or right turn lanes on the roadway cross section adjacent to the island location. In these locations there are several alternatives for placing islands depending upon the need to accommodate bicyclists and pedestrians as well as higher traffic volumes with more complicated traffic patterns. Normally, to accommodate bicyclists, the face of curbed islands will be offset a minimum of 5 ft [1.5 m] from adjacent curb lanes. In areas with high pedestrian traffic where there is a need to minimize the distance between refuge areas, the offset for bicycles may be reduced to 4 ft [1.2 m] from an approaching curbed section. In this case, it is preferable to offset the nose of larger islands 4 to 6 ft [1.2 to 2 m] from approaching curbed lanes to allow an errant vehicle to recover before striking the curb. To accommodate the variety of vehicle, pedestrian and bicycle movements in urban areas various offsets and configurations may be needed within the same intersection. The preferred curb type for islands is mountable P.C.C. Type 2.

Figures 7-8 and 7-9 show a typical preferred island layout for rural and urban conditions.

Figure 7-8
Typical Island Layout–Rural Areas

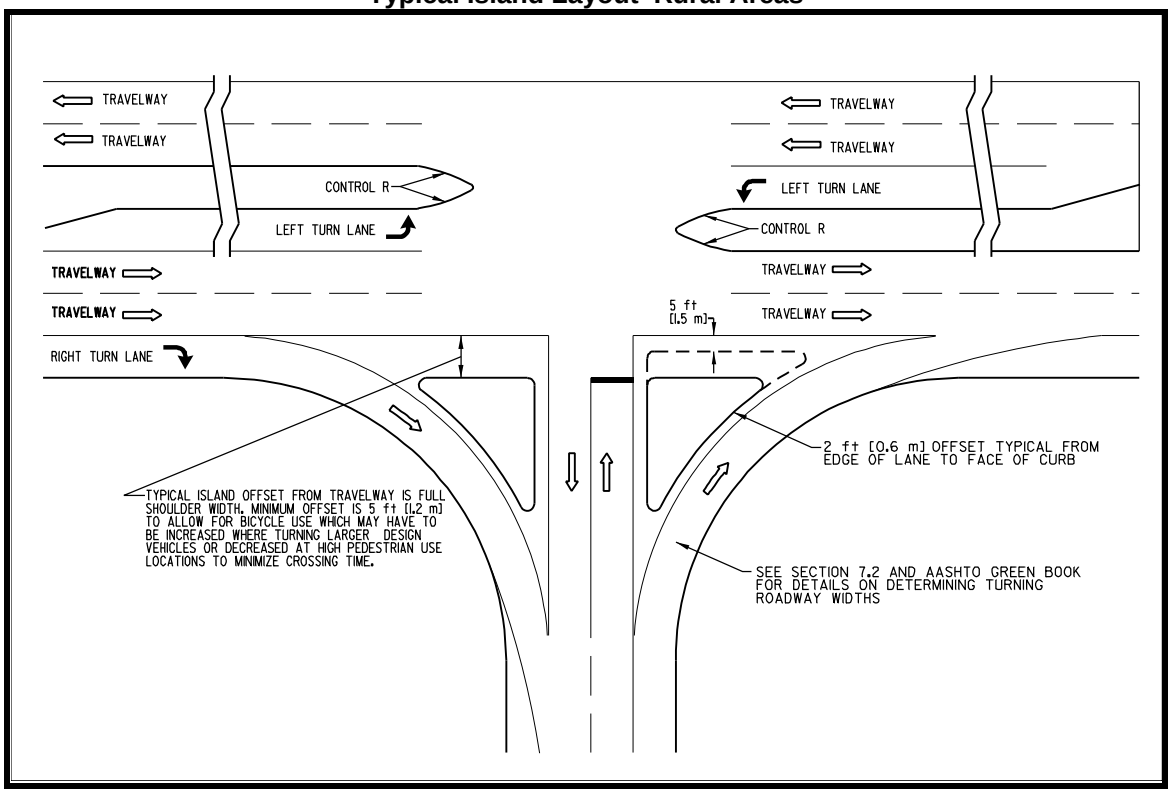
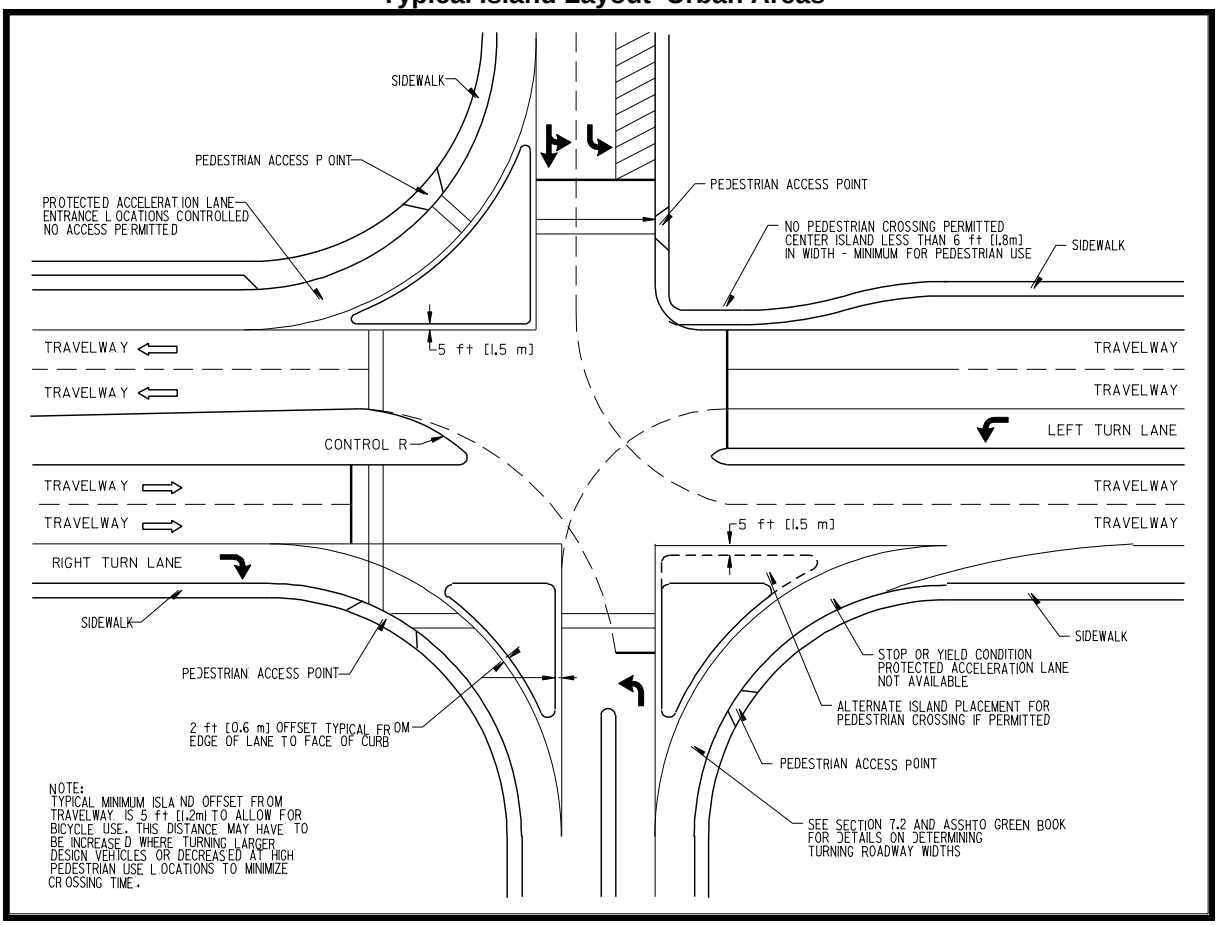


Figure 7-9
Typical Island Layout–Urban Areas



7.4 SIGHT DISTANCE

The operator of a vehicle approaching an at-grade intersection should have an unobstructed view of the whole intersection and of a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions which is termed "approach sight distance". (See Figure 7-10.) The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and the resultant distances traversed during perception, reaction time, and braking. In addition to approach sight distance, sight distance is also provided to allow stopped vehicles sufficient view of the intersecting roadway to decide when to enter the intersecting roadway or to cross it, which is "departure sight distance." Both sight distances must be checked on all intersection designs based on the procedures set forth in the Green Book, pages 650 to 679.

7.4.1 MINIMUM SIGHT DISTANCE TRIANGLE

Sight triangles are areas of unobstructed sight along both roads at an intersection and across their included corner for a distance sufficient to allow the operators of vehicles approaching simultaneously to see each other in time to prevent collision at the intersection.

The length of the legs may vary based upon traffic volumes, design speeds, operating speeds and type of intersection traffic control.

Normally in less densely populated areas the minimum sight distance of any leg would be that required to meet the design stopping sight distance for the major road. At high volume intersections the need for large sight triangles is diminished and is a function of the types of traffic control devices and the presence or absence of other vehicles. The Green Book, pages 654 to 679, provides details for determining sight triangles for several different conditions that may occur at intersections, primarily based on the type of traffic control.

In each case, assumptions are made about the physical layout and the actions of vehicle operators on both intersecting roads. For each case, the space-time-velocity relations indicate the minimum sight triangle that is required to be free of obstructions. Any object within the sight triangle high enough above the elevation of the adjacent obstruction should be removed or lowered. Such objects include cut slopes, trees, hedges, bushes, or tall crops. There should be no parking within the sight triangle.

The minimum stopping sight distance in the Green Book for open highway conditions are also valid for turning roadway intersections of the same design speed. Figure 7-11 includes stopping sight distance for lower turning speeds than commonly used under open roadway conditions. These values should be available at all points along a turning roadway, and should be increased wherever practical. They apply to both vertical and horizontal alignment.

Figure 7-10
Sight Distance Triangles—Elements for At-Grade Intersections

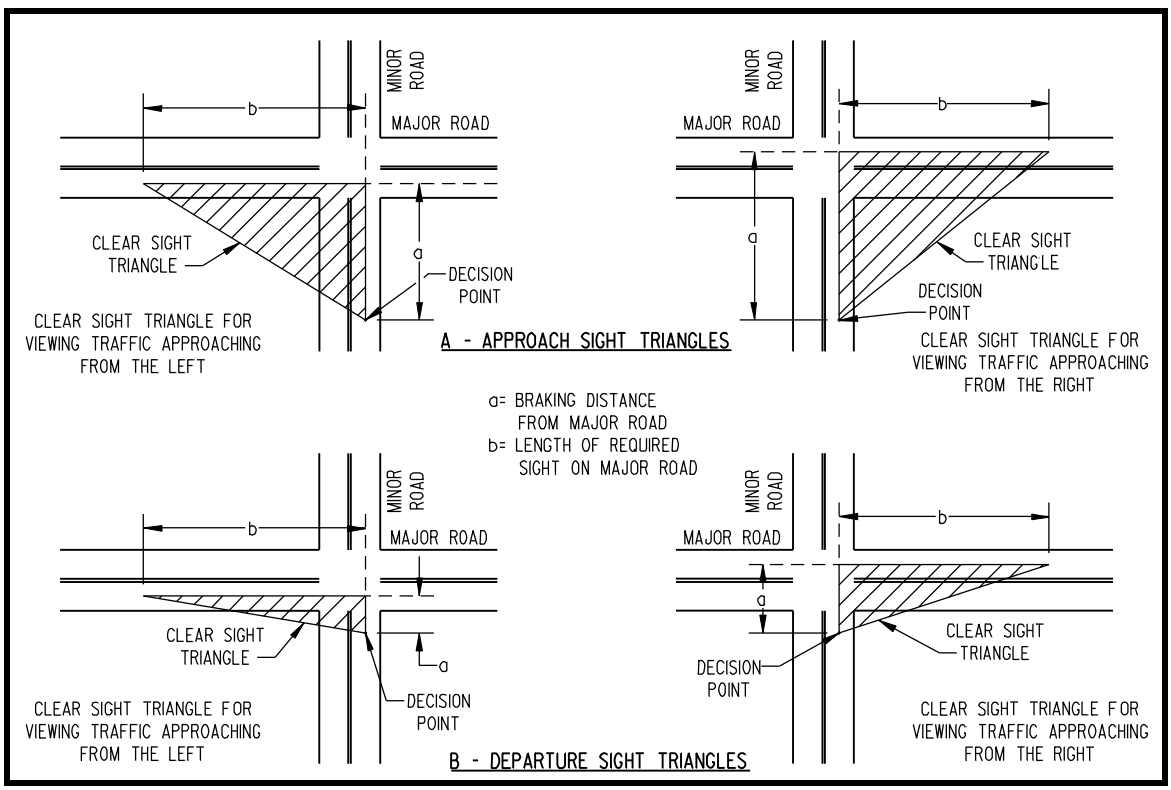


Figure 7-11
Minimum Stopping Sight Distance for Turning Roadways at Intersections

US Customary	
Design (Turning)Speed (mph)	**Stopping Sight Distance (ft)**
10	50
15	80
20	115
25	155
30	200
35	250
40	305
45	360
Metric	
Design (Turning)Speed [km/h]	**Stopping Sight Distance [m]**
15	15
20	20
30	35
40	50
50	65
60	85
70	105

7.4.2 INTERSECTION MANEUVERS

When traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position. There are three basic maneuvers that occur at the average intersection. These maneuvers are:

1. To travel across the intersecting roadway by clearing traffic from both the left and the right of the crossing vehicle,
2. To turn left into the crossing roadway by first clearing traffic on the left and then entering the traffic stream with vehicles from the right, and
3. To turn right into the intersecting roadway by entering the traffic stream with vehicles from the left.

The stop condition criterion is applicable to two-lane, two-directional roadways through multi-lane divided highways. Where the principal roadway is either undivided or divided with a narrow median (the median is too narrow to store the design vehicle), the departure maneuvers are treated as a single operation. Where the major roadway is divided and has a median wide enough to safely store the design vehicle, the departure maneuvers are considered as two operations. The first operation concerns the traffic approaching from the left for all three maneuvers; that is, crossing the entire roadway, crossing part of the roadway and turning left into the crossroad or turning right into the crossroad. The second phase concerns traffic from the right for the first two operations; i.e., continuing to cross the major roadway or turning left and merging with traffic from the right. The Green Book, pages 650 to 679, provides details on analyzing the departure sight triangles for these maneuvers.

7.5 AUXILIARY TURNING LANES

Auxiliary turning lanes may be introduced at intersections under a variety of conditions including rural or urban locations and free flowing, signalized or stop controlled traffic designs. Using auxiliary lanes to handle turning movements at high volume intersections can reduce congestion, improve safety and provide better traffic control. Auxiliary lanes are also used on four-lane divided roadways and high volume two lane roadways under open road conditions. They improve safety and traffic flow when introducing median openings, intersections at minor crossroads or U-turn locations.

Auxiliary lanes include left and right-turn deceleration lanes or right turn acceleration lanes. The length of auxiliary lanes depends on local conditions, traffic volumes, traffic mix, design speed, posted speed, selected level of service, and operating speeds. Auxiliary lanes should be 12 ft [3.6 m] wide to minimize encroachment of turning vehicles upon the adjacent travel way. In restricted urban locations where space is limited and operating speeds are low, a minimum of 10 ft [3.0 m] plus the curb offset may be the only width attainable.

Providing an area for traffic to maneuver outside of the through lanes is a very important capacity and safety feature. However, the cost of introducing auxiliary lanes is significant. Guidelines for determining the need for left-turn lanes at both signalized and unsignalized intersections have been developed. These guidelines are based on the number of lanes, the design speed, the operating speed, left-turn volumes, and opposing-traffic volumes.

For signalized intersections, a general guide is that left-turn lanes should be considered when:

- Left-turn volumes exceed 20% of the total approach volume.
- Left-turn vehicles exceed 100 vehicles during peak hour.

For unsignalized intersections, left-turn lanes should be provided:

- At all median openings on high-speed divided highways.
- On approaches where sight distance is limited.
- At non-stopping approaches of rural arterials and collectors.
- At other approaches where required based on capacity and operational analysis.

There may be other needs, primarily safety, for left-turn lanes at other locations than mentioned in these general guidelines. For two-lane roadways, a high-volume, intersecting minor roadway or entrance may also create the need to separate movements with auxiliary turning lanes.

Figure 7-12 is a tabulated guide to traffic volumes where left-turn lanes should be considered on two-lane highways. For the values shown, left-turns and right turns from the minor street can be equal to, but not greater than, the left-turns from the major street. Introducing left turn movements on a two-way roadway is more complex than on a divided highway. The design involves safely introducing two new center lanes and transitioning the two approach lanes. The design approach is shown on Figure 7-13. Figures 7-14, 15 and 16 are graphical presentations of the data presented in Figure 7-12.

In commercial and industrial areas it may be practical to provide a continuous left-turn lane in the median. Areas evaluated usually have high property values, very restricted right-of-way, and a level of development that results in many points of access to accommodate left-turning vehicles. A general criteria of 45 access points or more (both sides of road) per mile [2 kilometers] with through traffic flow rates less than 1,000 vph in each traffic direction may suggest that a continuous left turn lane would benefit safety and traffic flow. This can be accomplished by having a paved, flush median of 14 to 16 ft [4.2 to 4.8 m] in width. This area can be used for left-turn maneuvers by traffic in either direction. Continuous left-turn lanes should be used only on lower speed roadways with not more than two through lanes in either direction.

The length of a deceleration lane consists of three components: (1) entering taper, (2) deceleration length, and (3) storage length. Desirably, the total length of the auxiliary lane should be the sum of the lengths of these three components. Conditions may require the designer to accept a moderate amount of deceleration within the through lanes and to consider the taper a part of the deceleration length. Tapers are used to transition from the through lane into the full width auxiliary lane.

Straight tapers at least 100 ft [30 m] long should be used for deceleration lanes.

The designer has at least four methods available (listed from the preferred to the least acceptable) for determining deceleration lane lengths: (1) design the intersection in accordance with the HCM based on detailed existing and projected traffic data, (2) provide the desirable lengths as discussed in the Green Book (3) design left turn lanes based on the methodology shown on Figure 7-17 or (4) provide the minimum lengths as discussed in this section and shown in Figure 7-18. The use of each of these approaches is also dependent upon the roadway classification, type of facility, the location of the intersection within the facility, project scope and funding.

The Green Book proposes that for arterials with a selected design speed of 30, 40, 45, 50 and 55 mph [50, 60, 70, 80 and 90 km/h], the desirable deceleration lengths of the auxiliary lanes, where practical, are 170, 275, 340, 410 and 485 ft [50, 70, 95, 120 and 150 m], respectively. These lengths allow the driver to comfortably decelerate to a full stop from the full design speed with grades of 3 percent or less. These values do not include taper or required storage length.

The Green Book further discusses the fact that on many urban facilities providing the full length is not practical, physically possible or economically reasonable to provide the suggested desirable lengths needed for decelerating from design speed or operating speed to a full stop condition. On urban facilities in densely developed areas, the need for storage length may override the desirable deceleration length. The Green Book concludes that on urban and collectors the designer may assume that a portion of the deceleration speed is accomplished in the through lane and/or on the taper before entering the full width auxiliary lane. The Green Book further states that: “Therefore, the lengths given above should be accepted as a desirable goal and should be provided where practical.”

Figure 7-17 illustrates a design methodology for determining a reasonable minimum length for an auxiliary turning lane under open highway conditions when complete traffic data is not available. In this figure, the typical average running speed on the main facility is used and some deceleration for the left-turn movement is assumed to occur prior to entering the turning lane. Based on assumed vehicle approach speeds, the desirable deceleration lengths are shown in Figure 7-18. See the Green Book, page 851, for lengths applicable to other exit curve design speeds.The lengths shown do not include any taper lengths or required storage lengths. These lengths are for open highway conditions. It should be recognized that operating speeds, traffic volumes, traffic mix, type of facility, project intent, roadside development, and intersection frequency and spacing all influence a designer’s ability to provide the lengths shown in Figure 7-18.

To reiterate, in the use of Figure 7-17, DelDOT has adopted the recognition in the Green Book, page 714, that a degree of deceleration can safely take place in the through lane depending upon posted speed, type of facility and traffic volumes. The suggested design approach for arterial and other high volume roadways assumes a reduction of 10 mph [15 km/h] below posted speed occurs in the through lane. For collectors and other medium volume roadways, an assumed reduction of 15 mph [20 km/h] is practical. For low volume collectors and local streets, a reduction of 20 mph [30 km/h] below the posted speed may be assumed in the design of auxiliary lanes. The deceleration lengths shown in the figures are applicable to both left and right-turn lanes.

Figure 7-17 is a general guide for use when the designer does not have existing or projected traffic volumes or turning counts. When this data is available the length and design of auxiliary lanes should be analyzed in accordance with the HCM.

Figure 7-12
Guide for Need for Left-Turn Lanes on Two Lane Highways

Opposing volume (vph)	Advancing volume (vph)			
	5% Left turns	10% Left turns	20% Left turns	30% Left turns
40-mph [60 km/h] operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph [80 km/h] operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph [100 km/h] operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

For signalized intersections when there are no current or projected traffic counts or studies available to indicate the needed storage length, then the following method suggested by AASHTO can be applied.

Storage length is based on the number of vehicles likely to arrive in an average cycle time period within the peak hour in accordance with the following formula:

$$S.L. = (N \ / \ C) \times V.L. \times 1.5$$

Where:

S.L. = Storage Length,
V.L. = Vehicle length—use 20 ft [6.0 m] for passenger cars,
N = Number of left-turn vehicles in peak hour, and
C = Number of cycles per hour.

At unsignalized intersections, the average cycle time is assumed to be 2 minutes, so:

$$C = \frac{60 \text{ minutes per hour}}{2 \text{ minutes}} = 30$$

At signalized intersections, "C" is computed using the actual cycle time, so:

$$C = \frac{60 \text{ minutes per hour}}{\text{Actual cycle time in minutes}}$$

Where there is a demonstrated need due to turning volumes versus available gaps in the opposing traffic, the recommended minimum storage length for median auxiliary lanes is 50 ft [15 m]; for separate left turn facilities where no median exists, the minimum recommended storage length is 100 ft [30 m]. These lengths will allow for storing one P and one SU design vehicle or an occasional WB-50 [WB-15]. The greater length where there is no median provides allowance for a decrease in available turning paths.

Acceleration lanes for right-turning vehicles entering a traveled way may need to be considered when turning volumes exceed 100 vph. However, as discussed earlier in this chapter acceleration lanes are not always desirable where entering drivers can wait for an opportunity to merge without disrupting through traffic, such as at a signalized intersection. The use of acceleration lanes should generally be restricted to rural, free-flow, or

controlled access situations. The length of the acceleration lane is a function of the pooled highway speed and speed of the turning vehicle as shown in Figure 7-19. The Green Book, page 847, gives additional lengths for other selected entrance curve design speeds.

7.5.1 MINIMUM TURN LANE LENGTHS

A project's intent or funding may not allow for providing the desirable left-turn lengths as suggested in the Green Book, or be designed in accordance with the HCM or the method shown on Figure 7-17. In this situation and for uniform application, the suggested minimum lengths for left turn lanes are as described in this section. The parameters are:

- Suggested minimum lengths apply to divided roadways at unsignalized locations.
- No previously identified history of problems with accidents, operation or safety.
- No established warrants based on traffic volume.
- Locations with observed or anticipated high truck use need more storage length.

The lengths are determined by general class of roadway. It should be recognized that each location is unique and has to be analyzed based on its characteristics, including traffic control devices and the selected length may be different than those that follow.

Divided rural arterials and collectors with moderate to heavy through traffic with a posted speed of 50 mph [80 km/h] or greater use a taper length of 100 ft [30 m], deceleration length of 250 ft [75 m], and storage length 100 ft [30 m].

Divided rural arterials and collectors with light through traffic and a posted speed of 50 mph [80 km/h] or greater use a taper length of 100 ft [30 m], deceleration length of 150 ft [45 m], and storage length of 50 ft [15 m].

Divided urban arterials and collectors with heavy to moderate through traffic and a posted speed of 50 mph [80 km/h] or less use a taper length of 100 ft [30 m], deceleration length of 200 ft [60 m], and storage length of 100 ft [30 m].

Divided urban arterials and collectors with light through traffic and a posted speed of 50 mph [80 km/h] or less use a taper length of 100 ft [30 m], deceleration length of 200 ft [60 m], and storage length of 50 ft [15 m].

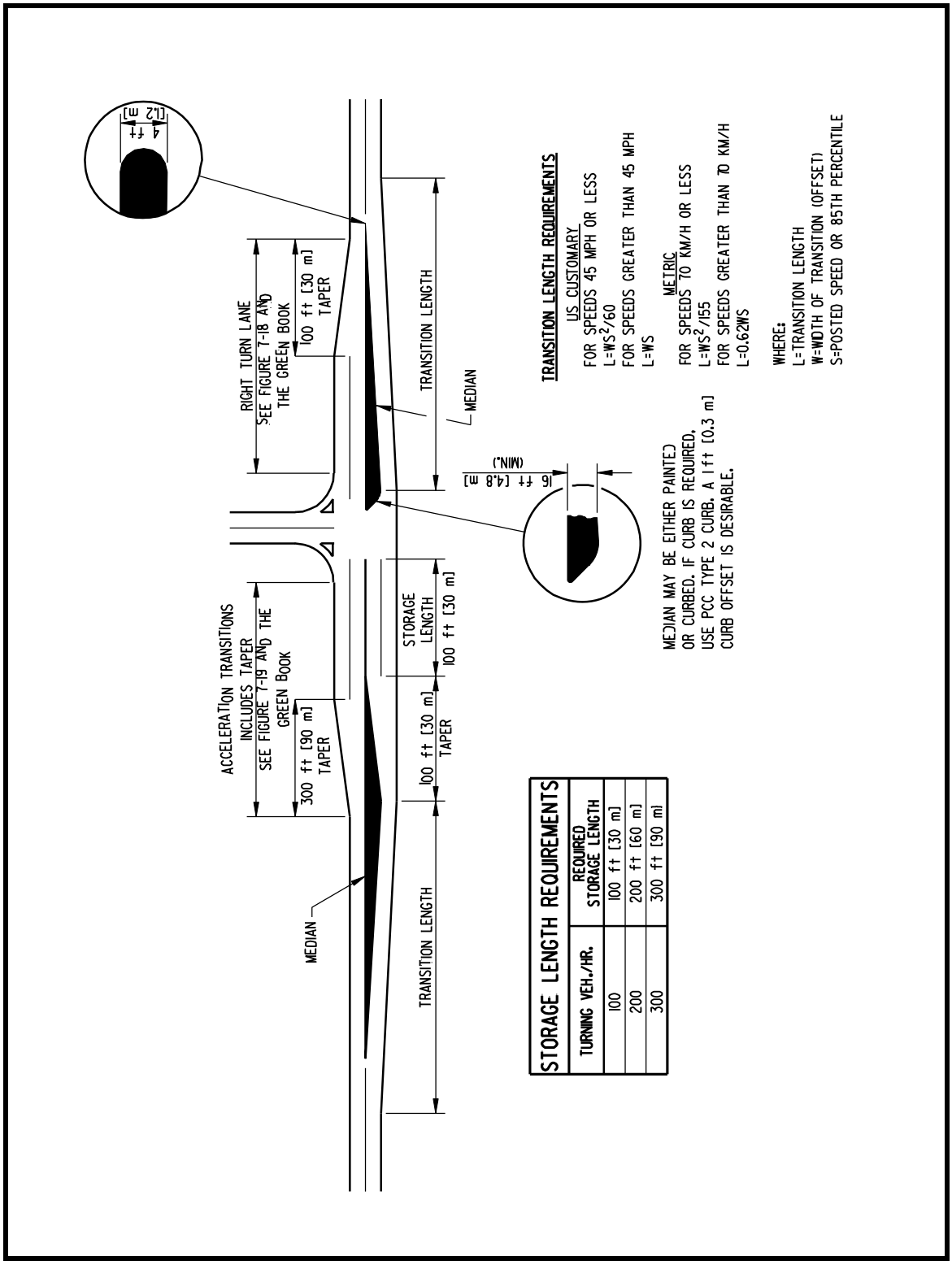
The designer should refer to the Green Book and the HCM for further discussion concerning these guidelines.

7.6 MEDIAN OPENINGS

The following is a general discussion of median opening design. The Green Book, pages 689 to 728, presents a comprehensive discussion on the concepts and design of median openings.

Median opening designs range from designing for simple U-turn movements to the more complex unsignalized and signalized rural and urban intersections that may include traffic from minor crossroads and streets or major roadways and commercial entrances. The design of median openings and median end treatments is based on traffic volumes, operating speeds, predominant types of turning vehicles and median width. Crossing and turning traffic must operate in conjunction with the through traffic on a divided highway. This requirement makes it necessary to know the volume and composition of all movements occurring simultaneously during the design hour. The discussion in this section is primarily directed to rural, unsignalized, divided roadways.

Figure 7-13
Typical Turning Lane Design for Two-lane Two-way Roadways



STORAGE LENGTH REQUIREMENTS

TURNING VEH./HR.	REQUIRED STORAGE LENGTH
100	100 ft [30 m]
200	200 ft [60 m]
300	300 ft [90 m]

Figure 7-14
Graphical Guide for Left-Turn Lane Need
40 mph {60 km/h] Operating Speed

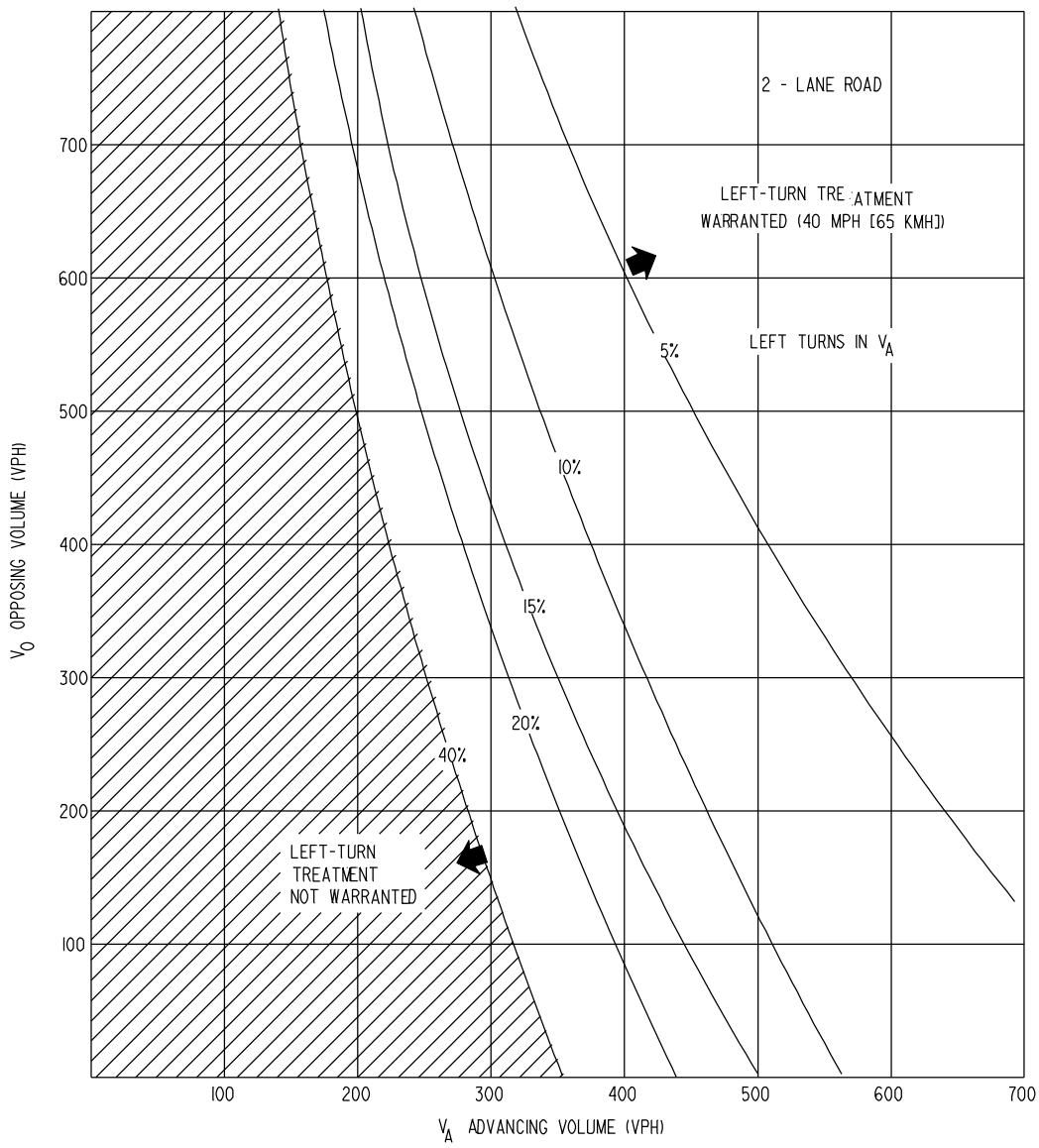


Figure 7-15
Graphical Guide for Left-Turn Lane Need
50 mph [80 km/h] Operating Speed

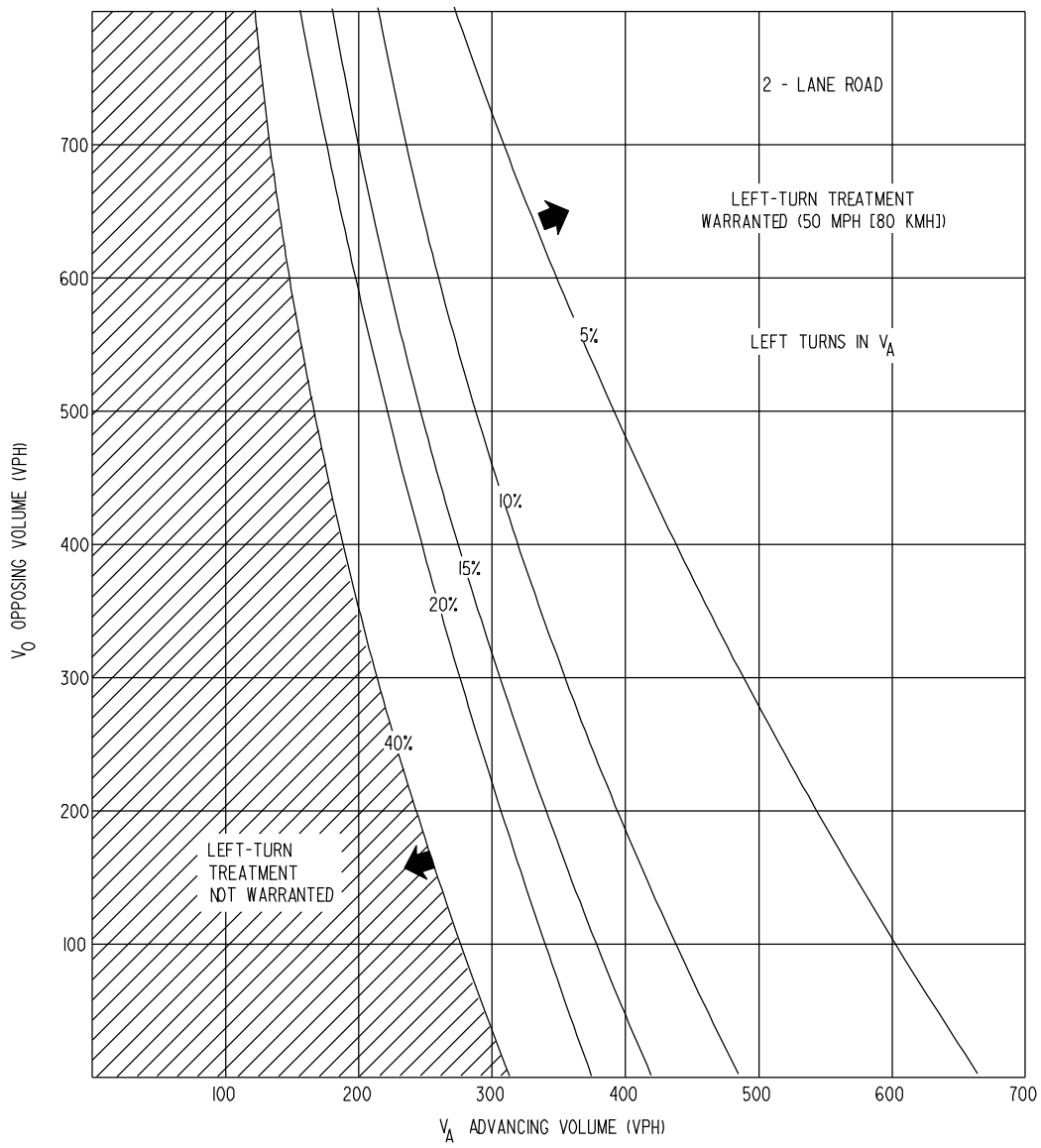


Figure 7-16
Graphical Guide for Left-Turn Lane for
60 mph [100 km/h] Operating Speed

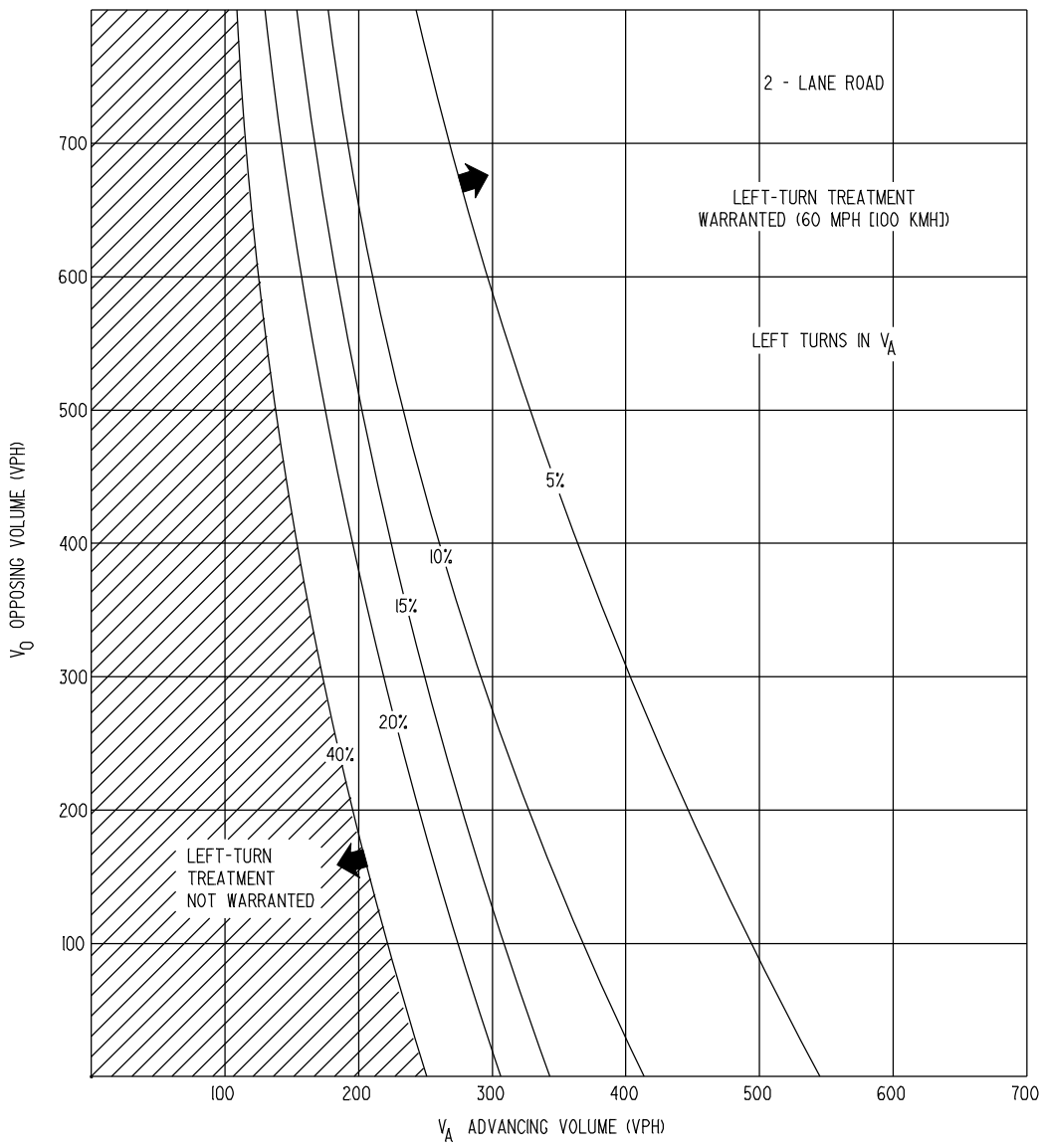
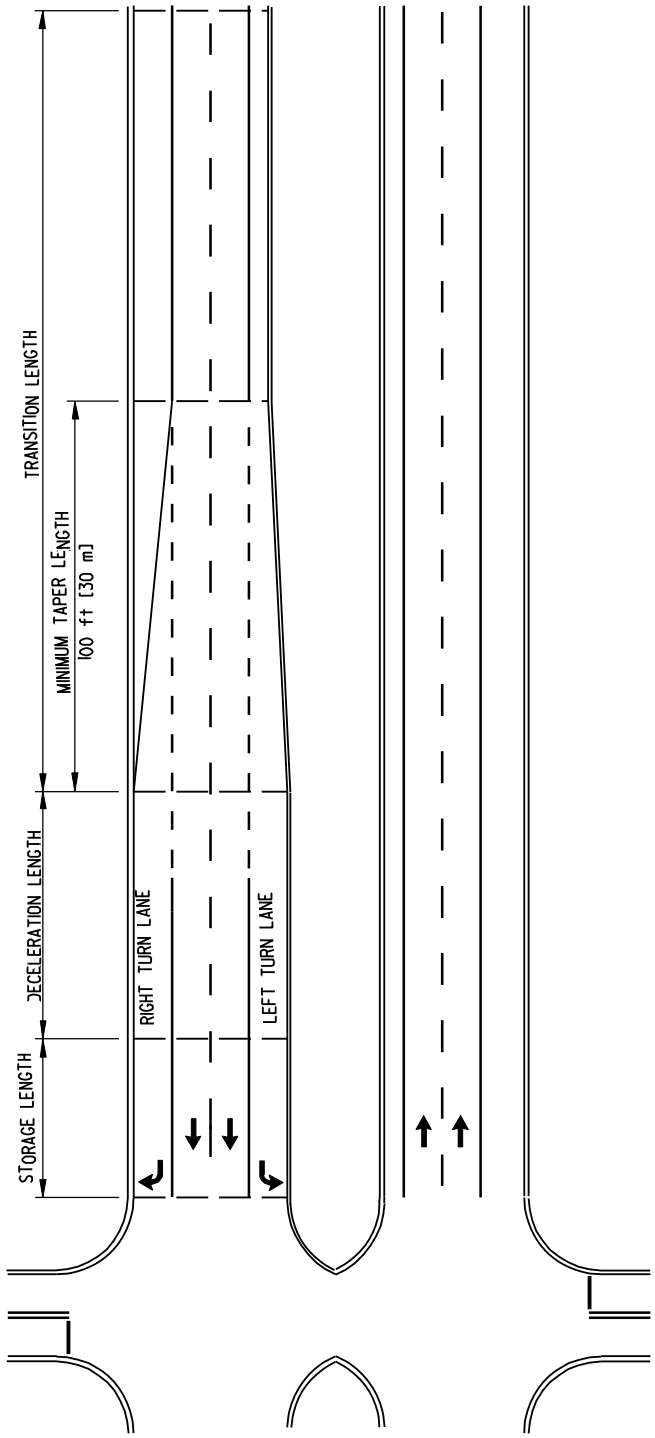


Figure 7-17
Suggested Minimum Lengths for Auxiliary Lane Design (Right And Left Turn Lane)
Four Lane Roadway–Open Roadway Conditions



SUGGESTED MINIMUM TURN LANE LENGTHS

TYPE OF FACILITY	TAPER	DECELERATION	STORAGE
RURAL ARTERIALS AND COLLECTORS-HEAVY THROUGH TRAFFIC POSTED SPEED ≥ 50 MPH [80 KM/H]	100 ft [30 m]	250 ft [75 m]	100 ft [30 m]
RURAL ARTERIALS AND COLLECTORS- LIGHT THROUGH TRAFFIC POSTED SPEED ≥ 50 MPH [80 KM/H]	100 ft [30 m]	150 ft [45 m]	50 ft [15 m]
URBAN ARTERIALS AND COLLECTORS-HEAVY THROUGH TRAFFIC POSTED SPEED OF ≤ 50 MPH [80 KM/H]	100 ft [30 m]	200 ft [60 m]	100 ft [30 m]
URBAN ARTERIALS AND COLLECTORS- LIGHT THROUGH TRAFFIC POSTED SPEED ≤ 50 MPH [80 KM/H]	100 ft [30 m]	200 ft [60 m]	50 ft [15 m]

DESIGN ASSUMPTIONS:

1. OPEN HIGHWAY CONDITIONS
2. UNSIGNALIZED INTERSECTION
3. CURRENT OR PROJECTED TRAFFIC VOLUMES AND MOVEMENTS ARE NOT AVAILABLE
4. ASSUME REDUCTION IN THE POSTED SPEED OCCURS IN THE TRANSITION LENGTH AS FOLLOWS:
   HIGH VOLUME ARTERIALS AND OTHER TYPES OF ROADWAYS - 10 MPH [5 KM/H]
   MEDIUM VOLUME ARTERIALS, COLLECTORS AND OTHER TYPES OF ROADWAYS - 15 MPH [20 KM/H]
   LIGHT VOLUME COLLECTORS AND OTHER TYPES OF ROADWAYS - 20 MPH [30 KM/H]

EXAMPLE CALCULATION:

HIGH VOLUME ARTERIAL WITH A 55 MPH [90 KM/H] POSTED SPEED
TO DETERMINE NEEDED DECELERATION LENGTH:

STEP 1. ASSUME A SPEED REDUCTION IN THE TRANSITION AREA OF 10 MPH [15 KM/H] OCCURS BY THE BEGINNING OF THE TAPER.

STEP 2. GO TO FIGURE 7-18 TO SELECT THE REQUIRED DECELERATION LENGTH BY ENTERING THE FIGURE WITH A SPEED OF 45 MPH [70 KM/H].

STEP 3. SELECT APPROPRIATE FINAL SPEED - FOR 0 MPH [0 KM/H] THE REQUIRED DECELERATION LENGTH WOULD BE 385 ft [120 m].

Figure 7-18
Minimum Deceleration Lengths (Without Taper) for Design of Exit Lanes
Urban Locations

US Customary			Metric		
Design Speed (mph)	Stop Condition	Yield Condition	Entering Speed [km/h]	Stop Condition	Yield Condition
	0 mph	15 mph		0 km/h	20 km/h
	Deceleration Length (ft)			Deceleration Length [m]	
30	235	200	50	75	70
35	280	250			
40	320	295	60	95	90
45	385	350	70	110	105
50	435	405	80	130	125
55	480	455	90	145	140
60	530	500	100	170	165
65	570	540	110	180	180
70	615	590	120	200	195

Figure 7-19
Minimum Acceleration Lengths (Without Taper) for Design of Entering Lanes
Urban Locations

US Customary			Metric		
Design Speed (mph))	Stop Condition	Yield Condition	Design Speed [km/h]	Stop Condition	Yield Condition
	0 mph	15 mph		0 km/h	20 km/h
	Acceleration Length (ft)			Acceleration Length [m]	
30	180	140	50	60	50
35	280	220			
40	360	300	60	95	80
45	560	490	70	150	130
50	720	660	80	200	180
55	960	900	90	260	245
60	1200	1140	100	345	325
65	1410	1350	110	430	410
70	1620	1560	120	545	530

Signalized intersections usually have higher volumes of traffic, more diversity in types of vehicles and a greater variety of traffic movements that have to be provided for in determining the median opening. The design will be based on more detailed traffic data and a capacity analysis.

The design of median openings is a matter of determining the traffic volumes to be accommodated and choosing the predominant design vehicle to use for the geometric and width controls for each crossing and turning movement. The proposed layout is then checked to see if larger vehicles can turn without undue encroachment on adjacent lanes. An intersection capacity analysis, using the HCM, may also be needed. If the traffic volume exceeds the capacity, the design may need to be expanded, possibly by widening or otherwise adjusting widths for certain movements. Traffic control devices such as yield signs, stop signs, traffic signals, islands or other channelization may be required to regulate the various movements effectively and to improve operational efficiency.

7.6.1 CONTROL RADII

An important factor in designing median openings is the path of the design vehicles that are anticipated to make this movement. The minimum left turn radius is that of the selected design vehicle making the turn at slow speed, 10 to 15 mph [15 to 25 km/h]. For locations that may have higher volumes, more frequent use by larger vehicles or higher turning speeds, the left-turn movement would operate more efficiently by selecting a radius of turn corresponding to a higher speed. However, the absolute minimum turning path for design and testing design layouts should be the P-vehicle. This design path does allow for an occasional single unit truck. The minimum design radius does assume the turning roadway has 12 ft [3.6 m] lanes with the vehicle located 2 ft [0.6 m] from the edge of travel way.

The paths of design vehicles making right turns are discussed in Section 7.2. Any differences between the minimum turning radii for left turns and those for right turns are small and considered insignificant in highway design.

The following control radii can be used for minimum practical design of median ends:

- A control radius of 40 ft (12 m) accommodates P vehicles suitably and occasional SU vehicles with some swinging wide;
- A control radius of 50 ft [15 m] accommodates SU vehicles and occasional WB-40 (WB-12) vehicles with some swinging wide; and
- A control radius of 75 ft [23 m] accommodates WB-40 [WB-12] and WB-50 [WB-15] vehicles with only minor swinging wide at the end of the turn.

7.6.2 SHAPES OF MEDIAN ENDS

The ends of medians at openings may be semicircular, bullet-nose or modified bullet-nose shapes. The shape normally depends on the effective median width at the end of the median. Criteria for selection of median shapes are given in Figure 7-20.

The two basic shapes are illustrated in Figure 7-21. The semicircular shape tends to create longer median openings and at crossroads leads vehicles into the opposing traffic lane. Therefore, its use is limited to medians less than 10 ft [3.0 m] in width. Even medians between 4 and 10 ft [1.2 and 3.0 m] will operate better, particularly where there is cross street traffic making left turns, if as a minimum the trailing edge of the median has a bullet nose configuration. The bullet nose design more closely follows the design vehicle's path and is formed by two symmetrical portions of the control radius and a small radius (about one-fifth of the median width) to round the nose. A narrow median that does not allow for development of the required radii will function better using a 1 ft [0.3] nose radius and then tapering the leading edge for 10 ft [3.0 m] and

the trailing edge for 20 ft [6.0 m].The bullet nose design can be designed to conform to the traffic movements permitted in the intersection. A wide median normally would have a portion of the nose flattened to be parallel to the median opening centerline and, depending upon the channelization design, a semicircular design may be more appropriate.

Figure 7-20
Preferred Median End Shapes
Based on Median Width

Controlling Median Width	Median End Shape
4 ft [1.2 m] or less	*Semicircular
4 to 66 ft [1.2 to 20 m]	Bullet Nose or Modified Bullet Nose
Over 66 ft [20 m]	Treated as separate intersection

*At locations with left turning cross road traffic, use a controlling radius of at least 40 ft [12 m].

7.6.3 LENGTHS OF MEDIAN OPENINGS

For any intersection on a divided highway, the length of the median opening should be as great as the width of the crossroad roadway pavement plus shoulders. The width and type of crossroad combined with the median width and selected control radius affect the median opening. The design should minimize any unsafe tracking encroachment into oncoming traffic from crossroads. AASHTO recommends that in no case should the opening be less than 40 ft [12 m] for a 90-degree intersection or less than the width of the crossroad pavement plus shoulders or plus 8 ft [2.4 m] for a crossroad without shoulders. Where the crossroad is a divided highway, the length of the opening should be at least equal to the width of the crossroad roadways, median and shoulders or 8 ft [2.4 m] if there are no shoulders.

Median openings are a function of median width and the selected control radius. Use of a 40 ft [12 m] minimum opening without regard to these two items should only be considered for minor, rural, unsignalized crossroads. Median openings of 50 to 64 ft [15 to 20 m] are more typical. The 40 ft [12 m] minimum length of opening does not apply to openings for U-turns where, depending upon the predominant vehicle, larger openings may be needed to ensure the vehicle can turn into the desired lane. As median widths become greater than 50 ft [15 m] the increased pavement area may create confusion as to proper vehicle paths and movements. These wider openings may need additional traffic control devices. ASSHTO recommends avoiding using median openings greater than 80 ft [25 m].

7.6.4 DESIRABLE MEDIAN OPENING DESIGNS FOR LEFT TURNS

Median openings that enable vehicles to turn on minimum paths, and at very low speeds, are adequate for intersections where traffic for the most part proceeds straight through the intersection. Where through-traffic volumes and speeds are high and left-turning movements are important, undue interference with through traffic should be avoided by providing median openings that permit turns without encroachment on adjacent lanes. This arrangement would enable turns to be made at speeds above those for the minimum vehicle paths and provide space for vehicle protection while turning or stopping.

For median openings having control radii greater than the minimum for the selected design vehicle, see the Green Book, pages 690 to 696. The three radii R, R1 and R2 control bullet-nose end designs. Figure 7-21 shows the layout. Radius R is the control radius for the sharpest portion of the turn. R_1 defines the turnoff curve at the median edge. R_2 is the radius of the tip.

When a sufficiently large R_1 is used, an acceptable turning speed for vehicles leaving the major road is assured, and a sizable area inside the inner edge of the through-traffic lane between points 1 and 2 on Figure 7-21 may be

available for speed change and protection from turning vehicles. Radius R_1 may vary from about 80 to 400 ft [25 to 120 m] or more.

The tabulated values shown in Figure 7-22 for R_1 (90, 170, and 230 ft [30, 50 and 70 m]) are established minimum radii for turning speeds under open highway conditions of 20, 25, and 30 mph [30, 40 and 50 km/h], respectively. In this case the ease of turning probably is more significant than the turning speeds, because the vehicle will need to slow to about 5 to 10 mph [15 to 25 km/h] at the sharp part of the turn or may need to stop at the crossroad. Radius R_2 can vary considerably, but is pleasing in proportion and appearance when it is about one-fifth of the median width. As illustrated in Figure 7-21, radius R is tangent to the crossroad centerline (or edge of crossroad median). Radii R and R_1 comprise the two-centered curve between the terminals of the left turn. For simplicity, the PC is established at point 2. Radius R cannot be smaller than the minimum control radius for the design vehicle, or these vehicles will be unable to turn to or from the intended lane even at low speed. To avoid a large opening, R should be held to a reasonable minimum of 50 ft [15 m].

The radii govern the length of median opening. For medians wider than about 30 ft [9 m] coupled with a crossroad of four or more lanes, the control radius R generally will need to be greater than 50 ft [15 m] or the median opening will be too short. A rounded value can be chosen for the length of opening, e.g., 50 or 60 ft [15 or 18 m], and that dimension can be used to locate the center for R. Then R becomes a check dimension to ensure the workability of the layout. The tabulation of values in Figure 7-22 shows the resultant lengths of median openings over a range of median widths, for three assumed values of R_1 and for an assumed control radius R of 50 ft [15 m]. This control radius allows for an SU design vehicle to turn without swerving into an adjacent traffic lane. Dimension B is included as a general design control and represents the distance from the P.C. of R_1 to the centerline of the median opening.

7.6.5 MEDIAN WIDTHS FOR LEFT-TURN LANES

For safety and/or capacity on divided roadways, median left-turn lanes are provided at intersections with primary, secondary and tertiary roadways on the state maintenance system, U-turn locations and, if determined necessary, major commercial and residential entrances.

The need for median openings is based upon operating speeds, traffic volumes, left-turn demand, percent of trucks, desired capacity, type of facility, density of roadside development, and the frequency of intersections. Under most conditions allowing turns from a through lane should be avoided. In fact, there is an expectation on the user's part that, except in the most congested traffic and densely developed areas, median openings will have a separate turn lane. Median turn-lane design is a function of median width, turning lane width, assumed operating speeds, intersecting roadway angles, types of traffic control devices, predominant vehicle types and the desired vehicle movements.

The available median width becomes an important element in the design. Median widths of 20 ft [6.0 m] or more are desirable at intersections with single median turning lanes, but widths of 16 to 18 ft [4.8 to 5.4 m] permit reasonably adequate arrangements in urban areas where right-of-way and operating speeds are low. Where two median turning lanes are used, a median width of at least 28 ft [8.4 m] permits the installation of two 12 ft [3.6 m] lanes and a 4 ft [1.2 m] separator. In urban areas where speeds are low and the intersection is controlled by traffic signals, a 10 ft [3.0 m] lane with a 2 ft [0.6 m] curbed separator or paint lines, or both, may be acceptable to separate the median lane from the opposing through lane. Where pedestrian use is anticipated, a 6 ft [1.8 m] separator should be provided. Left-turn and median designs have to

work together to provide an effective operation. In order to ensure this, it is necessary that the types of vehicles, the number of vehicles and their desired movements be identified.

7.6.6 MEDIAN OPENINGS FOR U-TURNS

Median openings designed to accommodate U-turn vehicles (crossovers) are provided on divided highways at intervals that serve adjacent properties without greatly inconveniencing property owners and other users. Section 7.8.3 presents general guidelines for locating crossovers. In some cases, intermediate U-turns may be included in a traffic operations plan by reducing the need for this movement at a nearby major intersection. Preferably, a vehicle should be able to begin and end the U-turn on the inner lanes next to the median but the required median widths are larger than practicable on most highways. Median widths of at least 16 ft [5 m] for passenger and 50 ft [15 m] single-unit trucks are required to turn from an inner travel lane to the outer travel lane of a two-lane opposing roadway. This does not include the width of the left-turn lane, preferably 12 ft [3.6 m]. A median width of at least 25 ft [8 m] is required to safely store a passenger car without impeding through traffic. This width will not adequately protect any other design vehicle.

The school bus is the preferred median turning design vehicle for rural divided highways. The minimum width to accommodate a school bus is 50 ft [15 m]—40 ft [12 m] length plus a clearance to each through traffic lane of 5 ft [1.5 m]. For isolated locations where there are frequent U-turns by semi-trailers it may be necessary to add a shoulder turn area or widen the existing or proposed shoulder to facilitate this movement. Figure 7-23 shows the preferred treatment for U-turns.

7.7 TRAFFIC CONTROL DEVICES

Chapter Eight and the MUTCD more fully discuss traffic control devices and their application to intersection design. Traffic control devices, such as signs, markings, signals and islands, are essential to effective traffic operations at intersections. The extent of such traffic control may range from a stop sign at a simple road approach to a complex system of synchronized traffic signals on a high-volume urban arterial. Consideration should be given to the need for traffic control devices during the geometric design of intersections–particularly those that carry considerable traffic volume with many turning movements The needed types of traffic control devices may influence the shapes of turning roadways and traffic islands. The designer needs to consider effective placement of signs and signals and the posts that support them, as well as the locations of crosswalks, where pedestrians are involved.

7.7.1 TRAFFIC SIGNALS

The determination of need, whether or not warrants are met, and design of traffic signals is the responsibility of the Traffic section in coordination with the designer. The MUTCD provides guidelines for analyzing the warrants for traffic signal installations, as well as criteria for the detailed design.

7.7.2 PAVEMENT MARKING AND SIGNING

Successful traffic operation depends largely on proper pavement marking. The designer works with Traffic in developing a project's signing and stripping plan.

All pavement marking and permanent traffic signs should be in accordance with criteria in the MUTCD and DelDOT's manuals. Chapter Eight describes this subject in more detail.

Figure 7-21
Median Nose Design Alternatives

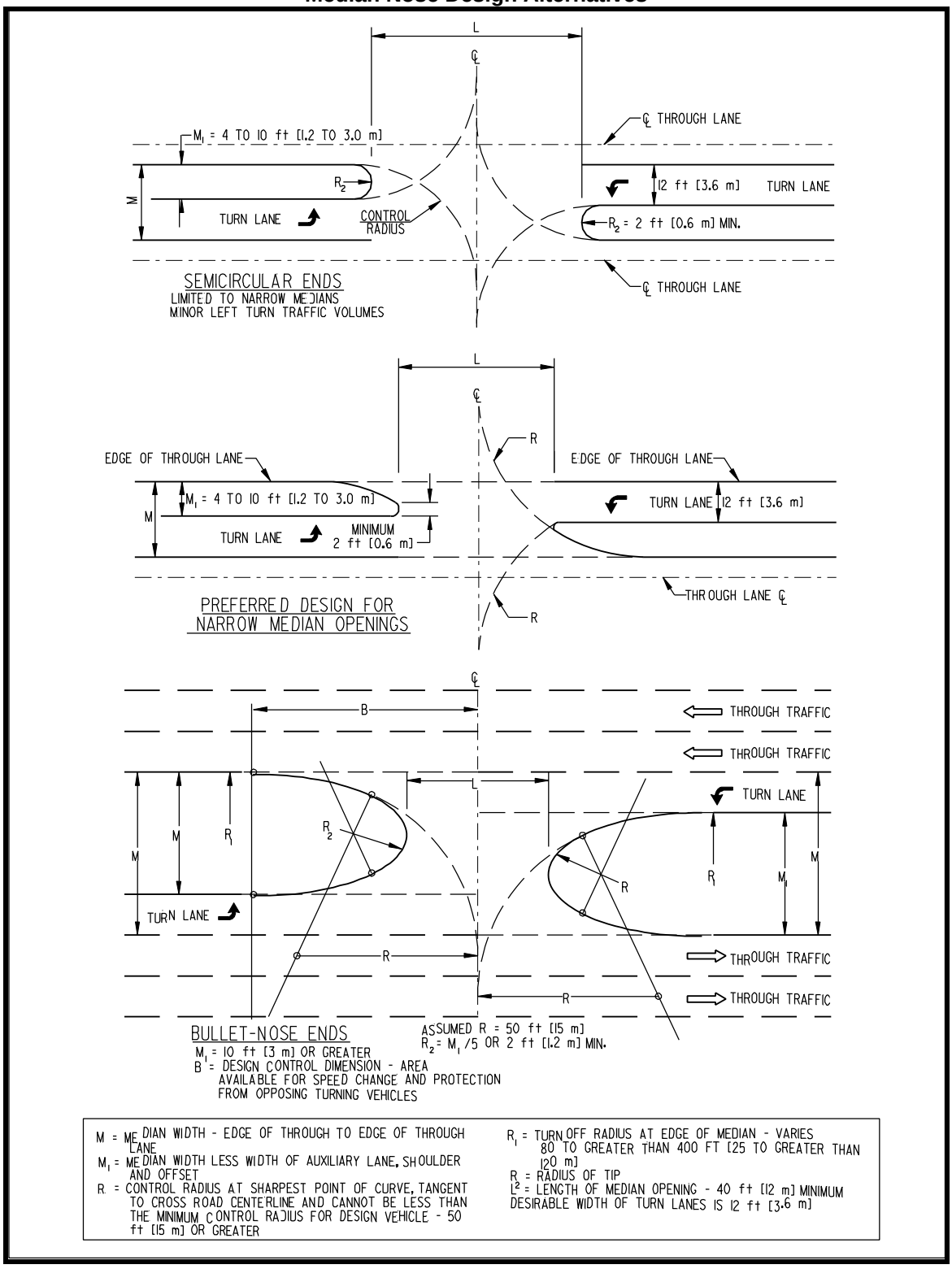


Figure 7-22
Desired Design Dimensions for Median Openings Using Bullet-nose Ends

US Customary						
M median width (ft)	**Dimensions in feet when R_1 =**					
	90 ft		**170 ft**		**230 ft**	
	L	**B**	**L**	**B**	**L**	**B**
20	58	65	66	78	71	90
30	48	68	57	85	63	101
40	40	71	50	90	57	109
50			44	95	51	115
60					46	122
70					41	128
Metric						
M median width [m]	**Dimensions in meters when R_1 =**					
	30 m		**50 m**		**70 m**	
	L	**B**	**L**	**B**	**L**	**B**
6.0	18.0	20.2	20.2	24.4	21.3	27.6
9.0	15.1	21.4	17.7	26.5	19.0	30.4
12.0	12.8	22.4	15.6	28.3	17.1	32.7
15.0			13.8	29.9	15.4	34.7
18.0					13.8	36.7
21.0					12.4	38.4

7.8 ACCESS CONTROL GUIDELINES

Driveways and entrances are the simplest types of intersections. However, dealing with access to the highway and control of access can be one of the most difficult tasks of the designer. It is particularly difficult on projects involving roadways constructed prior to today‘s more stringent land-use and access regulations.

7.8.1 STANDARDS

DelDOT’s Entrance Manual provides the designer with criteria for locating and designing driveways and entrances. The designer should be thoroughly familiar with these standards and make every effort to conform with them in the project design to the extent that is reasonable.

The design of projects on new alignment should adhere strictly to DelDOT’s entrance standards. If that is not possible for any reason, there should be documentation for the variance and approval obtained to deviate.

The standards must be used with caution and judgment on reconstruction and other type projects. Existing entrances affected by highway construction must be designed using engineering judgment to obtain as safe an access facility as possible given the restrictions peculiar to the site. In the absence of a demonstrated need, changes in access control that are costly and affect the property owner significantly are to be avoided.

Considering the needs, existing access provisions and the standards, the designer should strive for uniformity in the access provided to all property owners within the project. Property owners are extremely sensitive to lack of uniform treatment. When conditions make it impossible to do this, the property owner should be informed of the reasons for the lack of uniform treatment.

Safety is a primary consideration. The accident history should be checked to determine if there are an excessive number of accidents or unusual situations associated with entrances to the highway or at crossovers on divided highways. Special attention should be given to modifying those entrances with features or characteristics that have been identified as a possible factor in causing accidents.

The development of standards for retrofitting access control is extremely difficult because of the variability among sites. Rather than adopt a set of standards, some general objectives and guides are provided which should be considered when control of access is being addressed in the design process.

7.8.2 GUIDELINES–ENTRANCES

In retrofitting an existing point of access, the following criteria should be applied wherever possible.

- The point of access must be positively controlled by the use of curbs, islands or landscaped areas. Islands and landscaped areas must be constructed so as to restrict vehicular movement and force it to the defined point of access.

- No more than two points of access should be provided for each property.

- The number of movements permitted per point of access must be consistent with desirable traffic operations. The point or points of access should be located as far from any nearby intersection as possible.

- Entrance widths shall be sufficiently wide to accommodate turning movements necessary to obtain access to the site. The type of traffic expected to use the entrance must be considered. Where islands are constructed as part of a project, they should be set back from the roadway edge at least a distance equal to the shoulder width.

- An important consideration in entrance design is ensuring that internal parking arrangements do not interfere with the required sight distance. A developer's tendency is to provide as much parking as physically possible. Properly designed islands and other channelization can assure that adequate sight distance is provided.

In general, use of the right-of-way by private interests is to be avoided. Such use is frequently encountered, however, particularly on roads constructed many years ago. The public's interest should never be compromised. However, when a means of access control can be accomplished that meets the objectives of providing safe access to state controlled roads, consideration will be given to permitting present operations to continue. In such instances, an agreement should be made between the Department and the property owner establishing appropriate conditions, etc.

7.8.3 GUIDELINES–CROSSOVERS

Crossovers on divided highways that do not have control of access serve several important functions:

- They allow for U-turn movements to serve adjacent land uses.

- They allow local road or street traffic to enter or cross the major roadway.

- They can reduce turning volumes at major intersections by providing for turns away from the major intersection.

- They provide access for emergency and maintenance vehicles.

Crossovers are also points of vehicle conflict and potential accident locations. Therefore, the placement and location of crossovers require careful consideration to maximize their benefits while minimizing their accident potential.

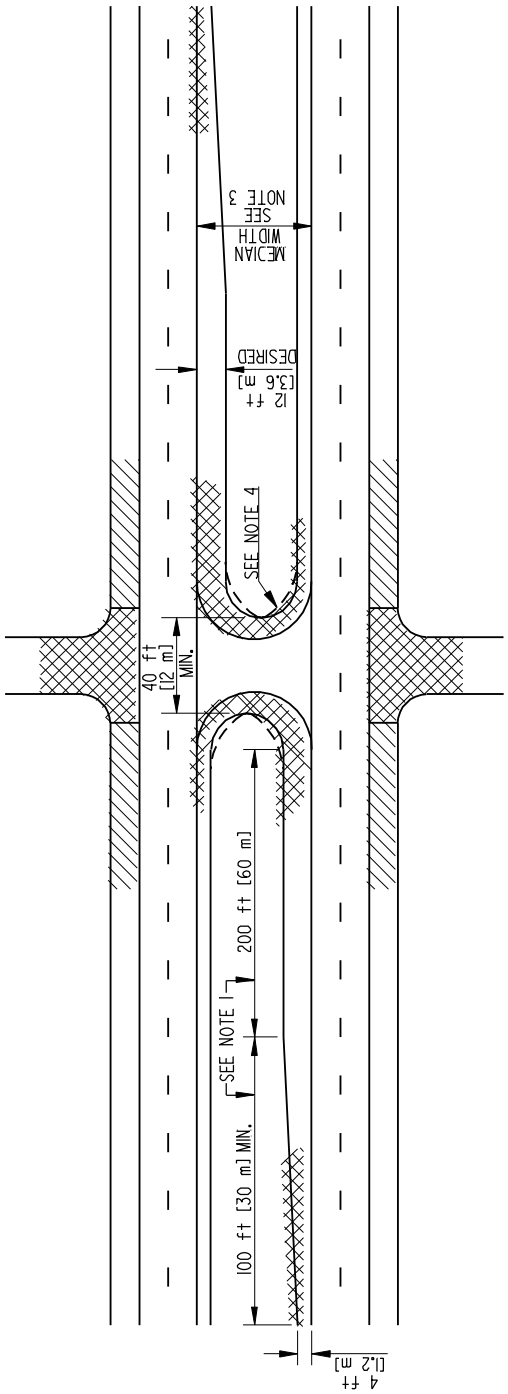
In general, crossovers should be located where sufficient sight distance is available. They should not be located on curves that require superelevation. Median side slopes adjacent to crossovers should be properly graded to reduce their potential as an obstacle to out-of-control vehicles and to provide proper drainage. In urban and other developed areas, the spacing of crossovers should not be reduced to the point where adequate lengths of left-turn lanes cannot be constructed and adequate signalization and coordination cannot be achieved. For new construction projects without control of access, crossovers should be located approximately 1000 to 1500 ft [300 to 1500 m] apart in urban areas and 2000 to 3000 ft [600 to 900 m] apart in rural areas.

On reconstruction and other lower type projects, the accident history at existing crossovers should be researched and any geometric deficiencies should be identified. Changes in crossover geometrics or channelization should be considered to address these problems. Crossovers should be removed only after careful consideration of how traffic patterns will be affected. In some instances, the problem that the designer is trying to correct by removing a crossover could show up at another crossover location because of the new traffic patterns. The removal or relocation of crossovers is potentially controversial and can be an important part of the public information program.

7.8.4 PUBLIC AWARENESS

Throughout the design process, it is very important to keep the public informed about the project, with special reference to proposed access changes. Contact with the public may be made through individual contacts, plans-available meetings or public meetings. See the Project Development Manual for details.

Figure 7-23
Typical Crossover Design for
U-turns and Minor Intersections on Rural Divided Roadways



NOTES:

1. CONSISTS OF A 100 ft [30 m] TAPER, 150 ft [45 m] DECELERATION LANE AND A 50 ft [15 m] STORAGE LANE. THESE ARE THE MINIMUM RECOMMENDED LENGTHS FOR EACH. IF THERE ARE OTHER CONDITIONS THAT EXIST SUCH AS HIGH TRUCK USAGE, HIGH TRAFFIC VOLUMES OR HIGH POSTED SPEEDS, THE LENGTHS MAY HAVE TO BE INCREASED BASED ON THE DISCUSSION IN SECTION 7.5 ON AUXILIARY LANES, THE GREEN BOOK AND FIGURE 7-17.
2. U-TURNS BY SCHOOL BUSES AND SEMI-TRAILERS CANNOT BE ACCOMPLISHED WITH A MEDIAN LESS THAN 50 ft [15 m]. THE DESIGN WILL HAVE TO INCLUDE SOME SHOULDER IMPROVEMENTS TO ALLOW FOR THIS MOVEMENT.
3. THE MINIMUM MEDIAN WIDTH TO SAFELY STORE A P-DESIGN VEHICLE IS 25 ft [7.5 m] AND A SU-DESIGN VEHICLE IS 50 ft [15 m].
4. FOR INTERSECTIONS WITH MINOR ROADS, CROSS STREETS, AND MAJOR ENTRANCES, THE MEDIAN NOSE SHOULD BE DESIGNED AS A BULLET NOSE WITH A SELECTED DESIGN VEHICLE RADIUS.

Chapter Eight
TRAFFIC SERVICES

The purpose of traffic control devices is to aid in insuring highway safety by providing for the orderly and predictable movement of traffic, and providing information, guidance and warnings as needed for safe operation of vehicles. Based on their use, they are generally classified as regulatory, warning, guidance or informational. Regulatory devices are the most important as they must be adhered to in order to maintain the safe movement of the mix of traffic. State laws specifying penalties for drivers, bicyclists and pedestrians who do not abide by these signs reinforce their importance. Warning signs advise of potential hazardous traffic operations. Guidance and information signs aid the driver to more easily find local sites, institutions, etc.

8.1 GENERAL CRITERIA

Because of a recognized need for reasonable nationwide uniformity and consistency in traffic control devices, the *Manual on Uniform Traffic Control Devices* (MUTCD) has been developed through a national advisory committee. This manual has been approved by FHWA for all highways and streets open to public travel. Under authority granted by Congress, the federal Secretary of Transportation has decreed that traffic control devices in each state shall be substantially in conformance with the most current edition of the MUTCD. These standards have been adopted by DelDOT and are applicable to all highways, roads and streets under DelDOT's jurisdiction.

The designer should be aware that DelDOT does develop separate publications that are based on the information found in the MUTCD. These publications cover applications that have been found to perform the best in this state and the designer should contact the Traffic Section for the latest publications.

The MUTCD is a comprehensive publication that provides the warrants, criteria and guidelines for virtually all traffic control installations. The subject matter discussed in the manual includes the following:

- Signs,
- Pavement markings,
- Traffic signals,
- Islands,
- Traffic controls for street and highway construction, maintenance, utility, and emergency operations,
- Traffic controls for school areas,
- Traffic control systems for railroad-highway grade crossings, and
- Traffic controls for bicycle facilities.

The MUTCD and supplemental supporting publications prepared by the Traffic Section are very comprehensive and stand on their own as guidelines for traffic control measures. It would be impractical to reproduce all the information found in them within this Chapter. Emphasis here will be placed on policies, procedures and responsibilities for effective application of the standards.

8.2 DESIGN PRINCIPLES

The first consideration in designing traffic control devices is that there is an optimum number. In many situations, as more devices are installed the goal to provide a safe and efficient transportation system is less likely to be reached.

To maximize the value and obtain the optimum installation of traffic control devices, the designer should ask the following questions when developing the signing and striping plans, and the maintenance of traffic plans (MOT):

- Is there a need for the particular device being considered and what is the correct type for the application under consideration?
- Is the device properly located and sized to command attention?
- Does the device convey a clear, simple and concise meaning?
- Has the device been so over used on the project or misplaced to the point where a driver no longer respects it, but in fact ignores it or even reacts adversely?
- Is the device located and sized with a message that provides the driver with adequate time to react in the manner intended?

To answer these questions the designer, along with traffic, construction, and maintenance personnel, must employ the following considerations.

8.2.1 UNIFORMITY

Conformance with the MUTCD assures that the size, contrast, colors, shape, composition, lighting and reflectorization are combined in a manner that draws attention: the shape, size, colors, and conciseness of the message convey the meaning intended; legibility and size combined with location provide adequate reaction time; and uniformity, size, legibility and the purpose for which the device is used commands a driver's respect.

8.2.2 PLACEMENT

The designer should assure the device is: in the viewer's uncluttered cone of vision so that it will command attention; and placed at the proper point near or at the object, or in the situation to which it applies to aid in conveying the proper meaning.

8.2.3 OPERATION

The designer should assure that the appropriate devices and related equipment are installed to meet the traffic control needs at a given location. The uniform and consistent placement and operation of devices will assure that drivers will respond properly, since they will have been exposed to similar traffic control situations previously. This principle is as important or perhaps more important for the maintenance of traffic during the construction phase of a project. Adapting to the frequent changes in traffic flow and mix is difficult enough for the driver without compounding it with inconsistent signing.

8.2.4 MAINTENANCE

The effort expended in assuring good design, placement and operation of devices will be of little value if the devices are not properly maintained. It is necessary that legibility be retained, that the device is visible and that it is removed when no longer needed. In addition to keeping the devices clean and legible, they should be properly mounted, in good working condition, and when replacing or shifting locations assuring that their functional integrity is maintained.

8.2.5 CONSISTENCY

Treating similar traffic control situations in the same manner simplifies the driver's recognition and understanding of the conditions to be anticipated. Consistency translates into faster reaction time and greater compliance with the functional intent of the device.

8.3 RESPONSIBILITIES

The Divisions of Transportation Solutions and Maintenance and Operations share the responsibilities for assuring adherence to the MUTCD during plan preparation and for the application of these standards to control traffic during the construction and maintenance phases of projects. Maintenance and Operations will advise designers on which products are performing the most effectively.

8.3.1 TRANSPORTATION SOLUTIONS

The assigned design team is responsible for ensuring that the final project plans include the necessary traffic control devices, including projects assigned to consultants for design. The design team is responsible for the design of the roadway sections and geometrics related to the installation of the traffic control devices. The team provides the design and plan preparation for signing and striping plans, construction phasing with necessary traffic control plans (MOT) and standard construction details for traffic control devices as determined appropriate for repetitive use in contract plans.

The Bridge Design Section is responsible for designing the overhead and large ground mount sign supports and foundations for most in-house design projects and the review of designs prepared by consultants.

Considerable coordination between the project design team and the Traffic Section is required throughout the stages of designing a project. Many decisions need to be made early in plan development, and will become an integral part of the design and defining the final right-of-way needs. Items to be coordinated include the following:

- Intersection geometrics — including the need for and the use of channelization and auxiliary lanes;
- The need for and use of traffic signals — including pole location, required coordination, associated underground conduit, and, in particular, the right-of-way required to accommodate the installation;
- Locations of any desired highway lighting or traffic sign illumination, identification of power sources and the need for any underground conduit to be installed during construction;
- The need for overhead signing or large ground-mounted signs that require consideration for support structure locations as well as inclusion in the work schedule of Bridge Design or consultants;
- The identification of a basic plan for any needed detours and traffic control measures to be incorporated in the contract plans; and
- The identification of any unusual pavement marking or other traffic service device concepts.

The findings of these coordination meetings are formally documented and serve as a basis for completing the detailed design and construction plans.

The Traffic Section and Maintenance and Operations provide a continuous review and technical resource during project development and construction of a project's traffic services. The areas of concentration are constructibility, product selection and its proper installation, operation feasibility, safety of the workers and the traveling public, and the practicality of future maintenance of the installed devices.

Maintenance and Operations personnel consult regularly with Transportation Solutions on the effectiveness of the different traffic control measures and plans employed on projects. In addition, any unusual local conditions that may require special consideration or a major change in the proposed treatment is also brought to the attention of Transportation Solutions. Again, engineering judgment and experience are an important element in the field implementation of traffic control plans and devices.

8.3.1.1 TRAFFIC SECTION

In the preparation of contract plans, Transportation Solutions' Traffic Section acts both in a support and review role. In its support role, the section provides the designer with:

- Engineering studies related to the use of particular devices at a requested location(s) within a project;
- Preliminary layout, final design and plan preparation for signals, signal control and coordination plans;
- Preliminary layout for overhead signs and large ground mounts with messages;
- Highway lighting and illumination, including for overhead signs,
- Detailed information for the preparation of off-site detours;
- Changes in the standard details for traffic device installation, and
- Formulation and dissemination of traffic control criteria and procedures related to construction and maintenance operations.

In their review role, the Traffic Section participates in all project reviews. Their review emphasis is placed on signal locations and coordination, overhead and large ground mount sign locations and messages, the maintenance of traffic plans and the final signing and striping plans showing the traffic operation after project completion.

It should be pointed out that the MUTCD is not a substitute for engineering judgment. It is intended that the manual be a standard for providing a basis for the uniform approach in the use and installation of traffic control devices, not a legal requirement for their use and installation. The exercise of engineering judgment and experience is inherent in the selection of traffic control devices.

After contract award of a project, a construction group assumes primary responsibility for implementing and maintaining the various temporary and permanent traffic control measures provided in the contract documents. They ensure that the devices are constructed and installed as intended. In particular, they ensure that the prescribed traffic control measures employed during the construction phasing are carried out effectively.

8.3.1.2 SAFETY SECTION

The Safety Section is an integral part of a project's review process during the design phase as well as construction. Their emphasis is on construction sequencing and the associated traffic control during the various stages of construction. They are also responsible for the proper field implementation of the various devices ensuring compliance with the MUTCD and established Department guidelines.

8.4 IMPLEMENTATION GUIDELINES

The following general guidelines are applicable to the design and installation of the various categories of permanent traffic control devices.

8.4.1 SIGNS

Signs should be used only where warranted by facts and field studies. Signs are essential where special regulations apply at specific locations or where hazards are self-evident. Ex-

cessive use of signs may result in confusion, reducing their effectiveness.

Project signing usually includes three types: regulatory, warning and guide signs. How, when and where to use these signs is a function of the class of facility being designed.

Part 2 of the MUTCD provides the designer with detailed guidelines necessary to develop the signing plans for a project. The elements emphasized are consistency, uniformity and geometric design coordination to ensure that they are effective. Due to the importance of maintaining uniformity in signs, the designer does not normally have the freedom to develop unique signing. Size, shape, color, and locations of signs should be in conformance with the criteria set forth in Part 2 of the MUTCD. The Manual provides the sign design details defining the size, color, shape, letter size, mounting height, etc. for all the various types of signs. During plan development, the designer should refer to the sections that describe the general concepts that make a good signing plan. These include ensuring there is consistency of application, there is no over use, there is standardization of location, and the project geometrics allow for the installation of the signs to perform as intended.

However, the designer should remember that even in the best of conditions, visibility is a factor at night and during inclement weather. Therefore, although it may appear as overuse or redundant, additional signing to provide the driver with the information needed at critical locations or situations may be required.

The Traffic Section is to be consulted during the preparation of sign layouts, sign details, sign messages, quantity estimates, and cost estimates.

8.4.2 PAVEMENT MARKINGS

The striping plan is usually developed concurrently with the signing plan. Proper and consistent pavement markings are important to all drivers, especially the elderly, as they place a high priority on what they see on the pavement. This is particularly true during periods of reduced visibility such as at night or during inclement weather. Centerline, lane and edge line markings provide reliable and effective guidance. The advantage of pavement markings under normal conditions is that the driver does not have to divert attention to receive the warning, guidance or information intended.

The effectiveness of pavement markings is reduced quickly by heavy traffic, weather and other environmental conditions. Therefore, in most situations and conditions, pavement markings are used to supplement the traffic control design. Line markings, words and symbols shall be of the shapes, sizes and colors as set forth in Part 2 of the MUTCD.

The designer should coordinate product selection for striping with Operations and Maintenance as heavy traffic use and environmental conditions will affect the longevity of the striping material.

Permanent pavement markings are to be identified and quantified as contract items. The type of material to be used on a project will be specified by Operations and Maintenance and the Traffic Section during project reviews. The type of material that will be recommended depends on the past performance, pavement material, highway classification, anticipated traffic volumes and mix, and other factors that enter into a materials cost/benefit ratio.

8.4.3 TRAFFIC SIGNALS

Traffic signals are valuable devices for the control of vehicle and pedestrian traffic, but unless they are adequately justified and designed, they can adversely affect traffic operations.

Properly located and operated traffic signals can provide the following advantages:

- Provide orderly movement of traffic;
- Increase intersection traffic capacity;

- Reduce the frequency of certain types of accidents, particularly right-angle type; and
- Interrupt heavy traffic flow at intersections to permit vehicles or pedestrians to cross.

Conversely, traffic signals that are poorly located, poorly operated or unwarranted can cause the following adverse effects:

- Excessive delays in traffic;
- Disobedience of the signal indications;
- Use of less adequate routes to avoid the perceived unwarranted imposed delays; and
- Increase accident frequency, usually of the rear end type.

Part 4 of the MUTCD lists eight warrants to be included in an engineering study for justification for installing a traffic signal. The conclusion of this study should not be that one or more of the warrants have been met, but that installation of the traffic signal will improve the overall safety and/or operation of the location under consideration. The warrants to be studied include:

- Eight-hour vehicular volume;
- Four-hour vehicular volume;
- Peak hour volume;
- Pedestrian volume;
- School crossings;
- Coordinated signal system;
- Crash experience; and
- Roadway network.

The MUTCD provides a detailed description and guidelines for studying each of these warrants. The Traffic Section is responsible for performing the detailed engineering analysis related to the warrants for signal installation at a particular location. Traffic is also responsible for the detailed design and preparation of plans and specifications for signal installations. The plans include underground conduit locations with appurtenances, traffic coordination signal systems, quantity estimates and cost estimates.

8.4.4 ISLANDS

Another effective method of traffic control is the installation of islands. Islands can be used to separate traffic lanes and movements or provide refuge for pedestrians. The island can be created by pavement markings or physically constructed.

Islands, depending upon their construction and placement can pose a safety hazard and maintenance problem. Therefore, the designer should not use them in unnecessary locations. Chapter 7, Intersections, provides more detail on the use and design of islands.

8.4.5 MAINTENANCE OF TRAFFIC DURING CONSTRUCTION

Problems of traffic control occur when traffic must be moved through or around street and highway construction and the associated maintenance operations and utility work. No one standard sequence of signs or other control devices can be set up as an arrangement suitable for all situations due to the variety of conditions encountered. The designer is responsible for the Maintenance of Traffic Plan (MOT). In preparing the MOT plan, the designer must consider the construction sequencing, and during which and how many seasons of the year construction will take place. Materials used for temporary use may not be as durable as those used for the final signing and striping and thus may have to be replaced several times during construction phasing.

This plan is closely coordinated with DelDOT's Chief Safety Officer. The scope of the plan is determined during the preliminary design phase of a project. The actual MOT plan sheets are usually prepared concurrently with the construction phasing plans.

The principles for preparing the MOT plan are found in the MUTCD and applicable DelDOT publications.

8.4.6 OTHER TRAFFIC CONTROLS

Parts 7, 8 and 9 of the MUTCD present guidelines and criteria for traffic controls for school areas, railroad-highway grade crossings and bicycle facilities, Bicycle facilities are addressed in greater detail in Chapter Ten of this manual. The provisions in the MUTCD are applicable to related projects in Delaware and should serve as a reference source for designers preparing the MOT plan.

8.5 ROADWAY LIGHTING

8.5.1 OBJECTIVES

Nighttime visibility on roadways has long been recognized as a problem for motorists. Reflective lane lines, edge lines, delineators and signing were developed to help alleviate the problem. However, conditions on some roadway sections are such that additional measures, such as partial or continuous lighting, are needed to improve visibility and safety.

While providing lighting on all roadways might be desirable, it is impractical. Lighting is costly and expensive to install, operate and maintain. Statistically, nighttime fatal accident rates average higher than for the daytime. However, the benefits of lighting roadways must be worth the cost, as lack of nighttime visibility is only one of several factors influencing higher accident rates.

The design goal of highway lighting is to ensure that vehicular and pedestrian traffic can see well enough to react and function quickly, accurately and comfortably for the situation and conditions being encountered.

Good nighttime visibility not only maintains the efficiency and protects the large investment in the transportation system but provides other indirect social and economic benefits such as:

- Reduction in accidents;
- Aids in the prevention of crime;
- Increases roadway capacity and traffic flow;
- Promotes the use of commercial and industrial areas at night;
- Enhances and encourages community activity; and
- Saves in accident costs, lost working time and human injury.

8.5.2 DESIGN RESPONSIBILITY

The design team should submit preliminary plans, early in project development, for evaluation by the Traffic Section for highway lighting consideration and the development of designs, if warranted.

8.5.3 GENERAL WARRANTS AND CONSIDERATIONS

The lack of nighttime visibility is one of several factors that affect nighttime driving. Other factors include:

- Confusion and distraction caused by background lighting;
- Loss of environmental clues;
- Headlight glare and misuse;
- Adverse environmental conditions;
- Increase in driver fatigue;
- Increased occurrence of impaired drivers;
- Changes in the mix of traffic; and
- Changes in driver attitudes.

Since lighting is expensive and lack of visibility is not the only factor that affects nighttime driving, warrants have been established to determine when lighting should be considered. The evaluation of projects for lighting, whether partial or continuous follows the guidelines established in AASHTO's manual *An Informational Guide for Roadway Lighting* and the Department's guide *Highway Lighting Policy-Installation Determinates.* As with most AASHTO guidelines, the roadway's classification is a major factor in establishing the need for lighting. For determining lighting needs,

two general classifications are used based on traffic movement and access. The highest types of facilities are controlled access highways such as freeways, interchanges and expressways. The second type of facilities are non-controlled access highways such as major arterials, streets and highways, collector streets, minor highways, and local streets and roads.

The AASHTO guide provides a basis for the installation of highway lighting. This guide also recognizes that an agency has a considerable long term cost associated with lighting. Therefore, the guide provides only minimum conditions, allowing an agency to establish higher warranting values, including local conditions and subjective values, and stresses that meeting the warrants is not the only criteria for justifying whether or not lighting is to be installed. Therefore, the decision to use highway lighting is not reached through applying empirical, easily assigned values to graphs, charts and forms. Instead, it is reached through the use of engineering judgment as applied to the analytical methods adopted by AASHTO, considering the user needs and the resulting user benefits. The warrants used, based on research that has determined that the primary factors affecting the need for roadway lighting, can be classified as:

- Geometric,
- Operational,
- Environmental, and
- Accidents.

The geometric factors used consider that a driver's ability to perform the tasks required to operate a vehicle safely and efficiently is largely influenced by the roadway geometrics. Much of the needed information for the driver to mentally and physically perform effectively is transmitted to the driver through a facility's geometrics. Restricted access to this information, as with reduced visibility, affects a driver's responses.

The operational factors used consider a facility's level of service, operating speed, median width, pedestrian usage, types and frequency of signals, and channelization.

For the driver, the environmental conditions adjacent to a roadway are an important influence on how the driver will perform. The factors considered include the adjacent land use, access points, and any background lighting which may be distracting or objectionable to the driver.

Since a change in accident patterns normally occurs when lighting is installed, a roadway's accident history is a factor in determining lighting needs. Accident experience is developed and the night-to-day accident ratio and its relationship to other similar types of facilities become an important part of the decision-making.

8.5.4 GENERAL LIGHTING DESIGN CONSIDERATIONS

Lighting design includes determining a power source, the type of luminaire, the type of lamp, mounting height, luminaire overhang, luminaire spacing, pavement reflectance and energy consumption. These are all influenced by the roadway geometrics and characteristics, the adjacent environment, expected maintenance, economics, aesthetics, and overall objectives.

The design should provide uniform lighting on the pavement surface, reduce glare from the installation placement itself, reduce headlight glare, and provide adjacent lighting spillover only where desired. In addition, the lighting design should gradually transition the illumination into and out of the site allowing the driver's eyes to adjust. Lighting pole locations should be in conformance with a project's clear zone and have the least number of poles necessary to meet the design criteria.

AASHTO provides two design methodologies—illuminance and luminance. Illuminance is based on determining the average illumination measured in lux on one square meter of roadway area. The luminance method determines the amount of reflected light from the pavement surface visible to the driver's eyes.

The Department has adopted the illluminance method for designing highway lighting.

The design level of illuminance on a facility and other design considerations are established in the Department's publication *Highway Lighting Policy-Installation Determinates.*

8.6 SIGNING AND STRIPING GUIDELINES

The purpose of signing and striping a facility is to improve traffic flow and traffic safety by providing the driver with adequate guidance, warnings and regulatory information. Both have a service life, but striping is particularly subject to rapid wear and pavement adhesion problems. Lack of visibility at night, when wet and during periods of snow cover also reduce the effectiveness of striping.

The value of striping is that it supplements and enhances other devices. It is the only traffic device that can convey certain messages without diverting the driver's attention from the roadway. The performance limitations of striping materials, however, may need to be supplemented with proper signing which requires that the signing and striping plans be designed as a total package.

The MUTCD discusses the general principles that govern the design and use of signs and pavement markings. Again, the level and priorities given for application of these principles to a facility are based on the AASHTO classification system. The interstate system receives the highest attention and local streets the lowest. This heirachy applies to the application of traffic control devices and their maintenance.

The Department has utilized the MUTCD principles and prepared a set of standard details for signing and striping different types of facilities and situations that may occur on a facility. These typical layouts are included as figures at the end of this chapter. The designer should use these to develop the signing and striping plans.

Figure 8-1
Signing Guide - Junction of Dual Route/ Signalized Road

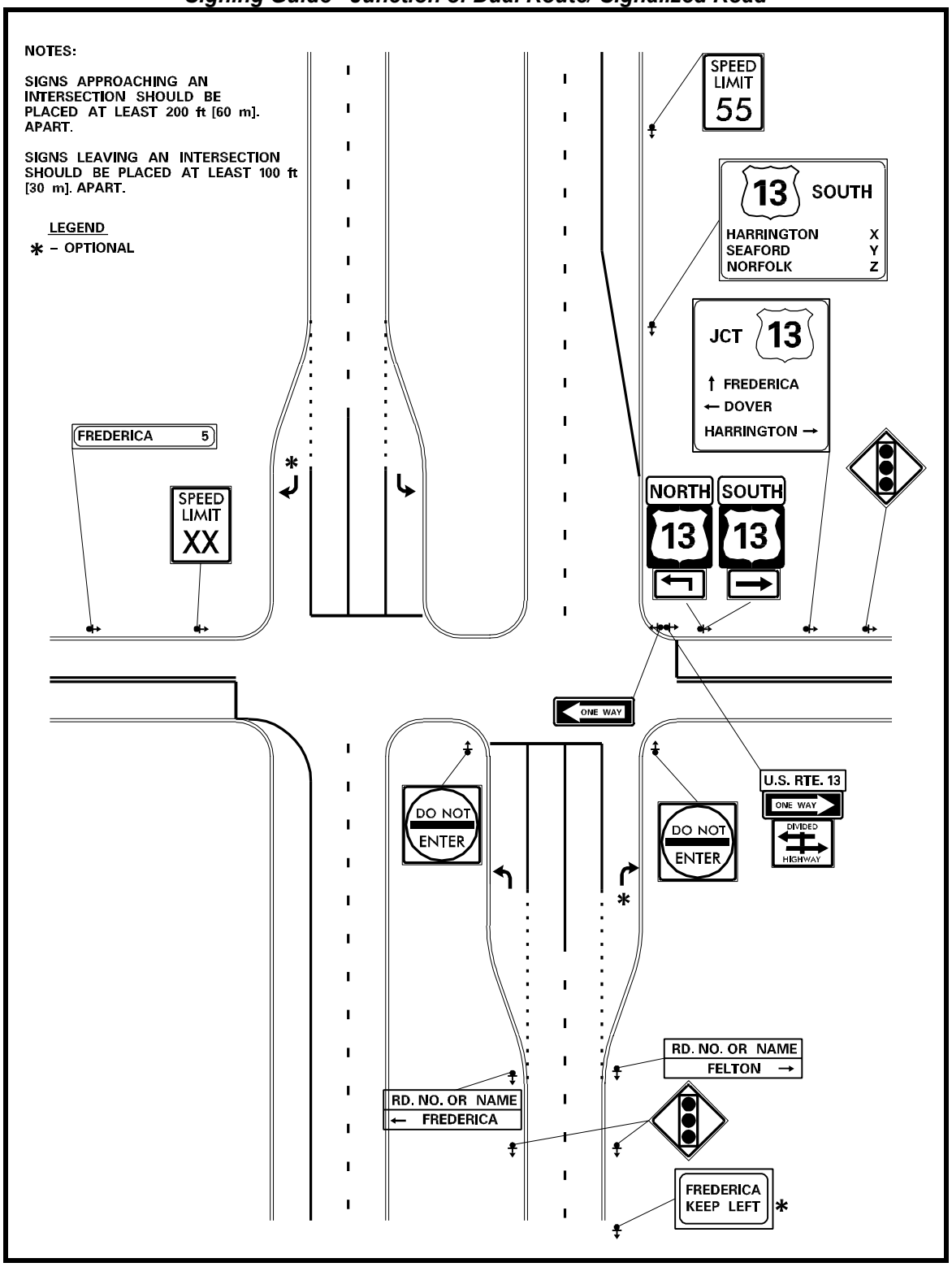


Figure 8-2
Signing Guide - Junction of Dual Route/Major to Minor Stop Road

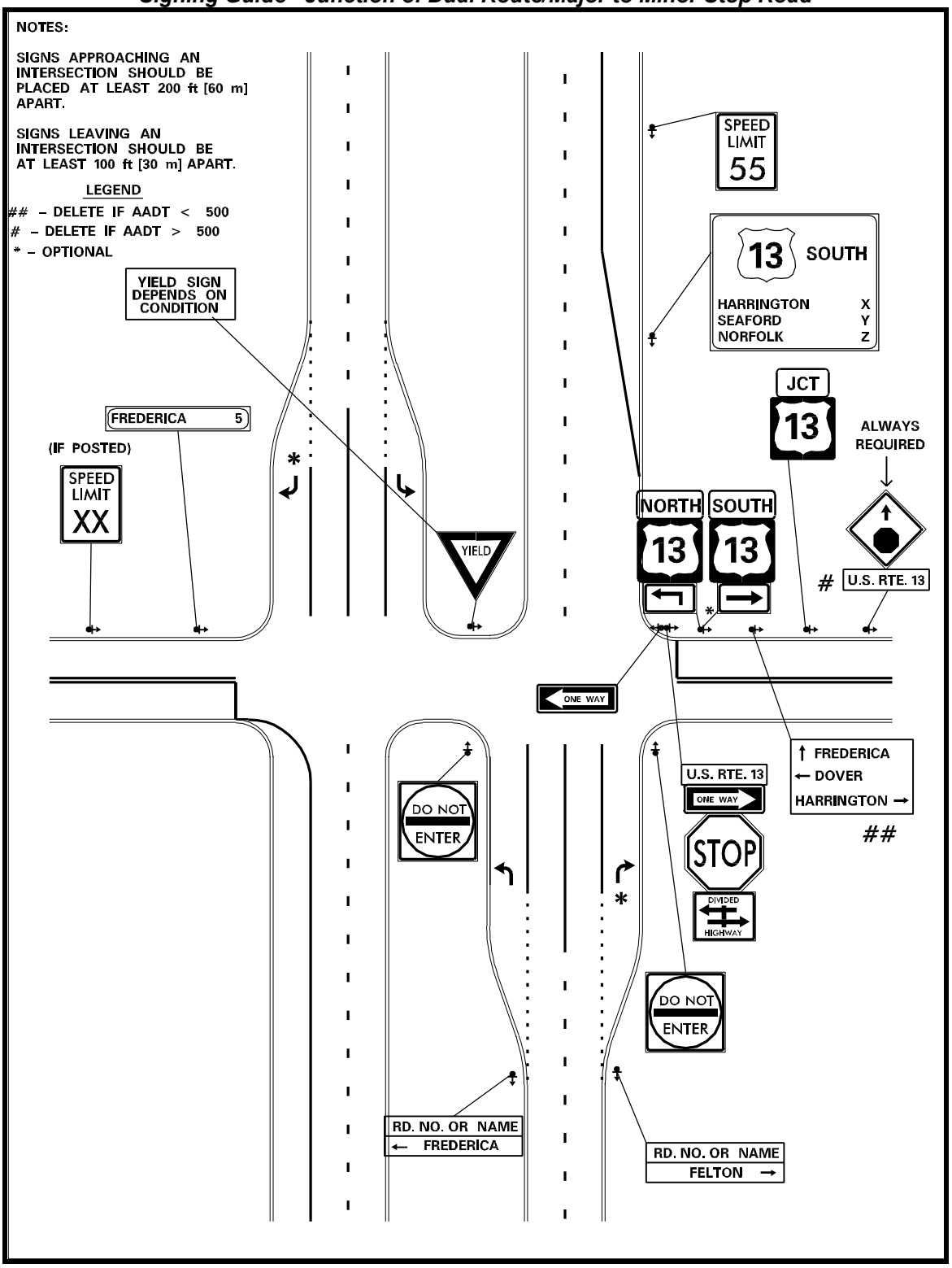


Figure 8-3
Signing Guide - Junction of Two Routes/Two-Way Signalized Road

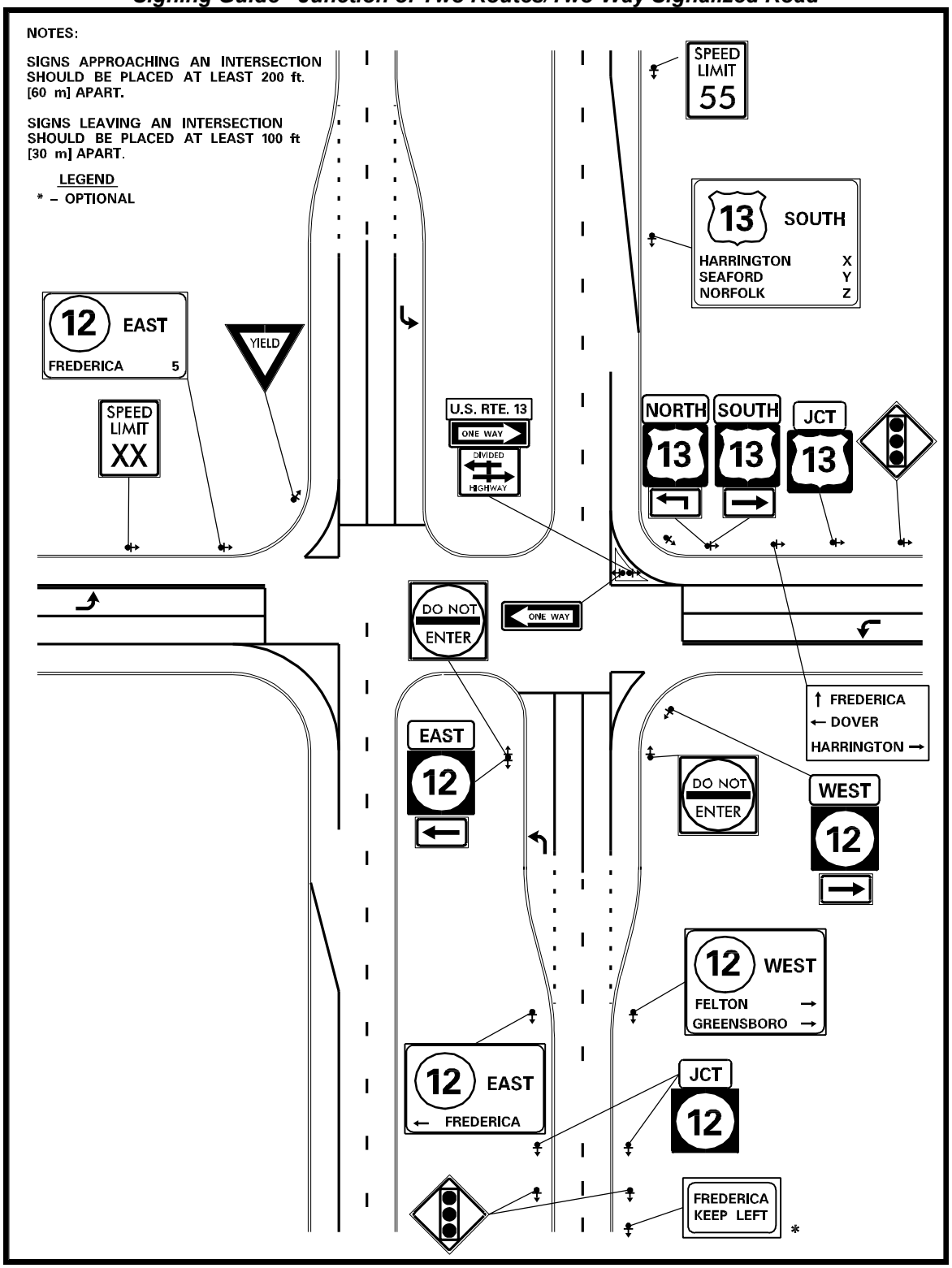


Figure 8-4
Signing Guide - Crossovers

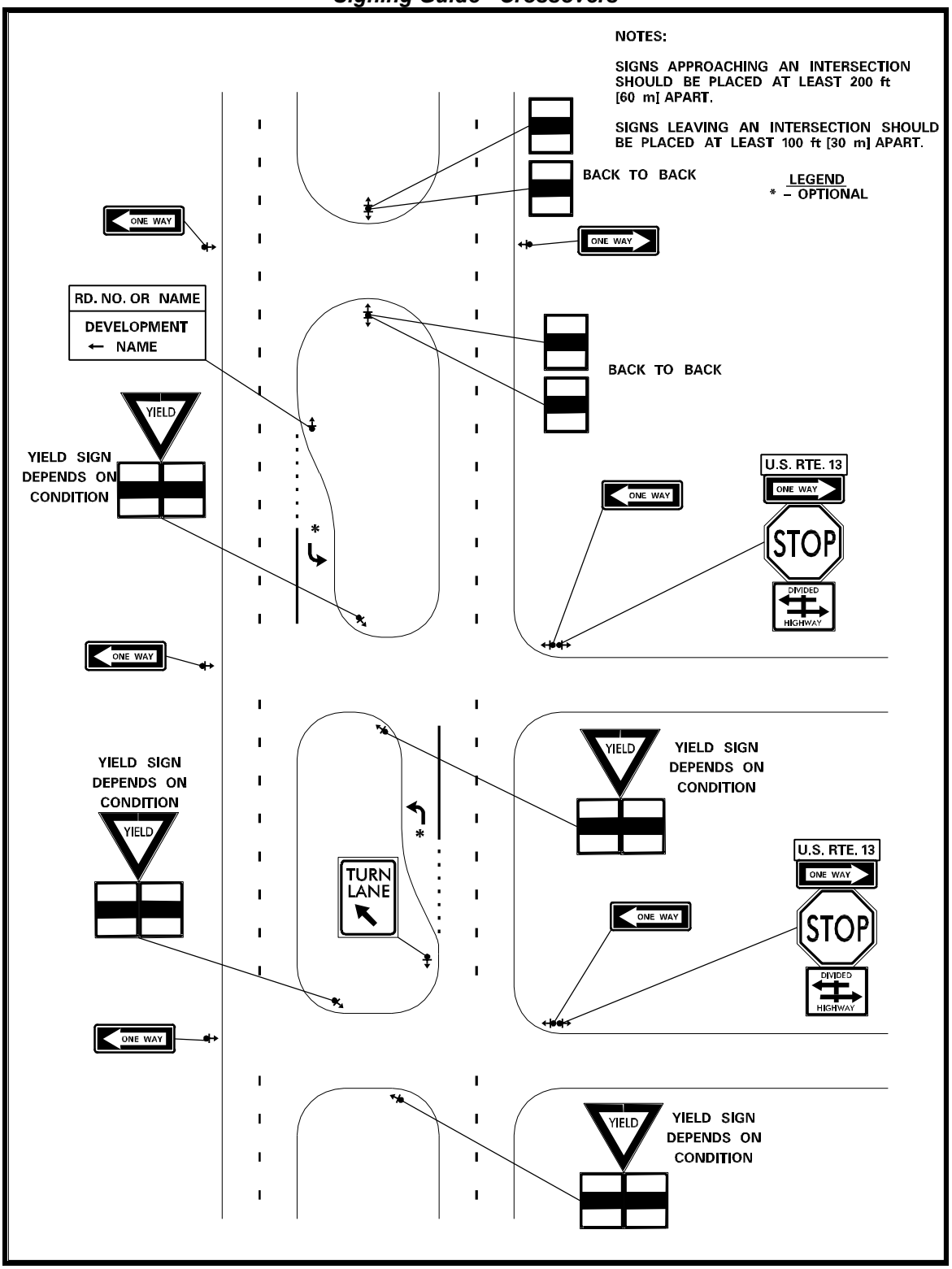


Figure 8-5
Typical Intersection Pavement Marking

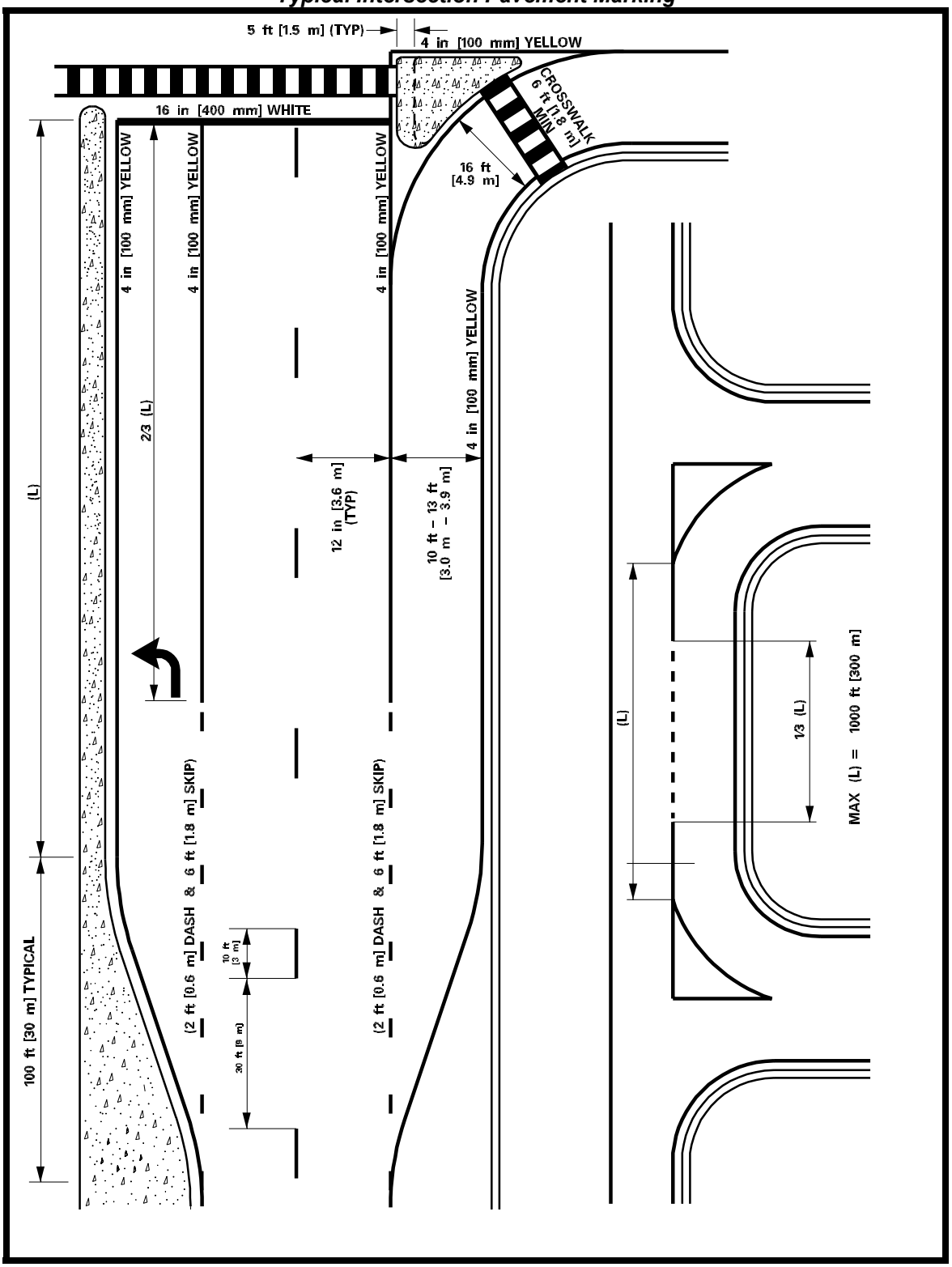


Figure 8-6
Typical Lane Reduction Transition Markings and Signing
Typical Multi-Lane, Two-Way Marking With Single Lane, Two-Way Left Turn Channelization

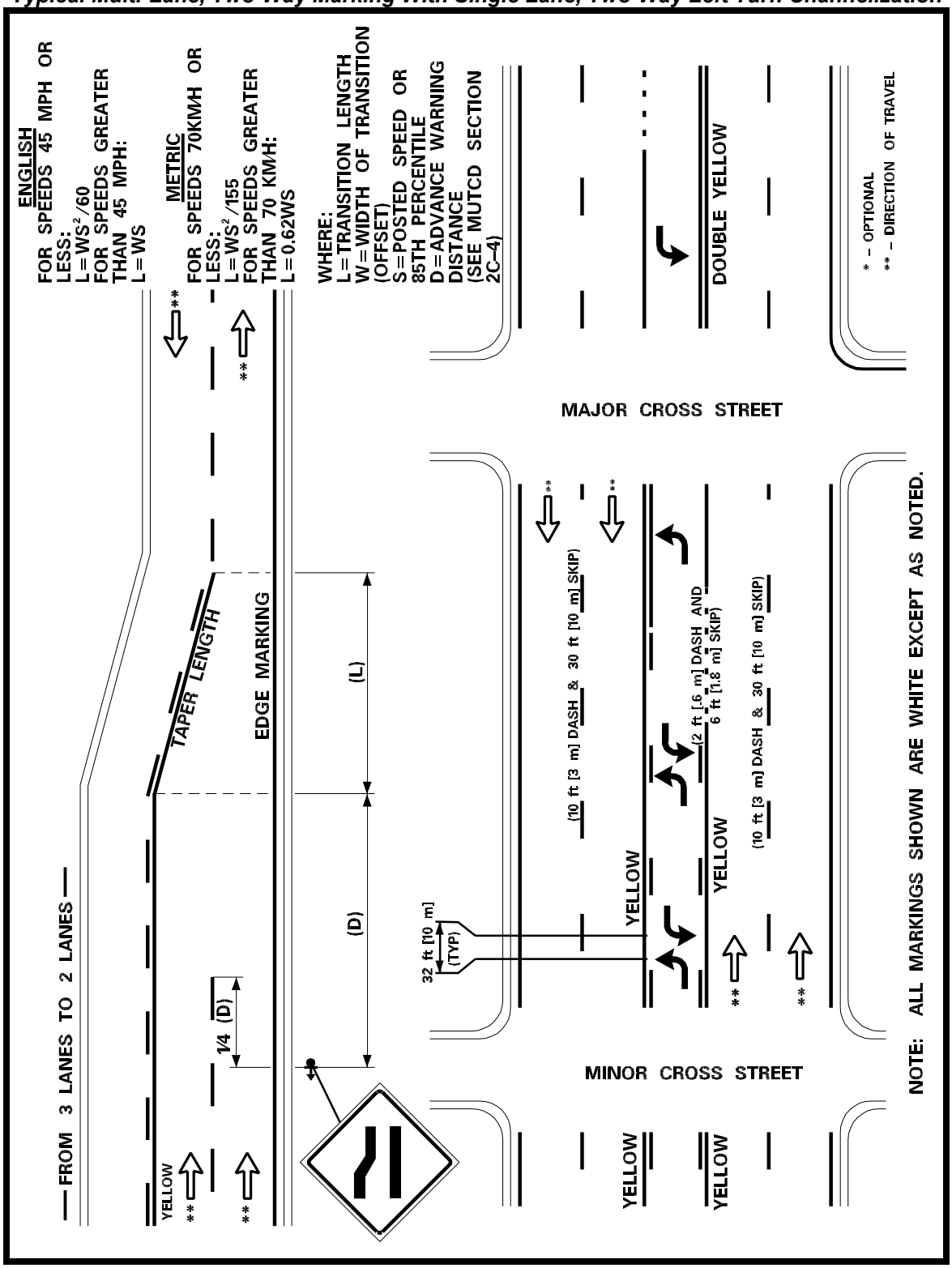


Figure 8-7
Typical Pavement Markings at Railroad-Highway Grade Crossing

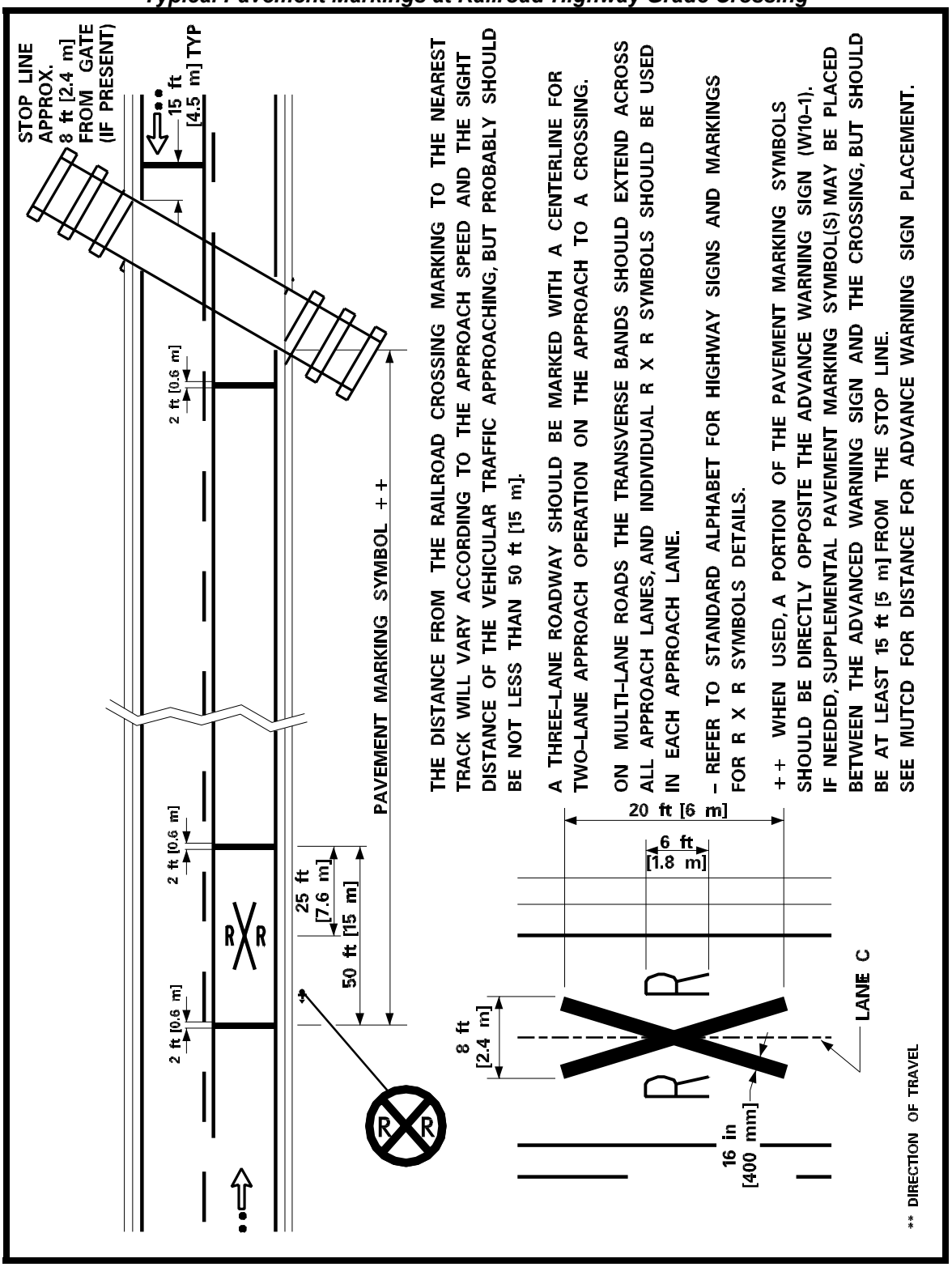


Figure 8-8
Typical One Way and Divided Highway Marking Applications

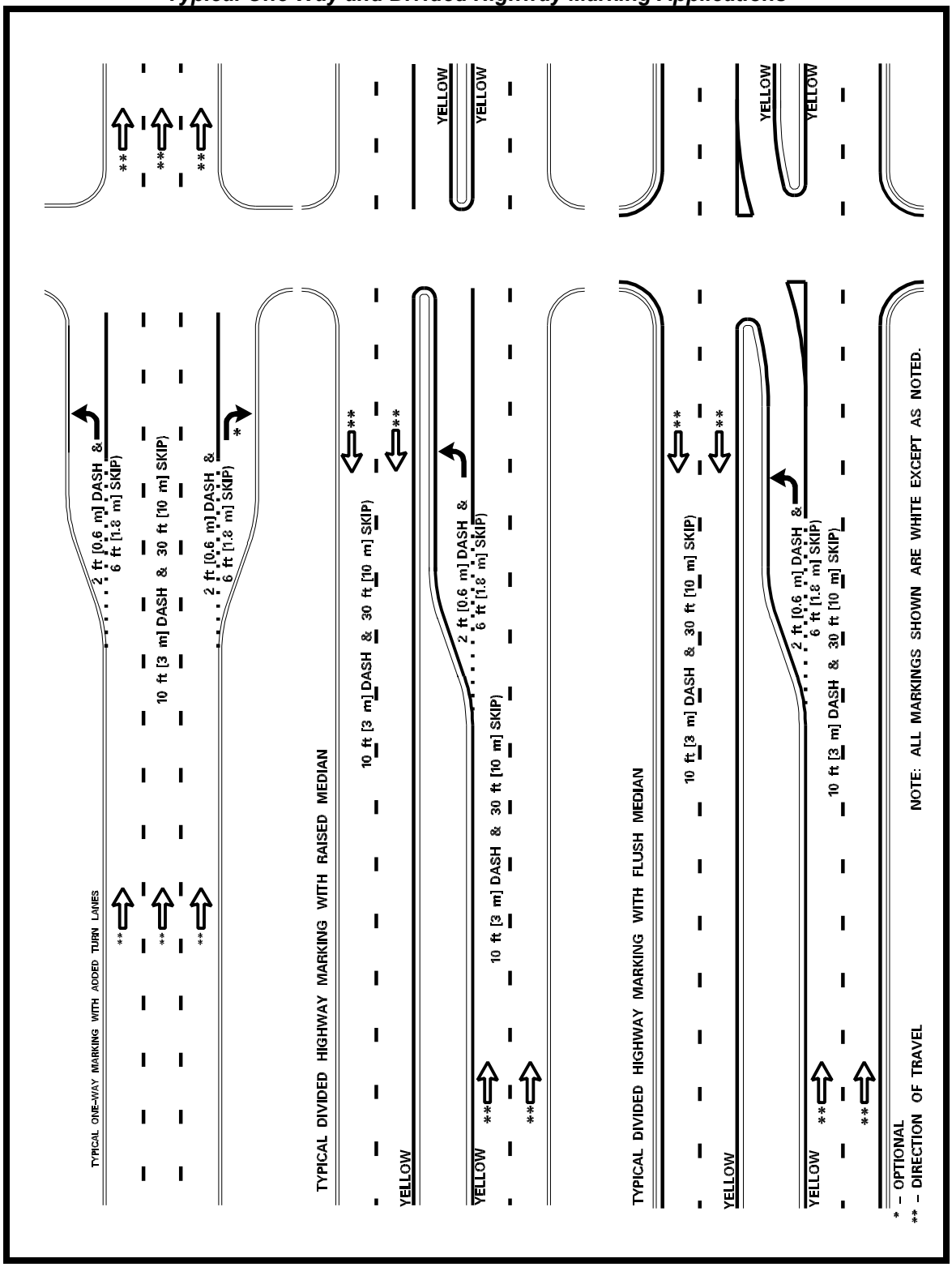


Figure 8-9
Typical Expressway/Freeway Acceleration and Deceleration Lane Pavement Markings

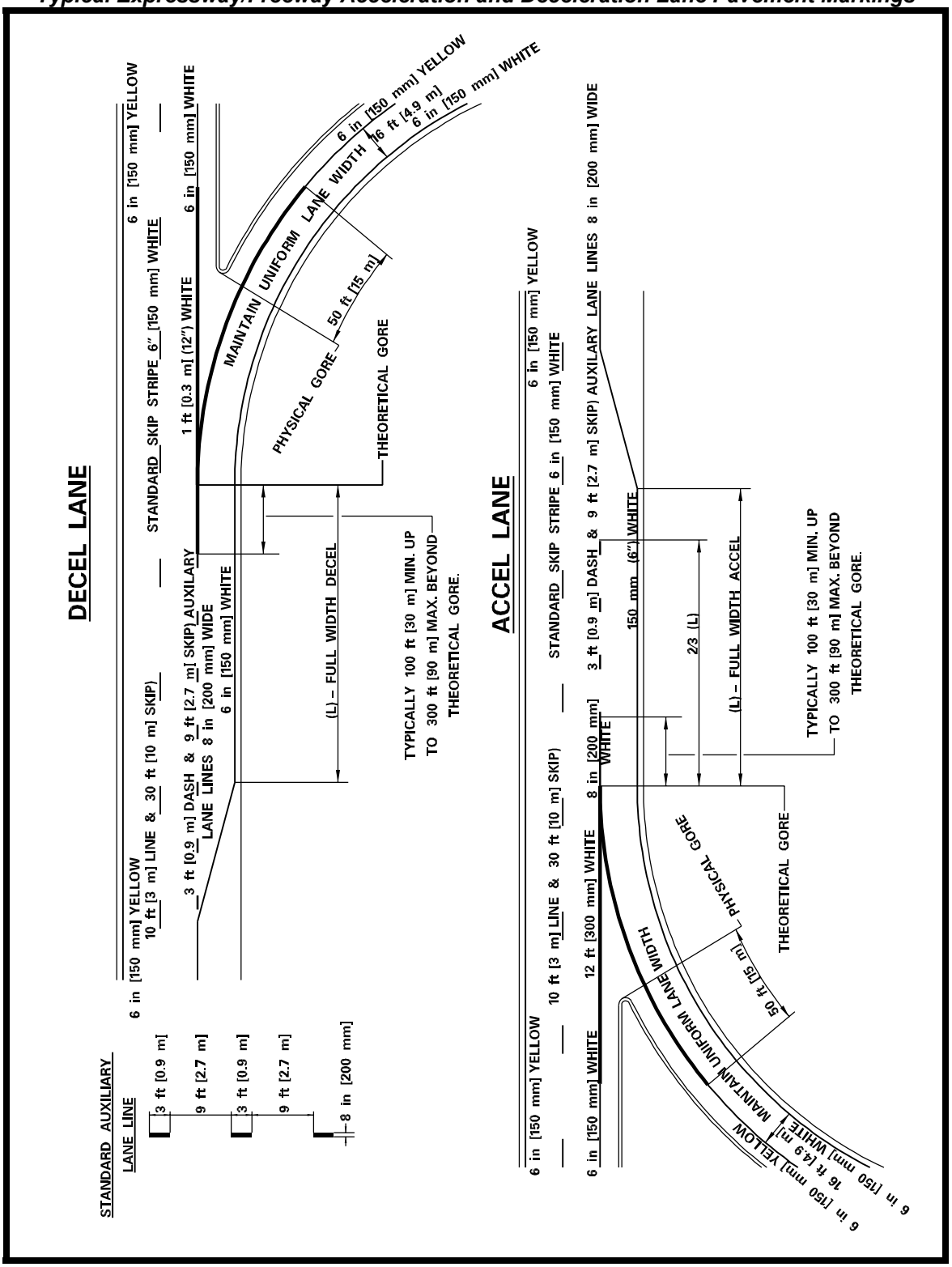


Figure 8-10
Typical Entrance and Exit Ramp Pavement Markings

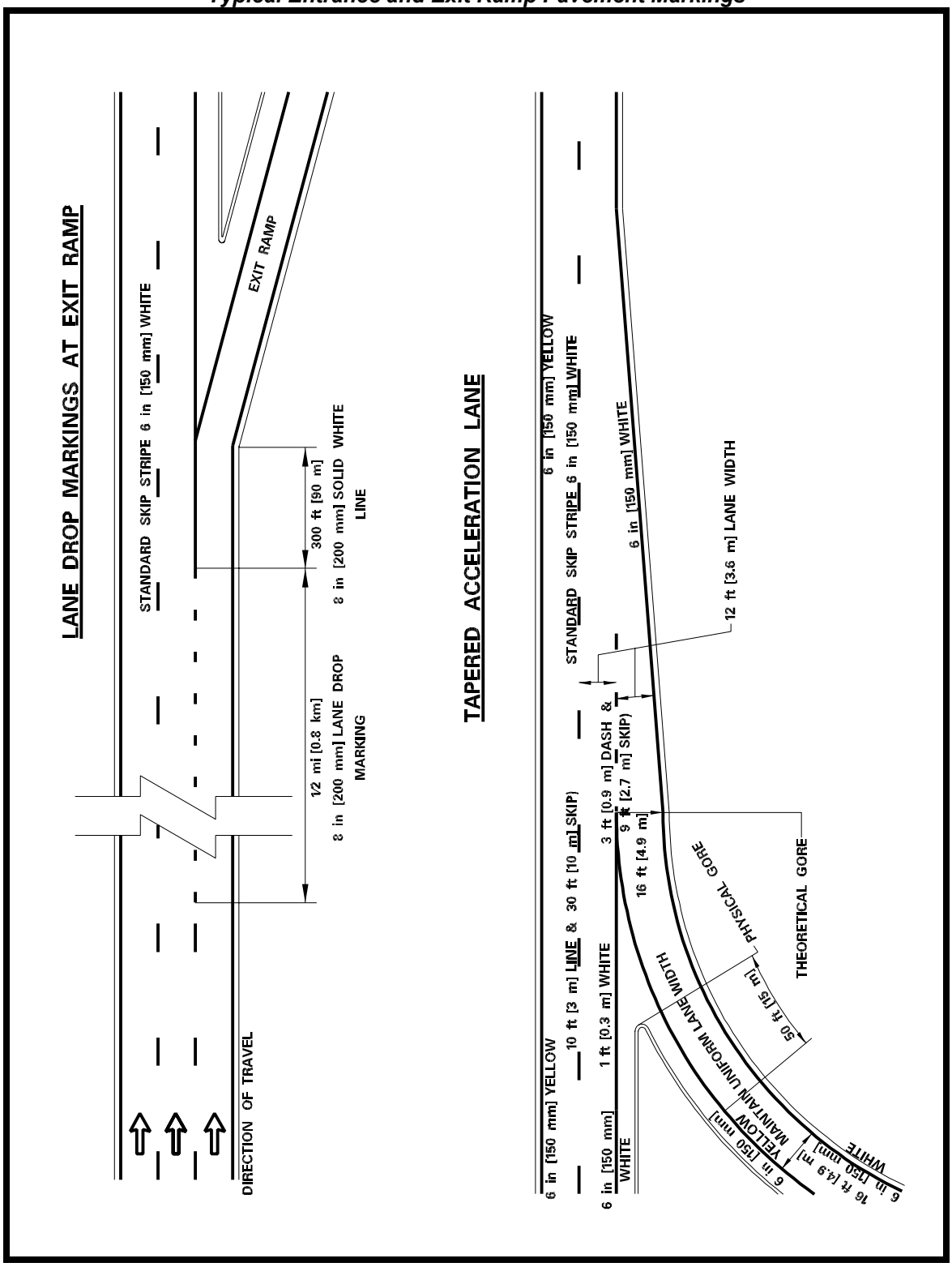


Figure 8-11
Guidelines for Advance Placement of Warning Sign-US Customary

Posted or 85th per-centile Speed (mph)	Advance Placement Distance[1] (ft)						
	Condition A: High Judgment required[2]	Condition B: Stop condition[3]	Condition C: Deceleration to the listed advisory speed (mph) for the condition[4]				
			10	20	30	40	50
20	175	N/A[5]	N/A[5]				
25	250	N/A[5]	100	N/A[5]			
30	325	100	150	100			
35	400	150	200	175	N/A[5]		
40	475	225	275	250	175		
45	550	300	350	300	250	N/A[5]	
50	625	375	425	400	325	225	
55	700	450	500	475	400	300	N/A[5]
60	775	550	575	550	500	400	300
65	850	650	650	625	575	500	375

Figure 8-12
Guidelines for Advance Placement of Warning Signs-[Metric]

Posted or 85th per-centile Speed [km/h]	Advance Placement Distance[1] [m]									
	Condition A: high judgment required[2]	Condition B: stop condition[3]	Condition C: Deceleration to the listed advisory speed [km/h] for the condition[4]							
			10	20	30	40	50	60	70	80
30	50	N/A[5]	N/A[5]	N/A[5]						
40	70	N/A[5]	25	N/A[5]	N/A[5]					
50	100	30	50	40	35	N/A[5]				
60	130	60	80	70	60	50	40			
70	160	80	100	100	90	80	70	35		
80	180	110	130	120	120	110	100	70	50	
90	210	140	160	150	150	140	130	100	80	60
100	240	180	190	180	180	170	160	130	110	90
110	270	220	220	210	210	200	190	160	150	130
120	300	260	240	240	230	230	220	190	180	160
130	320	300	270	270	270	260	250	220	210	190

Notes for Figures 8-11 and 8-12:

[1] The distances are adjusted for a sign legibility distance of 175 ft [50 m] which is the appropriate legibility distance for a 5 in [125 mm] series D word legend. The distances may be adjusted by deducting another 100-ft [30 m] if symbol signs are used. Adjustment may be made for grades if appropriate.

[2] Typical conditions are locations where the road user must use extra time to adjust speed and change lanes in heavy traffic because of a complex driving situation. Typical signs are Merge, Right Lane Ends etc. The distances are determined by providing the driver a PIEV time of 6.7 to 10.0 seconds plus 4.5 seconds for vehicle maneuvers minus the legibility distance of 175 ft [50 m] for the appropriate sign.

[3] Typical condition is the warning of a potential stop situation. Typical signs are Stop Ahead, Yield Ahead of Signal Ahead. The distances are based on the 1990 AASHTO Policy for stopping sight distance (page 119) providing a PIEV time of 2.5 seconds, friction factor of 0.30 to 0.40, minus the sign legibility distance of 175 ft [50 m].

[4] Typical conditions are locations where the road user must decrease speed to maneuver through the warned condition. Typical signs are Turn, Curve, or Cross Road. The distance is determined by providing a 1.6 second PIEV time (1990 AASHTO, page 119}, a vehicle deceleration rate of 10 ft/sec^2 [3 m/sec^2], minus the sign legibility distance of 175 ft [50 m].

[5] No suggested minimum distances are provided for these speeds, as placement location depends on site conditions and other signing to provide an adequate advance warning for the driver.

Chapter Nine

PAVEMENT SELECTION

This chapter discusses the general criteria, procedures and responsibilities for structural design of highway pavements. In addition there is information given on the various types of pavements, pavement rehabilitation techniques, and other factors that enter into pavement design and final pavement selection.

For the purposes of uniform and consistent design practices, the Department has adopted the criteria and procedures as set forth in the *AASHTO Guide for Design of Pavement Structures 1993*. This chapter briefly reviews the concepts and criteria used. Reference should be made to the AASHTO Guide for more detailed information on design procedures, if needed.

The design procedures include the determination of total thickness of the pavement structure as well as the thickness of the individual components using input parameters specified in the DARWIN 3.01 computer program. Provision is made for the design of equivalent alternate pavement sections, with the selection primarily a function of availability of materials, comparative costs, constructibility, and availability to traffic.

The discussion and explanatory material presented here are intended to give the designer a general understanding of pavement design concepts, alternative paving treatments, and a basic understanding of the information contained in a soil survey and pavement design report.

9.1 DESIGN RESPONSIBILITY

The design of pavement structures has some jointly shared responsibilities between the Materials and Research (M&R) Section and the responsible Project Development section. However, the primary responsibility for structural design and final recommended pavement sections is that of the M&R Section.

9.1.1 SOIL SURVEY/PAVEMENT EVALUATION REQUEST

There are several elements in the design process that need to be accomplished before a pavement section recommendation can be requested from the M&R Section. The project handoff package will describe the project scope. If the intent is to construct new pavement or replace the existing pavement then a soil survey, pavement design and pavement type recommendation will have to be requested.

The following should be available when requesting borings for a soil survey from DelDOT's Geotechnical Engineer and/or corings of the existing pavement from DelDOT's Pavement Design Engineer:

- preliminary surveys,

- survey plans with the location of the investigation, including the road name and state maintenance road number, and any predetermined locations marked,
- existing right-of-way verified,
- Right-of-entry to trespass, if needed,
- Purpose of the investigation in order to determine what pertinent information is required from the cores or the borings.
- If a boring is required, the depth of the boring. If the designer is unsure of depth to sample, contact the Geotechnical Engineer for guidance.

Coring and boring requests are typically processed within 30 days. If a coring and boring request is received simultaneously, they will be processed concurrently. If the boring request is received after the coring request has been completed, it will be treated as a different request and will be processed within 30 days of receipt.

For a pavement design, the following information is required with the request:

- Design year traffic data (AADT)
- Design year truck percentages
- Weight group pattern of trucks
- Directional split
- Existing pavement structure (if applicable)
- Subsurface investigation report (if applicable)
- Description of any existing pavement deterioration

Copies of any existing corings, borings, or subsurface condition reports should be provided to the Pavement Design Engineer when sending the pavement design request. The Pavement Design Engineer may elect to have additional corings, borings, or other investigations performed to ensure the existing conditions are known and considered when performing the pavement design. All information, including the design, will be forwarded to the designer.

For projects designed by a consultant, the design consultants must:

- Develop the subsurface investigation plan to have drilling and inspection services provided under their direction.
- Notify the DelDOT Project Manager of the scope of the subsurface investigation plan and provide a boring plan sheet(s) to the Department. The DelDOT Project Manager will forward the boring plan to the Materials & Research (M&R) Geotechnical Engineer
- Specify to M&R what soil testing is requested.
- Inform the Geotechnical Engineer at least 24 hours in advance when drilling and soil sampling is to begin and provide the name and phone number of the consultant who is to receive the M&R soil test information.
- Arrange to have the soil samples and copies of the field logs brought to the M&R lab within 24 hours after obtaining samples.

The Geotechnical Engineer shall:

- Perform, or arrange for, all soil testing required for the project as requested by the design consultant.
- Furnish completed boring logs and laboratory test summaries to the design consultant with a copy to the DelDOT Project Manager.
- Provide the DelDOT Project Manager regular weekly updates on the progress of the drilling/testing programs.
- Inform the DelDOT Project Manager when problems arise and when the testing programs are completed.

Many projects are initiated with the intent of extending the service life of the existing pavement section. Rather than rebuilding the entire pavement, improvements are made in the riding quality, skid resistance and limited structural improvements through various rehabilitation methods. For projects of this type a pavement

evaluation is requested. A pavement evaluation includes an extensive pavement survey noting pavement conditions, drainage and major distress problems. The initial evaluation will determine which rehabilitation method(s) should be considered. Based upon the alternatives being considered, it may be necessary to conduct detailed measuring and testing, including coring and sampling, and accurately determining structural clearances to any overhead structures. The final report received by the designer will contain most if not all this collected field data and will include the recommended rehabilitation method. Much of the field data collected can be a very valuable tool in designing the project.

9.1.2 SOIL AND PAVEMENT DESIGN REPORT

The M&R Section performs the soil sampling and pavement coring. This field data is tested in the laboratory and the results documented in the form of a Soil Survey. This report includes the soil profile data summary showing the sample location, depth, soil profile description, soil classification and any remarks. The soil samples are tested and a summary of this analysis is included in the report.

Based upon the subgrade soil characteristics, the report may provide recommendations for muck excavation, limits of special fill, need for underdrains, grade adjustments, embankment construction, the use of geotextiles or other special construction considerations.

An important part of the report is the pavement design portion. Using the AASHTO Guide, the soil survey data, and past experience, the recommended pavement type and thickness by components is included in the report. Pavement sections for shoulders and turn lanes are also included.

The report may provide an alternate design including at least one rigid pavement and one or more flexible alternatives. Project Development and M&R will meet and mutually agree on which pavement to use on the project. If necessary, the M&R Section may perform an economic and life cycle cost analysis or other studies in making the final determination and recommendation for pavement type and section. For many projects, only one pavement design recommendation is made.

The Soil and Pavement Design Report should be evaluated thoroughly and considered as the design progresses. In the design process significant changes in the proposed profile or alignment can dramatically affect the pavement design and need to be discussed with M&R, perhaps even before such decisions are finalized. In addition the proposed construction sequencing, methods of construction and maintenance of traffic plan can dramatically affect whether the recommended pavement materials can be placed in a timely manner and meet the in-place performance quality necessary for the intended service life.

9.1.3 PAVEMENT SELECTION

As mentioned in Section 9.1.2, there may be several choices of structurally equivalent pavements. The choice of pavement, particularly on reconstruction and new construction is a major decision and needs to be approved prior to proceeding with the design.

The factors considered in making the final decision on pavement type are quite varied from empirical to subjective and may include several of all of the following:

- Project scope—as initiated,
- Cost to construct or rehabilitate the pavement,
- Available project funding,
- Construction sequencing as it relates to controlling through and local traffic,
- Construction sequencing as it relates to serving commercial areas,

- Construction sequencing as it relates to maintaining quality control of pavement construction,
- Availability of work areas for the paving equipment,
- Projected future traffic control and other costs to perform maintenance, restoration or rehabilitation
- Minimum desirable service life,
- Performance of similar pavements under similar soil conditions and traffic loadings,
- Geotechnical design problems created by the depth of pavement structure, which could increase drainage costs,
- Effect on underground utilities, and
- Effect on existing vertical clearances.

9.2 PAVEMENT TERMINOLOGY

The pavement section is treated as a structural element consisting of several different materials of varying depths and supporting strengths. Knowledge of the following definitions and the terminology as shown on Figure 9-1 is needed to understand the pavement design and rehabilitation concepts in this chapter.

- Base Course—the layer or layers of specified or select material of designed thickness placed on a subgrade to support a surface course
- Bituminous Concrete—A designed combination of dense graded mineral aggregate filler and bituminous cement mixed in a central plant, laid and compacted while hot.
- Bonded Overlay—An overlay of concrete placed over a Portland cement concrete pavement.
- Flexible Pavement—A pavement structure of bituminous concrete that distributes loads to the subgrade and depends on a firm continuous subgrade, aggregate interlock, particle friction, and cohesion for stability.
- Grinding—Patterns cut into a concrete pavement with closely spaced diamond blades to restore pavement smoothness and skid resistance.
- Grooving—Patterns cut in asphalt or concrete pavements to promote surface drainage to reduce wet weather hydroplaning.
- Micro-Surfacing—A polymer modified cold-mix paving system consisting of a mixture of dense-graded aggregate, asphalt emulsion, water, and mineral fillers.
- Modulus of Subgrade Reaction (k)—A value used in rigid pavement design that is the ratio of the load in pounds per square inch [in kilograms per square mm] on a loaded area of the subgrade or subbase divided by the deflection in inches [mm] of the subgrade soil or subbase, psi/in [kPa]. Typically, k is adjusted for potential loss of support due to subbase erosion.
- Open Graded Mix—A special mix, containing aggregate that resists polishing, placed on the surface course to drain surface water, improve skid resistance, and reduce hydroplaning.
- Rigid Pavement—A pavement structure that distributes loads to the subgrade, having as one course a Portland cement concrete slab of relatively high bending resistance.
- Pavement Milling—The use of carbide cutting teeth mounted on a rotary drum to chip off as much 3 to 4 inches [75 to 100 mm] of asphalt concrete surface.
- Pavement Structure—A combination of subbase, base course and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed
- Recycling—Salvaging and processing portions of existing pavement for use in construction of new pavement structures.
- Resilient Modulus (M_R)—A measure of the essential properties of untreated subgrade soils through determination of dynamic elastic modulus under conditions that represent a

reasonable simulation of the physical conditions and stress states of subgrade materials beneath flexible pavements subjected to moving loads.

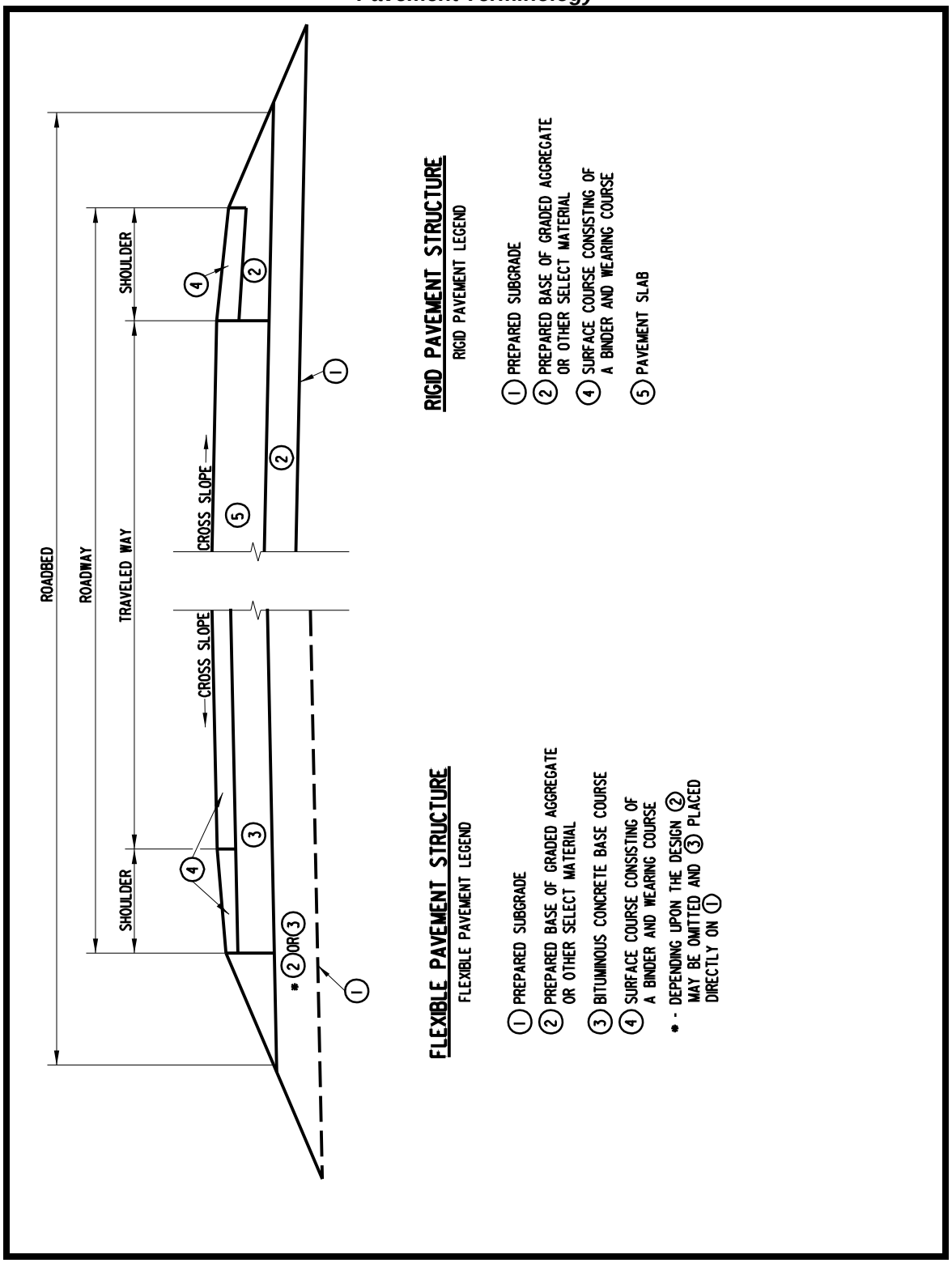
- Roadbed Material—The material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.
- Roadbed—The graded portion of a highway between top and side slopes prepared as a foundation for the pavement structure and shoulders.
- Select Material—A suitable native material obtained from a specified source such as a particular roadway cut or borrow area having specified characteristics to be used for a specific purpose.
- Subbase—The layer or layers of specified or select material of designed thickness placed on a subgrade to support a base course (or in the case of rigid pavements, the Portland cement concrete slab).
- Subgrade—The top surface of a roadbed soil upon which the pavement structure and shoulders are constructed.
- Superpave—An asphalt pavement with a laboratory design mix that provides superior performance.
- Surface Course—One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called "wearing course."
- Ultra-Thin-Whitetopping—An overlay of concrete less than 4 inches [100 mm] thick placed over an asphalt base, usually 2 to 3.5 inches [50 to 90 mm].
- Whitetopping—A concrete overlay of an asphalt pavement of 4 inches [100 mm] or more.

9.3 PAVEMENT DESIGN FACTORS

Pavement design methodology for both new pavements or rehabilitation of existing pavements consider several, if not all, of the following factors:

- Pavement design life,
- Pavement performance,
- Traffic volume and vehicle class,
- Roadbed soil,
- Materials of construction,
- Temperature changes,
- Drainage,
- Reliability,
- Life-cycle costs, and
- Shoulder design

Figure 9-1
Pavement Terminology



Of these factors, the most influential factors in determining a pavement's required structural strength are the characteristics of the underlying roadbed material, the projected traffic volumes and the percentage and weight of vehicles in the traffic mix using the facility over the expected design life of the pavement. The following sections briefly discuss the factors considered.

9.3.1 PAVEMENT DESIGN LIFE

Each pavement design has a selected design life. Roadway cross section elements and other components of the project such as the pavement structure are expected to remain structurally sound for a designated period of time defined as design life. Although the roadway cross section may become operationally obsolete or the pavement distressed and in need of restoration or rehabilitation, they have not reached the end of their design life but rather have reach the end of a condition defined as service life. Not until they need complete replacement are they considered to have reached the end of their design life.

9.3.2 PAVEMENT PERFORMANCE

The goal of a pavement design is to produce a pavement that when placed will perform functionally and structurally while maintaining its safety characteristics for at least the selected service life.

Functional performance of a pavement identifies how well a pavement will serve the user. The characteristics identified are riding comfort and ride quality. This concept is called serviceabilty-perfomance and provides a means to measure functional performance. In the pavement design procedure, this factor is expressed in terms of the present serviceability index (PSI). PSI is a measurement of roughness and distress of a pavement during the service life of a pavement. Therefore, a reliable method of measuring roughness and maintaining and updating historical performance data is an integral part of pavement design. The major factors influencing the loss of serviceability are traffic, age, and environment.

The structural performance of a pavement relates to its physical condition; including occurrence of cracking, faulting, raveling, or other conditions which would adversely affect the load-carrying capability of the pavement or would require maintenance.

A pavement's safety performance primarily relates to its ability to provide adequate skid resistance during its service-life but also can be affected by its ability to maintain a smooth and rut free surface. Age, traffic, physical properties of materials used to construct the pavement and environmental conditions influence a pavement's safety performance.

9.3.3 TRAFFIC

Traffic volumes using a facility, in particular the number and weight class of trucks, is a major factor in determining how strong a pavement structure must be. In the design procedure, traffic data is reduced into axle loads, axle configuration, and number of applications of these loads. The result is a design number representing the damage done to the pavement caused by the effect a single axle carrying a load on the pavement over its design-life. For the design calculations, traffic data is converted into 18-kip [80 kN] equivalent single axle loads or ESAL's. Since it is one of the more important design considerations, accurate traffic data will ensure that a pavement's design life will be attained and the pavement sections selected will not be over or under designed.

9.3.4 ROADBED SOIL

The pavement structure rests on a graded and compacted roadbed either of suitable natural material or on specified imported material. The roadbed soil has measurable material characteristics that are used in the pavement design. This measurement is defined as a soil's resilient modulus (M_R) and is a measure of the elastic

property of soil. The resilient modulus is used directly for designing flexible pavements but must be converted to a modulus of subgrade reaction (k-value) for the design of rigid pavements or composite pavements. The resilient modulus is also a soil property used in analyzing multilayered material systems for predicting roughness, cracking, faulting, rutting, and other potential distresses.

The value of a roadbed's resilient modulus is dependent on how well the roadbed soil is placed in conformance with the specified compaction parameters. For most projects, the material is to be placed in accordance with the *Standard Specifications* with no special treatment. However, the Soil and Pavement Design Report may indicate that there is anticipated difficulty with the existing roadbed soil meeting the design M_R value. For soils that are excessively expansive the report may recommend these soils be covered by select material sufficiently deep enough to reduce or eliminate the expansive affect of the natural material. Other solutions may include the adding of an admixture to reduce the water content or the use of a geotextile.

One of the more difficult soils encountered on projects are those having a large organic content. These materials are extremely compressible, unstable and frequently non-uniform in properties and depth. These soils are the most complicated and expensive to deal with in order to provide an adequate roadbed. Small, shallow or localized deposits are most often excavated and replaced with suitable material. Deeper and more expansive areas involve more detailed geotechnical design, more complicated construction techniques and costs. Treatments other than complete removal are more time dependent allowing for the slow consolidation and removal of excess moisture. Methods available include surcharge embankments for preconsolidation of the underlying material usually involving sand drains which allow the water to rise to the surface and be removed. The M&R Section is responsible for identifying and designing the most economical method treating this type of problem area.

Underdrains (a system of perforated pipes to collect and transmit the water to an outfall site) are recommended for use on all roadway projects to adequately address drainage and removing water from the roadbed. If site conditions indicate that underdrains may not be required, contact the M&R Section to initiate further investigation.

The soil and pavement condition survey will normally identify roadbed drainage problem areas or soils highly susceptible to expansion or loss of strength with increase in water content. When either of these conditions exist, the M&R Section may recommend additional work and/or materials to address the existing conditions.

Another type of material encountered in constructing roadbeds is classified as cohesionless (sandy) soil and is much more difficult for the contractor to place and compact; it is readily displaced under the load of the equipment. To stabilize this type of soil it may be necessary to blend granular material or add a suitable admixture. Wet clay soils may also be encountered. Because of high moisture content this type of soil is unstable and cannot be compacted. Long periods of dry weather and exposure to the air are required to reduce the water content. To reduce the time necessary to reuse these materials, the recommendation may be to add a suitable admixture that hastens drying or cover the area with a more suitable select material. Removing the material and replacing it with suitable material allowing construction to continue is an option. The material may be used in areas that don't require compaction or moved to an available site for air-drying and reuse at a later time.

9.3.5 PAVING MATERIALS

Depending upon materials that comprise a pavement, the pavement structure is identified as either a flexible or rigid pavement. Combining these two types of paving materials in a pavement structure as a subbase or surface course results in a composite pavement.

9.3.5.1 FLEXIBLE PAVEMENTS

Flexible pavements consist of a prepared roadbed with a subbase of graded aggregate or bituminous concrete and a bituminous concrete base with a surface course. Properly preparing a uniform roadbed compacted to the prescribed density is especially important for providing the design support value necessary for flexible pavement to perform as designed.

The subbase course usually consists of a compacted layer of granular material or an untreated graded aggregate. If additional support strength is needed or the roadbed soil is questionable either of these materials may be treated with an admixture. In some instances, the subbase may be recommended to be a freely draining-highly permeable material to provide a means for water to migrate from under the pavement structure to the side slopes or to an underdrain system. The subbase also prevents intrusion of fine-grained roadbed soils into the base course, minimizes the damaging affects of frost action, and provides a working platform for construction equipment.

The base course is a specified depth of bituminous concrete that is primarily designed to provide the structural strength needed to support and distribute the projected traffic loads.

The surface course is a bituminous concrete mixture placed as the upper course and is usually constructed on a base course. The surface course provides some structural strength. However, the major functions of the surface course are to provide a smooth riding surface that resists distress, minimizes the amount of water that may penetrate the lower more porous layers, provides and maintains its skid resistance for selected service life. To meet these requirements, the surface course mix must have the optimum gradation of aggregate and percent of bituminous binder to prevent raveling, provide durability, resist fracture, and remain stable under traffic use and adverse climate changes.

9.3.5.2 RIGID PAVEMENTS

Rigid pavements consist of a prepared roadbed, a layer of subbase material and a Portland cement concrete pavement slab. The subbase may be either stabilized or unstablized. Although concrete pavements can span failed subgrade and subbase areas easier than flexible pavements, they are still susceptible to damage and failure from excessive moisture in the support structure.

The subbase of a rigid pavement structure consists of one or more compacted layers of granular or stabilized material placed between the roadbed and the rigid slab. The subbase functions to:

- Provide uniform, stable, and permanent support,
- Increase the modulus of subgrade reaction,
- Minimize the damaging effects of frost action,
- Prevent pumping of fine-grained soils at joints, cracks, and edges of the rigid slabs, and
- Provide a working platform for construction equipment.

The pavement slab is composed of Portland cement concrete, longitudinal tie steel, load transfer devices between slabs, and joint sealing materials.

Joint sealing is a critical element in the long-term performance of a rigid pavement. Proper joint sealing prevents infiltration of water under the slab that reduces the support strength of the subgrade and reduces pumping action between slabs caused by the transfer of moving traffic loads between joints. The vertical movement of the slabs eventually erodes the subgrade material through any unsealed joints allowing a slab to break or settle.

There are two types of joint sealants in use, liquid sealants and preformed elastomeric seals. Liquid sealants include a wide variety of materials including hot-poured rubber, elastomeric compounds, and polymers. The materials are

placed in the joint in a liquid form and allowed to set. Preformed seals are extruded neoprene seals having internal webs that exert an outward force against the joint face. The size is determined by the amount of joint movement anticipated.

Transverse joints are needed in a rigid slab to form cracks at desired locations that can be controlled and sealed. Joints can be keyed, butted or tied. Transverse joints are sawed or formed to a depth of one quarter to one third the slab thickness. Timing the saw cutting operation to the curing of the concrete is critical.

Load transfer devices are used between slabs and usually consist of smooth round steel dowels. The dowels should distribute the load stresses without over stressing the concrete surrounding it, offer little restraint to longitudinal movement of the joint, be mechanically stable under the loads and load frequencies expected during the design period, and should be resistant to corrosion from moisture and road salts.

Materials used in the design of shoulders can be either flexible or rigid. Differences in material types and the subbase combined with unexpected wheel loads along the pavement edge can cause joint problems. With proper care and attention, this potential problem can be minimized. Solutions include widening the full depth pavement slab, using tied concrete shoulders, properly sealing the joint, and ensuring compatibility between subbase materials.

9.3.6 TEMPERATURE CHANGES

Temperature changes affect (1) the creep properties of asphalt concrete, (2) thermal-induce stresses in asphalt concrete, (3) contraction and expansion in Portland cement concrete, and (4) freezing and thawing of the roadbed soil. Temperature differences between the top and bottom of concrete slabs create uneven stresses on the slab and can be of concern. Temperature and a poorly drained pavement structure or subgrade, although normally not a concern in Delaware, can combine to create freeze-thaw cycles that rapidly deteriorate the pavement.

9.3.7 DRAINAGE

Keeping the pavement subgrade and soilbed dry is a major design consideration. Excessive moisture combined with increasing traffic and load applications will inevitably lead to premature pavement distress. Water can enter the pavement structure from many directions including a permeable pavement surface, unsealed or poorly sealed joints, surface cracks, high water tables, and even local springs. If water is trapped within the pavement structure, pavement performance will be affected through loss of support due to erosion of any granular material and loss of material strength.

Addressing the problem areas that allow water to enter the pavement structure is difficult to prevent and expensive to correct. In fact it is a shared responsibility. The pavement designer should recommend pavement designs that use dense non-permeable surface courses to reduce surface infiltration, specify underlying material courses that freely drain, and a pavement structure that is strong enough to resist the effects of the traffic loads and water. The roadway designer must ensure proper pavement crown, drainable grade lines, proper ditching and other adequate drainage systems to remove water quickly.

9.3.8 RELIABILITY

Reliability is a method to determine the probability that a particular pavement design will perform as desired during its design life. In selecting the appropriate level of reliability, the pavement designer relates the projected level of usage to the risks involved if a thinner pavement section is recommended.

For facilities with higher importance to the transportation system a pavement design considers the traffic disruption caused by closing or restricting traffic flow due to higher levels of distress, maintenance, and rehabilitation associated with an inadequate or a marginal initial

pavement structure. For lesser facilities, it may be perfectly acceptable and even economical to use a reduced thickness.

9.3.9 LIFE-CYCLE COSTS

Life-cycle costs are costs and benefits that accrue during a pavement's complete life cycle. These costs include the initial construction costs, maintenance costs, rehabilitation costs, resurfacing costs, maintenance of traffic costs, salvage or residual value and user costs.

As a part of the pavement selection decision, particularly when more than one pavement design or rehabilitation procedure is proposed, an economic comparison may be needed. Two methods are detailed in the ASSHTO Guide to determine life cycle cost comparisons, net present worth and equivalent uniform annual cost. Whichever method is used, it is essential that the analysis periods be of equal length.

9.3.10 SHOULDER DESIGN

The inclusion of a shoulder adjacent to the main pavement structure improves pavement performance. The AASHTO guide does not provide a design method for determining the shoulder section. The M&R Section recommends a shoulder section that is compatible with the proposed mainline pavement section, has good constructibility and has performed well in the past. Shoulders are usually designed to carry 10 percent of the projected Average Daily Traffic (ADT).

9.4 DESIGN FOR NEW CONSTRUCTION OR RECONSTRUCTION

Constructing a pavement section is one of the most costly items on new construction and reconstruction projects. In making a selection on the type of pavement to construct it is usually necessary to analyze alternate pavement types and combinations of various support materials. In performing the analyses, the AASHTO procedure requires the pavement designer to provide input in several categories:

- Design variables,
- Performance criteria,
- Material properties for structural design,
- Structural characteristics, and
- Reinforcement variables.

9.4.1 DESIGN VARIABLES

A set of design criteria is established for each project including the pavement's expected service life (performance period) and projected design life (replacement). The traffic projection for the cumulative expected 18-kip [80 kN] equivalent single axle loads (ESAL) during the analysis is determined. The level of reliability is selected and any detrimental environmental factors identified. There is a heirachy of application of these variables with the most important roadways assigned the most stringent and detrimental values to a pavement's performance.

9.4.2 PERFORMANCE CRITERIA

Performance of a pavement is measured by its serviceability to the expected users. The concept is to design a pavement, which at the end of the proposed performance period will still have a predefined minimum level of serviceability (PSI). The terminal level of serviceability is selected based on the lowest index the user will tolerate, or as defined in a pavement management strategy before rehabilitation, resurfacing or reconstruction becomes necessary. DelDOT typically uses a terminal PSI of 2.5 or 3.0, which varies based on functional classification and use.

9.4.3 MATERIAL PROPERTIES FOR STRUCTURAL DESIGN

For the design of flexible pavements, roadbed materials are characterized based on their effective elasticity or resilient modulus, M_R. Their

resilient modulus is determined for periods of stress and moisture conditions simulated for the primary moisture season.

For rigid pavement designs, an effective modulus of subgrade reaction (k-value) is developed. The k-value is directly proportional to roadbed soil resilient modulus. However, the effective design k-value is also dependent upon the effects of the characteristics of the subbase.

Another important input is the compressive strength of the materials composing the pavement structure. In the design of rigid pavements, the modulus of rupture (flexural strength) is determined using the mean value as tested at 28 days. This value is specified in the Standard Specifications and verified as being consistently met or exceeded through laboratory records.

In flexible pavement design, the layer coefficient method is used. Each structural layer is assigned a layer coefficient value which is the relationship between the Structural Number (SN) and thickness of the layer. (See Section 9.5.5.) It is an empirical means to represent the relative ability of the material to function as a structural component of the pavement.

9.4.4 PAVEMENT STRUCTURAL CHARACTERISTICS

Drainage is an important element in the ultimate performance of all pavements. The designer assigns a factor that represents the expected quality of a project's drainage system to minimize moisture intrusion into the pavement structure.

For flexible pavements, the layer coefficients are modified to reflect the expected quality of drainage and percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation.

For rigid pavement design, the level of drainage is addressed through the use of a drainage coefficient. This coefficient represents the quality and effectiveness of drainage systems and the percent of time during the year the pavement structure would be subjected to moisture levels approaching saturation.

For rigid pavements, another structural factor used is its load transfer ability. Rigid pavements have the ability to distribute loads across discontinuities, such as joints or cracks and small voids under the slab. Load transfer devices, aggregate interlock, and the presence of tied longitudinal joints and concrete shoulders influence this value.

Another structural factor applied to rigid pavement designs is a loss of support value. This factor accounts for the potential loss of support arising from subbase erosion and/or differential vertical movement.

9.4.5 REINFORCEMENT VARIABLES

Rigid pavements may be reinforced with joints, unreinforced with joints or continuously reinforced with no joints. The steel reinforcement is used to control transverse cracking.

Joint spacing is particularly important in the performance of plain jointed pavement. The spacing is usually much closer than in reinforced pavements to control cracks due to temperature and moisture enduced stresses. The slab spacing selected must also minimize joint movement thus protecting the aggregate interlock value of the joint. DelDOT has a standard spacing of 20 feet [6 m] for 12 inch [305 mm] Portland cement concrete pavement. (See the Standard Construction Details.)

9.5 STRUCTURAL DESIGN FOR FLEXIBLE PAVEMENTS

A flexible pavement structure may consist of three layers, designated as a subbase course, a base course, and a surface course. A flexible pavement system distributes the load by particle-to-particle-contact by interlocking, friction, and cohesion through its thickness. The surface

course usually consists of a binder course and a surface course.

The concept of all pavement design procedures is to determine the required structural thickness based on the projected traffic loading. Using this as a basis, the most economical and constructable combination of materials is determined. For flexible pavements, this consists of determining the required thickness of the pavement's subbase, base, and surface courses.

9.5.1 SUBBASE COURSE

The subbase course is the portion of the flexible pavement structure between the subgrade and the base course. The subbase insulates the base and surface courses from frost penetration, provides a drainage medium, and a layer resistant to erosion and erosion of fine material into the subgrade. The subbase usually consists of a compacted layer of granular material, which may be either treated or untreated. Subbase materials are either a Soil Cement Base Course, Graded Aggregate Base Coarse or Borrow Type A.

This course has a less stringent specification requirement for strength, plasticity, and gradation. If roadbed materials are of high quality, the pavement design report may recommend that the subbase layer be omitted. If the roadbed materials are relatively poor quality, the design procedure will indicate that a substantial thickness of pavement is required. In this case, alternate designs are usually provided with and without the use of a subbase. The selection of an alternate may then be made on the basis of availability and relative costs of materials suitable for the base and subbase. By using less expensive materials in the lower layer of a flexible pavement structure, the use of a subbase course is often the most economical solution to construction of pavements over poor roadbeds.

9.5.2 BASE COURSE

The base course is the portion of the flexible pavement structure immediately beneath the surface course. The base course is the primary load-spreading layer. It has to be strong enough to withstand the shear stresses produced by the wheel loads, and be incompressible and rigid enough to distribute the load over the underlying materials.

Base course material typically consists of a Graded Aggregate Base Course or Bituminous Concrete Base Course. A graded aggregate base course may be crushed stone, crushed slag, crushed or uncrushed gravel and sand, or other combinations of these materials. These materials may also be used treated with suitable stabilizing admixtures such as Portland cement or asphalt. Base course specifications are generally more stringent than for subbase materials in requirements for strength, plasticity, and gradation.

9.5.3 SURFACE COURSE

The surface course of a flexible pavement consists of a wearing course and a binder course which are mixtures of mineral aggregates and bituminous materials, placed as the upper courses and usually constructed on a base course. In addition to providing a portion of the pavement structural support, the surface course must also be designed to resist the abrasive forces of traffic and weather. They are designed to be dense enough to minimize surface water penetrating the pavement. The proper aggregate selection will provide a non-polishing-skid-resistant surface, resist rutting and provide a smooth and uniform riding surface. The success of a surface course depends to a considerable degree on obtaining a laboratory mixture with the optimum gradation of aggregate and percent of bituminous binder. Open graded mixtures that provide good surface drainage and skid-resistance are available for use on high-speed, high-traffic facilities.

9.5.4 STRUCTURAL NUMBER (SN)

The structural number is an abstract number expressing the structural strength of pavement required for a given combination of the effective resilient modulus, M_R, of the roadbed soils, the total equivalent 18-kip [80 kN] single-axle loads (ESALS), the design serviceability loss, and the standard deviation and reliability factors. The AASHTO Guide provides a nomograph for determining this value. The required SN must be converted to an actual thickness of surfacing, base and subbase by means of appropriate layer coefficients representing the relative strength of the material to be used for each layer.

A design equation is used to solve for the total required SN for the entire pavement structure. By solving the equation with the effective resilient modulus value representative of the roadbed soil, an SN for the entire pavement structure is obtained represented by the general equation:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

Where:

a_1, a_2, a_3 = layer coefficients representatives of surface, base, and subbase course, respectively;

D_1, D_2, D_3 = actual thickness, in inches [mm], of surface, base and subbase courses, respectively (Open-graded surface courses are excluded from this calculation.) and,

m_2, m_3 = drainage factors modifying base and subbase layer coefficients.

The nomograph allows the pavement designer to determine the SN and provides a means through assigning a reliability factor (R) to incorporate some degree of certainty into the validity of the design process. In addition, it allows for assigning a standard deviation factor, which accounts for the variance in the projected traffic capacities and their reliability.

The SN equation does not have a single solution since many combinations of layer thickness will satisfy the equation. However, the pavement designer must use past experience, consider cost effectiveness and construction and maintenance constraints in order to avoid an impractical design.

The design procedure allows for doing a life cycle cost analysis based upon planned rehabilitation. This allows the designer to analyze the tradeoffs between thickness designs of the initial pavement structure and any subsequent overlays.

The procedure allows for considering the adverse effects of changing environmental conditions. The objective is to perform an iterative process to determine when the combined serviceability loss due to traffic and environment reaches the terminal level.

9.5.5 LAYER COEFFICIENTS

The layer coefficient expresses the empirical relationship between the SN and thickness, and is a measure of the relative ability of the material to function as a structural component of the total pavement structure.

To design a flexible alternative, the structural number over the roadbed soil is computed. Then, the structural numbers for the subbase and the base layers are determined. Using the differences between these values, the maximum thickness of any layer can be computed.

The SN for any combination of courses is determined with the design equation, the layer coefficients and the proposed thickness of each course. Alternate designs can be prepared by varying the thickness. The computed SN should equal or exceed the required SN determined from the nomographs in the AASHTO Guide.

The layer coefficients per 1 in [25 mm] of material have been established for various types and classes of flexible pavement, base course, and subbase.

Figure 9-2
Layer Coefficients

Material Type	Layer Coefficient
Type C Surface Mix	0.40
Type B Binder Course	0.40
Bituminous Concrete Base Course	0.32
Soil Cement	0.20
Graded Aggregate Base Course	0.14
Select Borrow	0.08

9.5.6 MINIMUM LIFT THICKNESS

Although the equations allow for a great number of thickness variations, there are the practicalities of constructing and maintaining a facility, which must be considered. Depending upon the material being placed, there are minimum and maximum limits in the placement depth that are practical for the available equipment to compact and are economical.

Minimum lift thickness for hot-mix is recommended to be three times the nominal aggregate size in the mix. Figure 9-3 shows the practical maximum and minimum lift thickness (compacted) that are to be applied to the materials normally used in constructing a flexible pavement section.

9.5.7 TEMPORARY PAVEMENTS

It is not practical to attempt to follow the formalized AASHTO procedures for design of temporary pavements such as needed for detours during construction. Variations in speed and ease of placement as well as the anticipated required service life of the detour significantly affect the economic justification for the structural design.

When temporary pavements are needed designers should closely coordinate with the M&R Section in the development of a practical pavement design based on knowledge of local conditions and engineering judgment.

Figure 9-3
Lift Thickness

Type of Material	Minimum Lift Thickness	Maximum Lift Thickness
Type C Surface Mix (9.5 mm)	1-¼ inches [30 mm]	2 inches [50 mm]
Type B Binder/Base	2-¼ inches [60 mm]	3 inches [75 mm]
Bituminous Concrete Base Course	3 inches [75 mm]	6 inches [150 mm]
Graded Aggregate Base Course	4 inches [100 mm]	8 inches [200 mm]
Soil Cement	4 inches [100 mm]	6 inches [150 mm]
Select Borrow	4 inches [100 mm]	8 inches [200 mm]
Open Graded	1 inch [25 mm]	1 inch [25 mm]

9.6 DESIGN FOR RIGID PAVEMENTS

Rigid pavements consist of a Portland cement concrete slab on a subbase course. The design procedure consists of developing an effective modulus of subgrade reaction based on subbase treatment and thickness, determine the slab thickness, allowing for any stage construction, adjusting for adverse environmental conditions, determining type of joints, joint sealant, and the required reinforcement.

9.6.1 SUBBASE—EFFECTIVE MODULUS OF SUBGRADE REACTION

Before the slab thickness can be determined, it is necessary to determine the strength (modulus), of the material on which the slab will be supported. This is done by determining an effective modulus of subgrade reaction (k), of the soilbed and subbase.

The effective k-value is dependent upon several factors including the roadbed soil resilient modulus, the type of subbase, the thickness of subbase, the potential of loss of support due to erosion of the subgrade and in northern New Castle County whether there is rock underlying the proposed pavement.

The subbase used in a rigid pavement structure consists of one or more compacted layers of granular material, graded aggregate or a stabilized material such as bituminous concrete. This material is placed between the subgrade and the rigid pavement. The subbase provides several very important functions:

* Provides uniform, stable and permanent support,
* Increases the effective modulus of subgrade reaction (k),
* Prevents pumping of fine-grained soils at joints, cracks and edges of the pavement,
* Reduces cracking and faulting, and
* Provides a working platform for construction equipment, especially the paver.

9.6.2 PAVEMENT SLAB THICKNESS

After developing the effective k-value the process of selecting the optimum slab and subbase thickness can begin. Past experience, economics, equipment limitations, ease of construction, and other subjective factors influence the final recommended section(s).

The AASHTO Guide provides a nomograph which provides the slab thickness based on inputting the k-value, the estimated future traffic, the reliability factor to be achieved, the standard deviation, the design serviceability loss, the concrete elastic modulus, the concrete modulus of rupture, the load transfer coefficient, and the drainage coefficient.

9.6.3 JOINTS

Joints are a very important part in assuring a rigid pavement will perform as intended. They allow for the stresses created by the expansion and contraction of the concrete during curing and during seasonal temperature changes. They also are used to facilitate construction.

The Department's Standard Construction Details for Construction provide details for locating and constructing the required rigid pavement joints. The following is a general discussion on the need, use and treatment of joints.

9.6.3.1 JOINT TYPES

The three types of joints used in constructing a rigid pavement are expansion, contraction and construction. Expansion joints provide space for the pavement to expand, preventing buckling of the slabs.

Contraction or weakened plane (dummy) joints provide relief for the tensile stresses caused by the effects of temperature, moisture and friction. Without these joints to control cracking, the slabs would crack randomly.

Construction joints are required to facilitate construction. Although used at the end of a day's pour, they are particularly dictated by the width of the paving machine and the pavement thickness.

Joints may be developed by sawing, forming, or with inserts. When sawing joints, timing is very important to prevent uncontrolled cracking and will vary during the day depending upon the slab temperature, curing conditions, and the concrete mix.

9.6.3.2 JOINT GEOMETRY

Joint geometry refers to the spacing and layout of the joints.

Transverse and longitudinal contraction joint spacing is dependent upon local conditions of materials (coarse aggregate type) and the environment, whereas expansion and construction joints are dependent on layout and construction methods.

Contraction joints are spaced to prevent intermediate natural cracking due to thermal changes and subbase friction created by the movement of the slabs. The type of joint sealant and slab thickness affects their spacing.

Expansion joints are usually used at structures and where pavement types change. They are expensive, complex to construct, and have not performed well in the past. Therefore, they are used only where absolutely necessary.

The spacing of construction joints is a function of the daily construction activities and equipment. They are used when equipment breaks down and at the end of the day's pour. Longitudinal construction joints are placed at lane edges to maximize pavement smoothness and minimize load transfer problems

The width of the joint is a function of the slab length, movement due to opening and closing by temperature cycles and concrete shrinkage. More movement expected at the joint affects the quality and cost of the joint sealant used.

The depth of a joint is selected to ensure that the slab will crack where intended.

9.6.4 REINFORCEMENT DESIGN

Concrete pavements inherently crack. Depending upon the slab length and depth selected, it may be necessary to provide steel reinforcement within the pavement slab. The purpose of the reinforcement is not to prevent cracking but to control the crack width. Excessive cracking allows for moisture intrusion into the subgrade which is the leading cause for distress in the slab.

Stresses leading to cracking are temperature and moisture related contraction of the slab. These stresses are resisted by the subbase as friction and shear between it and the slab. The result is tensile stresses that are minimum at the mid-point of the slab. To resist these stresses and limit the crack width, reinforcement is installed. The AASHTO Guide provides methods for designing the necessary reinforcement for both jointed reinforced concrete pavement and continuously reinforced concrete pavements.

9.7 PAVEMENT DESIGN FOR REHABILITATION OF EXISTING PAVEMENTS

9.7.1 REHABILITATION CONCEPTS

Bituminous concrete pavements deteriorate because of climatic conditions, age, and traffic. Transverse and longitudinal shrinkage stresses occur due to temperature changes. Over time, material problems can develop causing surface problems with stripping, raveling, weathering and bleeding of the asphalt. Repeated traffic loadings eventually cause fatigue cracking allowing moisture into the subbase causing loss of subgrade support leading to pavement cracking or failure.

For composite pavements with both concrete and asphalt as components of the pavement structure, the most prominent problem is reflective cracking from joints and cracks in the concrete base. This is caused by a combination of underlying slab movement due to temperature changes and heavy loads crossing the joints and cracks. The primary resulting distress is spalling of the asphalt as well of the concrete if severe enough.

Concrete pavements react differently depending whether or not they are reinforced. Over time and load applications each reacts different-

ly. The only reinforcement in non-reinforced pavements is tie bars across the longitudinal joints to keep the slab from separating and dowels at the transverse joints to transfer loads across the joints. If dowels are not used, the design depends only upon aggregate interlock to transfer these loads. With loss of subgrade support, cracking of the slab can occur at almost any location. However, more common problems occur at joints. Once the joints are no longer free to move, spalling, buckling, and random slab cracking can result. If joint sealant is lost, aggregate interlock is lost, or cracks become too wide, pumping of the subgrade within the travel lane and the shoulder can occur. This leads to erosion of support material and faulting and cracking

Reinforced concrete pavements perform much like non-reinforced pavements. They usually have longer joint spacing and internal reinforcement to resist the larger tensile stresses. There are fewer joints to relieve stresses; when their free movement is restricted, rapid faulting of the pavement can occur. In addition, normal shrinkage, thermal curl, and load applications cause cracks in the slab that over time grow in width, allowing moisture and road salt to infiltrate. Corrosion of the reinforcing mesh occurs. As the loads are repeated, pumping begins leading to faulting and spalling of the pavement.

Continuously reinforced concrete pavements deteriorate under heavy truck loading and are also adversely affect by moisture under the slab. This type of pavement is more complicated to construct and deterioration will occur due to inadequate consolidation, poor vertical steel placement, and inadequate steel overlap.

Because of the need to maintain the pavement systems now in place, several rehabilitation strategies have been developed to address the various problems that may affect a pavement's performance. The objective is to extend the pavement’s service life. The alternative methods of rehabilitating a pavement range from a simple overlay to complete removal and replacement. When developing a rehabilitation strategy a combination of the following alternatives are considered:

1. Resurfacing to provide structural capacity and/or serviceability either using concrete or asphalt,
2. Replacing or restoring malfunctioning joints,
3. Pavement subsealing prior to resurfacing or as a part of concrete restoration,
4. Grinding rigid pavements to restore smoothness.
5. Removing and replacing deteriorated materials,
6. Reworking or strengthening bases and subases,
7. Recycling existing material,
8. Improving the subdrainage or adding underdrains,
9. Joint and crack sealing,
10. Full depth pavement repair,
11. Partial depth pavement repair, and
12. Cracking and seating.

M&R and other team members develop the most effective strategy through a detailed pavement evaluation. This evaluation is normally prepared as a part of the Department’s long term pavement rehabilitation program and will be a part of the project initiation data. The three steps for determining the preferred strategy are to (1) determine the cause of the pavement distress, (2) develop a list of possible solutions that cure and hopefully prevent reoccurrence, and (3) recommend the preferred solution. The preferred solution includes an analysis of funding, traffic control problems, minimum desirable service life, utility conflicts, clearances to overhead structures, available materials and equipment, contructibility, future maintenance and reliability based on past performance.

9.7.2 TYPES OF DISTRESS

Although furnished at the project initiation stage, each project usually will have a project scoping meeting and preliminary field review prior to beginning detailed design. The field review will include checking the pavement for condition and any damage that may have occurred since the last pavement survey to confirm the proposed strategy is still valid. The following discussion is a brief description of the various conditions the designer may observe depending upon the type of pavement being evaluated.

9.7.2.1 ASPHALT PAVEMENTS

Asphalt pavements usually have the following major distress conditions: (1) alligator or fatigue cracking, (2) longitudinal and transverse cracking, (3) depressions, and (4) rutting.

Alligator or fatigue cracking is caused by repeated traffic loadings. They are a series of interconnecting cracks caused by fatigue failure of the bituminous concrete surface. The crack starts at the bottom of the asphalt surface or the stabilized base due to high tensile stresses and propagates to the surface as a longitudinal crack. After repeated load applications, a network of these cracks form that look like chicken wire or the skin of an alligator. This type of cracking does not occur in asphalt overlays over concrete—only in high load areas and is considered a major structural distress. Pattern cracking not in high load areas is called block cracking.

Longitudinal cracks are parallel to the pavement's centerline or paving laydown direction. They may be caused by: (1) a poorly constructed paving lane joint; (2) shrinkage of the bituminous concrete surface due to low temperatures or hardening of the asphalt; or (3) a reflective crack caused by cracks beneath the surface course, including cracks in Portland cement concrete. Transverse cracks are perpendicular to the pavement's centerline; they are caused by (2) or (3) above and are not usually load-related.

Depressions are localized pavement surface areas that are slightly lower than the surrounding pavement. Depressions are most noticeable during and after a rain. If deep and large enough, depressions may cause hydroplaning or an unpleasant ride. Depressions may be initially built into the pavement by the paving operation or as a result of settling of the surface support structure.

Rutting is a surface depression in the wheel paths. Usually, there is uplift along the sides of rutted areas. Rutting is the result of permanent consolidation or lateral movement of any of the pavement layers or subgrade due to traffic loads. Rutting may also occur because of plastic movement of the asphalt due to high temperatures, poor design mix or inadequate compaction during construction.

In addition to the major distresses, the pavement survey may indicate surface corrugation areas, joint reflection cracking, lane and shoulder drop off, lane and shoulder separation, patch deterioration, polished aggregate, potholes, raveling, and weathering.

If necessary, the M&R Section may determine that field observations are not adequate or need to be supplemented to identify underlying problems. In that case nondestructive testing (NDT) will be conducted. NDT is used to:

- Evaluate the in-situ (in-place) structural capacity of the pavement,
- Evaluate the capacity of joint and load transfer, and
- Detect the presence of voids under the pavement.

9.7.2.2 CONCRETE PAVEMENTS

Jointed concrete pavement may have the following distresses: (1) pumping, (2) longitudinal cracking, (3) spalling of transverse or longitudinal joints, or (4) Alkali-Silica-Reactivity (ASR).

Pumping is the ejection of material through joints or cracks, caused by the deflection of the slab under moving traffic. In some poorly drained pavements particularly in superelevated sections, water may bleed through the joints and cracks after a rain or continuously if large amounts of water are present.

Longitudinal cracks are caused by a combination of heavy load repetition, locking of load transfer devices, thermal and moisture stresses, and curing shrinkage stresses. Cracks that are spalling and/or faulting are considered a major structural problem.

Spalling of cracks and joints is the cracking, breaking or chipping of the slab edges within 2 ft (0.6 m) of the joint or crack. Spalling usually does not extend the full depth of the slab, but intersects the joint or crack at an angle. Spalling is a result of one or a combination of the following: (1) excessive stresses at poorly sealed or cleaned joints and cracks which allow incompressible material to accumulate preventing the pavement from expanding, (2) disintegration of the concrete, (3) weak concrete overstressed by repeated loading, and (4) a poorly designed or placed load transfer device.

Continuously reinforced concrete pavements usually show punchout and patch distress. Punchout is the loss of aggregate interlock between closely spaced cracks. The cracks fault, spall and under load applications the steel reinforcement ruptures causing concrete pieces to punch down. This type of distress is considered a major structural problem. Due to the difficulty in patching continuously reinforced pavements the failure of previously constructed patches can be anticipated.

ASR is another type of distress commonly found in Delaware. The pavements exhibiting the most severe ASR were generally constructed in the 1980's. ASR is the reaction between the alkalis (sodium and potassium) in Portland cement and certain siliceous aggregates. The product of this reaction is a thermodynamically metastable. The gel in the presence of water absorbs it, and causes expansion and cracking of the concrete. Once an ASR pavement has been identified, one possible solution is to apply lithium treatments. Lithium treatment will not repair the concrete, but will slow the further progression of ASR. Normally the pavement will have to be removed and replaced. Overlaying ASR concrete with a standard hot-mix overlay may trap moisture in the slab and accelerate the ASR causing premature failure of the overlay. However, hot-mix overlays with water proofing properties may minimize further deterioration for several years. M&R will recommend the most suitable solution when encountering ASR pavements.

In addition, other pavement distresses that may be observed in a field review are blow ups, corner cracks, depressions, durability "D" cracking, lane and shoulder drop-offs, lane and shoulder separation, patch deterioration, popouts, and staining of the pavement due to subgrade drainage problems.

Concrete pavements can also display roughness caused by irregularities in the pavement surface that adversely affect the ride quality, safety, and vehicle maintenance costs. Roughness is measurable based on the multi-frequency of waves, wavelengths and amplitudes. Roughness can be built into the pavement when constructed or develop over time due to traffic, climate, and other factors. Equipment is used to measure the roughness and a profile developed showing the vertical movement between the trailer axle and body. The results are reported in in/mile or m/km for the International Roughness Index (IRI). The roughness survey identifies areas where severe roughness exists and needs correction. Data provided can be used in developing a PSI which estimates the user's subjective assessment of the pavement condition. Surveys taken before and after a project can be used to document the benefits of the work to the traveling public.

9.7.3 DRAINAGE SURVEY

As emphasized throughout this chapter, the presence of moisture is a primary cause of distress or failure of all pavements. Therefore, a drainage survey is an important component of pavement evaluation. Moisture conditions are caused externally by the climatic conditions and internally by the properties of the materials composing the pavement structure. The severity of damage caused by excessive moisture will influence the decision on which rehabilitation strategy to select.

Since moisture problems can exist in any layer of the pavement structure, more than visual observations may be needed. Cores and even nondestructive deflection testing may have to be conducted. It is necessary to determine which material is responsible for the moisture-related damage and if an economical rehabilitation to correct the problem is to be initiated. Not identifying and correcting the problem could lead to a failed project. A valuable tool in this evaluation is the as-built plans.

In addition to determining if the pavement structure is freely draining and moisture resistant, the entire roadway section should be evaluated including:

1. Are the ditchlines free of standing water? If not, how high does it stand and will it infiltrate the pavement structure?
2. Are ditchlines and pavement edges clear of the type of growth that would indicate excessive moisture?
3. After a rain, is water standing in the joints or cracks. Is there pumping, is there standing water adjacent to the pavement or on the shoulder?
4. If there are drainage outlets including underdrains, are the outlets clear, at the proper elevation, and working?
5. Are drainage inlets clear and cross slopes adequate to remove the water from the pavement surface?
6. Are joint and crack sealants in good condition and preventing surface water infiltration?
7. Are there signs of pumping, such as pavement discoloration or the presence of fine material at joints or pavement edges?

Recommending drainage improvements to the pavement structure can be a very expensive item and should be carefully evaluated and documented.

9.7.4 RESTORATION

Restoration of a pavement includes the work required to return the pavement's level of serviceability for a designated time period. Frequently, some level of restoration is performed prior to an overlay or resurfacing. Work undertaken to restore pavements is quite varied and includes:

- Full-depth repair of jointed concrete pavement
- Full-depth repair of continuously reinforced concrete pavement,
- Patching with bituminous mixtures,
- Partial-depth spall repair,
- Slab stabilization and slab jacking,
- Diamond Grinding, grooving and cold milling,
- Pressure relief joints,
- Load transfer restoration,
- Joint and crack sealing,
- Surface treatments,
- Subdrainage, and
- Shoulder improvements.

9.7.4.1 FULL-DEPTH REPAIR

One of the most expensive restoration alternatives used on all types of pavements is full-depth repair. To limit the repair areas and the

potential of overruns, the specific distress or distresses to be addressed should be clearly established both during the design and construction phases. Depending upon the severity, distresses that may lend themselves to full-depth repair are blow-ups, corner breaks, durability "D" cracking, ASR, excessive spalling, and loss of load transfer. Intermediate working cracks may also have to be repaired by full-depth replacement or a working joint. Not addressing areas that need full-depth repair prior to an overlay could result in continued deterioration and premature failure of the overlay.

The major considerations to ensure satisfactory performance using this rehabilitation approach are:

1. Joint design,
2. Selection of the repair areas and their boundaries,
3. Preparation of the repair area,
4. Placement and finishing of the repair material,
5. Joint sealing material and its installation, and
6. Curing time and traffic control.

Usually the full-depth repair material is the same as the adjacent pavement. However, funding, traffic control or other reasons may require that concrete pavement be repaired with bituminous material. Using materials of different physical properties and characteristics can cause several problems. Differentials in expansion and contraction lead to pushing, shoving and humping requiring frequent milling to maintain rideability. The bituminous patch is more compressible and may allow excessive opening of remaining joints in the concrete pavement resulting in spalling, pumping, and faulting. In overlaying the patched pavement, there may be an increase in reflective cracking from the underlying joints The patch design may not have an equivalent structural strength. The difference in initial cost and possible future maintenance problems may not make using dissimilar materials the best solution.

9.7.4.2 PARTIAL-DEPTH REPAIR

Some distresses within the upper third of the slab in concrete pavements lend themselves to partial-depth repairs. Partial-depth repairs may or may not be more cost effective than full-depth depending upon the size, location, number, materials used, lane closure time and production limitations.

The distresses applicable to partial-depth repair are:

1. Spalls due the use of certain types of joint inserts,
2. Spalls caused by joint movement locking due to failed sealant and subsequent intrusion of incompressible material,
3. Spalls caused by misplaced dowels or other load transfer devices, and
4. Localized areas of scaling.

The M&R Section should be contacted for details on partial-depth patches as they require the use of better designs and construction techniques to be successful.

9.7.4.3 SLAB STABILIZATION AND SLAB JACKING

Although not that frequently used, voids under pavements can be filled, restoring the support strength of the pavement structure. The loss of support is caused by erosion of the subbase and/or subgrade by pumping or, in severe cases, movement of freely flowing water. Slab stabilization does not increase a pavement's structural strength, correct depressions or faulting and other distresses.

A slab that has settled may be raised through a process called slab jacking, also known as mudjacking. This procedure which injects water-soil-cement slurry (i.e. grout or mud) through holes drilled into the pavement slab under high pressure lifting the pavement back into its origi-

nal profile is also used infrequently. A process using expanding polyurethane foam may also be used. Another alternative is the injection of a flowable fill that consists of a mixture of fly ash and water that expands and hardens as it dries out. If not correctly done, all three of these methods can crack the slab.

9.7.4.4 DIAMOND GRINDING, GROOVING AND PAVEMENT MILLING

Diamond grinding is used to restore a concrete pavement's smoothness using closely spaced diamond-impregnated blades. Diamond grinding is effective in: (1) removing joint and crack faulting, (2) remove wheel path ruts, (3) correct joint unevenness caused by slab warpage, (4) removing areas of roughness and (5) restoring transverse drainage. Diamond grinding does not correct the problem that created the distress and may have to be use in conjunction with other rehabilitation techniques. Subsealing, slabjacking, full-depth, and partial depth repairs should be completed before grinding.

Grooving of concrete pavements is used to reduce hydroplaning by improving surface drainage and improving surface friction on curves or polished aggregate surfaces.

Pavement milling is used to remove asphalt surfaces as much as 3 to 4 inches [75 to 100 mm] in depth using carbide-cutting teeth mounted on a rotary drum. If used on concrete pavements, pavement milling leaves an extremely rough surface and spalled joints. Therefore, unless an overlay is included in the work, it should not be used on concrete.

Pavement milling is a very effective rehabilitation technique addressing several problems including:

1. Restoring the curb line of asphalt pavements,
2. Restoring cross slope and correcting inlet drainage problems,
3. Improving the friction resistance of asphalt pavements,
4. Removing asphalt overlays on concrete pavements,
5. Providing a good bonding surface on concrete for overlaying,
6. Removing material as part of a recycling project, and
7. Restoring pavement smoothness.

All three of these restoration techniques are cost effective, relatively quick and proven to perform well but do not improve the structural integrity of the pavement.

9.7.4.5 PRESSURE RELIEF JOINTS

As concrete pavement ages, there can be a net increase in its length. These increases can result from: poorly sealed joints and cracks filled with incompressible material, pumping of the base material into the joints and cracks, or the use of expansive or reactive aggregates in the initial mix.

Because pressure relief joints can adversely affect contraction joints, they are not usually installed unless there are major blow-up problems or the pavement has expanded to the point of shoving bridge abutments. These type of joints provide no load transfer and are also subject to closing through the intrusion of incompressibles, eventually becoming part of a pavement distress problem.

Pressure relief joints are full-depth and full-width saw cuts at mid-slab and are 2 to 4 inches [50 to 100 mm] in width. The joint must be cut across all lanes within the same time period to relieve the compressive stresses and is usually done at night under reduced traffic flow.

9.7.4.6 LOAD TRANSFER RESTORATION

Poor load transfer between joints can cause spalling, rocking, pumping, faulting, and corner breaks. Doweled joints are effective in providing

load transfer but under repeated load applications can work loose and lose this capability.

Whether or not joints and cracks have lost their load transfer ability, is determined through field observation and measurements conducted in the cooler parts of the year or day. The Falling Weight Deflectomator (FWD) may be used to test for load transfer capability at joints.

There are two techniques used to restore load transfer, both of which require the installation of dowels. Slots can be cut in the pavement, if the adjacent pavement is sound, and the dowels installed. More frequently and, usually a better alternative, load transfer is restored through full-depth repair.

9.7.4.7 JOINT AND CRACK SEALING

One of the easiest, cost effective and proven techniques in maintaining serviceability and restoring concrete and asphalt pavements is sealing and resealing joints and cracks. Inadequate or failed sealant allows free water and incompressibles to enter the pavement structure causing erosion of pavement support and blow-ups.

Joint sealants are constantly being improved and may be field poured—self-leveling, hot-poured, cold-poured, preformed compression seals, or field-poured—nonself-leveling sealants.

The performance of a sealant depends primarily upon the proper preparation of the joint to receive the sealant. The factors include the shape of the reservoir created for the sealant, the bonding ability of the sealant to the sidewalls, the surface preparation, the cleanliness of the surface, the dryness of the surface and the properties of the sealant. In addition, it is necessary to predict the amount of movement expected in the crack or joint to customize the type of sealant to fit the distresses encountered and techniques to be implemented.

Cracks occur randomly and are irregular in dimension and direction. Usually cracks do not experience the faulting and movement that happen at joints and sealing is not as tightly controlled. If the field survey indicates the width of cracks are large enough to cause large movements, then the cracks are sealed and treated similar to joints.

9.7.4.8 SURFACE TREATMENTS

The use of surface treatments or seal coats is a very effective way to rehabilitate. The technique is to apply asphalt and/or aggregate to a roadway surface at a depth of less than 1 inch [25 mm].

Surface treatments are classified by their composition and include:

Asphalt Chip Seal (Surface Treatment)—one or more applications of asphalt and stone chips.

Open-Graded Friction Course— the asphalt and aggregate mix is designed to be freely flowing to remove surface water thus reducing hydroplaning. In addition, it provides some improvement in skid resistance.

Micro Surfacing—a process using a moving pug mill which mixes latex rubber with an asphalt emulsion, aggregates and other additives that is placed on the surface.

Slurry Seal—a diluted emulsion mixed with a sand-size aggregate in a special mixer, and squeegeed onto the pavement. The thickness is usually less than one half inch [12 mm].

9.7.4.9 SUBDRAINAGE IMPROVEMENTS

The long-term presence of water within the pavement structure is largely responsible, directly and indirectly, for many of the distress and performance problems, which are found in the pavement systems. A pavement survey and evaluation includes a very detailed study of the drainage and subdrainage within the entire roadway cross section. Considerable design effort and cost are involved when correcting this

type of problem. Unless the rehabilitation program involves complete pavement reconstruction, improvements are limited to design and construction of longitudinal drains. Any modifications to or additions of transverse drains or drainage blankets would be limited to projects involving complete pavement replacement. However, the addition of longitudinal perforated underdrains to collect and outlet excess moisture in the pavement structure is a feasible and cost effective option.

9.7.4.10 SHOULDER IMPROVEMENTS

Shoulders are evaluated much like the mainline pavement and are include in the distress survey, drainage survey, traffic survey, structural evaluation, and subgrade and materials evaluation. Shoulders provide not only safer traffic flow but also give lateral support for the mainline pavement structure.

Shoulders can be rigid or flexible and distress similar to mainline pavements. Many of the same rehabilitation techniques are used. Reviewing the existing cross section can be very valuable. If the initial shoulder pavement design was not compatible with the mainline pavement structure, it may be a major contributor to its failure.

Two of the most common problems which occur are lane/shoulder joint separation which allows water into the subbase, and blockage of water draining out of the mainline subbase which is usually of more granular and higher quality. The material used in shoulder construction also may be of a different thickness. All of these affect the interaction between the two pavement structures.

Other distresses found in shoulders are pumping, fatigue cracking, lane/shoulder drop-off, frost heaving and differential shoulder support.

9.7.5 RECYCLING

Recycling is the term used to describe the process, which uses existing pavement materials to construct new pavements. The primary purpose is to conserve natural resources. In some cases, there may also be a net cost savings.

9.7.5.1 RECYCLING RIGID PAVEMENTS

Eventually, all pavements reach the end of their useful and/or structural life and must be reconstructed. This occurs when there is little or no structural life left due to extensive cracking, extensive slab settlement or heave, extensive joint deterioration requiring excessive full-depth repair, extensive concrete deterioration due to poor durability, and failure to meet geometric design standards.

Two methods of recycling are: to break the pavement slabs into smaller sections and leave them in place (rubblizing) as a base for resurfacing; or to break and remove the pavement slabs to an area for crushing and reuse. The crushed pavement material may be reused as aggregate in untreated dense graded aggregate bases for Portland cement concrete surfacing, asphalt concrete surfacing, fill, filter material, and as a drainage layer for edge drains.

Each method of recycling has its own cost and feasibility studies that need to be conducted before making a selection. Both methods involve the use of vibratory, hydraulic, pneumatic, or diesel chisels or hammers to demolish the existing pavement. Common factors to consider are: is the resulting recycled material actually reusable; are the underlying soils adequate to support an upgraded pavement; and are there shallow utilities (older gas, water or sewer lines such as vitrified clay or cast iron) that are sensitive to the equipment pounding and vibration and may rupture during the demolishing process.

The pavement may be demolished, removed, crushed and used for other construction purposes. This involves hauling the material to a crushing plant and having an electromagnet remove any reinforcement. The crushed material and salvaged steel would be available for reuse. The coarse-aggregate-sized particles resulting from the crushing process have a good shape and an-

gularity, high absorption and may have a lower specific gravity than virgin aggregates. For reuse of this material the concerns are the possible contaminants in the rubble including reinforcing steel, dowel bars and baskets, admixtures, chemical substances such as deicing salts, sea salt, oil, joint sealant material and soil.

9.7.5.1.1 RUBBLIZATION

Rubblization is initially the most expensive pavement recycling method, however, a selective use of it can be very effective in cost and construction time.

When rubblizing plain or reinforced concrete pavement, the rubble is leveled and left in place as the base for a new pavement. Maintaining traffic is a consideration as there must be enough area for the equipment to operate as well as a protective barrier. Even on a four lane facility this will require a median shoulder strong enough to temporarily carry full traffic loads or possibly a detour.

The rubblized surface essentially remains at the same elevation as the existing pavement. Therefore, matching existing curbs, drainage structures, intersecting roadways, and driveways is more complicated and time consuming. In addition, it may become difficult to meet height standards for guardrail and other appurtenances, as well as maintain vertical clearances for structures.

Rubblization is for total reconstruction. Rubblization reduces the structural value of a PCC pavement to a stone base. It requires a thick overlay, typically 11 inches [280 mm] of asphalt or 10 to 12 inches [255 to 305 mm] of concrete. It is a major reconstruction technique and therefore it should be used only when the pavement has reached the end if its service life, as indicated by severe deterioration, ASR, or severe freeze and thaw damages, etc.

Rubblization can be used when other concrete pavement restoration methods will not work. Thoroughly evaluate the existing condition of PCC pavements. Concrete pavement restoration (CPR) techniques, such as diamond grinding, patching or sealing, should be ruled out before the use of rubblization is specified. Pavement Management and M&R should be consulted in the selection of reconstruction/restoration technique.

Rubblization cannot be used over a subgrade demonstrating widespread instability or of poor condition. Many concrete distresses result from poor subgrade support conditions. Rubblizing a pavement destroys the concrete slab's bridging action, causing problems to become more pronounced. The poor support condition could be due to poor soils, poor drainage or high moisture content. If the problem is widespread, rubblization cannot be used. Thorough subgrade investigation is essential for the successful application. Contact M&R for subgrade investigation.

Treat rubblized PCC pavement as a subbase. Although rubblization provides the benefits of an in-place recycle opportunity and an interlocked stone base, it has challenges as well. As for all subbase materials, gradation and density are two important factors, but the control of these two factors is more difficult since it is in-place recycling.

The pavement breaker may be powerful, but as energy dissipates through the depth of the slab, it produces smaller pieces at the top and larger pieces at the bottom. A soft subgrade or the reinforcement in the slab only compounds the difference in sizes. The recommended requirement is for the top pieces to be 3" maximum and 12" maximum at the bottom. Acceptance of a larger size will increase the probability of future reflective cracking. A good density is achieved through the interlocking and good compaction.

With good control of gradation and density, it is reasonable to expect a good fatigue resistance performance of an asphalt overlay, which is a major controlling factor for a flexible pavement service life.

Rubblization is a balancing act. The size of rubblized concrete can be controlled. A larger size will provide a stronger support (thus a better structural value), but it increases the likelihood of reflective cracking (thus a reduced service life). The designer needs to balance these two in the pavement design. A long term performance evaluation to validate this design value may be necessary.

Install a drainage system. An adequate subgrade drainage system is essential for rubblization and future performance of the pavement. Rubblization cannot be successfully done over a wet subgrade. The drainage system should be in place at least two weeks prior to the rubblization. In a special case, if the drainage system cannot be installed prior to actual rubblization, then a time limit should be specified to have the drainage system installed immediately following rubblization.

Use a test section, not a second pass of the breaker. A second pass over a rubblized area does not enhance the quality of rubblization, and it could cause more damage. Test sections should be done to calibrate all rubblization variables (machine related-velocity, frequency, pressure or force, shoe size and conditions related-concrete condition or state of distress, thickness and subgrade conditions). The objective is to achieve the required sizes of rubblization both at the top and bottom of the PCC pavement for a good service life.

Selection of Rubblization Equipments and Production Rates - Among the different types of equipment for breaking the pavement, two frequently used types are resonant pavement breakers (a low impact, low-amplitude, high frequency vibration to the slab) and multiple head breakers (12 to 16 drop hammers mounted laterally in pairs with half of the hammers in a forward row and the remainder diagonally offset in a rear row). The multi-head breaker rubblizes a full lane width in a single pass with a production rate of about one lane mile per shift per day, while a resonant breaker may take up to 20 passes for a full lane width with a production rate of about ½ miles per shift per day.

The multi-head breaker may cause damages to the subgrade. the resonant breaker may produce concrete size too small at the top to meet the design requirement. Unless the equipment selection is specified on the plan, the specification will allow the use of either breaker.

A concrete overlay can be used. Rubblization was developed to eliminate reflective cracks in the asphalt overlay. Engineers still have an option of using concrete overlay, considering cost, service and construction time. Consult M&R for pavement design options.

Soft spots need to be repaired. Original PCC pavement could bridge the soft spot and this soft spot will show up after rubblization. Any depression one inch or greater in depth from the immediate surrounding area should be examined to see if it is due to poor underlying subgrade before the application of filler aggregate as required by the specification. A bearing capacity failure during rubblization could cause depression on one area and heave in other areas.

The repair of soft spots is necessary not only for a long term performance of the pavement but also for a good working platform for paving operations.

Pavements with delamination type cracks should not be rubblized. Horizontal cracks hinder the rubblization process by absorbing energy and decreasing the effective depth of rubblization.

Survey and set a new profile. Although rubblization does not significantly change the existing grade, simply specifying a few inches over the existing grade may not be adequate. The existing PCC pavement may not have adequate cross slope, or it is distressed due to poor subgrade which might have resulted in an irregular profile. Comparing survey results with the original geometry could provide clues on distress or subgrade conditions.

The design service life should be 20 years or more; this should be noted on the plans. Rubblization with an overlay is major reconstruction. It should not be used as a short-term fix.

9.7.5.2 SURFACE RECYCLING OF BITUMINOUS PAVEMENTS

Surface recycling is the process that either reworks or removes and replaces a limited portion, usually about one inch [25 mm], of the asphalt surface. The distresses addressable through this process include: raveling, flushing or bleeding, low surface friction, weathering, poor drainage profile, shallow rutting, minor corrugations and block cracking. It may also be used to correct problems in the profile grade line.

The major advantage for selecting this rehabilitation method is the minimal amount of work involved. There are two processes. The first is hot surface recycling involving heating, scarifying, remixing, and repaving recycled material. Other surface recycling methods include hot pavement removal using a heater-planer or cold milling using a rotary drum equipped with closely spaced carbide teeth.

The material to be reused has a rejuvenating agent and soft asphalt added to restore the pavement consistency, viscosity or penetration. DelDOT does not typically use this method for recycling surface courses.

9.7.5.3 IN-PLACE RECYCLING OF BITUMINOUS PAVEMENTS

In-place recycling is a process in which the pavement surface is ripped up or pulverized to a depth greater than 1 inch [25 mm]. The material is cold worked and reused as an aggregate base. The recycled material may be further strengthened by the addition of admixtures such as asphalt, lime, cement or fly ash. The recycled material will perform similar to new stabilized material upgrading the structural capacity, correcting surface distresses and mixture problems in the asphalt pavement, and correcting base course problems such as gradation, moisture and density. Correction of profile grade problems can be made with this material.

9.7.5.4 HOT-MIX RECYCLING OF BITUMINOUS PAVEMENTS

Hot-mix recycling is the process in which all or some of the pavement structure is removed, reduced to the required size, and mixed hot with added asphalt cement at a central plant.

This process is used to correct surface roughness, cracking, rutting, surface friction, raveling, inadequate structure, and inadequate traffic capacity. It should be remembered that the underlying cause for structural inadequacy would not be corrected using this process.

9.7.6 RESURFACING

The most common type of rehabilitation for existing pavements is resurfacing. Resurfacing can correct many common distresses and add additional strength to the pavement structure.

Problems encountered with overlay projects include inadequate thickness to correct surface problems when the problem is actually structural, inadequate repair of the deteriorated areas, unanticipated increasing traffic loadings, and not addressing reflective cracking.

9.7.6.1 TYPES OF OVERLAYS AND THEIR FUNCTIONS

Overlays can be of asphalt or Portland cement concrete. There are several variations of these overlay techniques, which are designed for specific applications.

The most commonly used overlay is dense-graded, hot-mixed asphalt concrete, which may be used on existing asphalt or Portland cement concrete pavements.

Portland cement concrete overlays are designed specifically for the type of existing

pavement and may be unbonded, partially bonded or fully bonded.

In analyzing overlay alternatives, the designer should look at how extensive are the proposed pre-overlay repairs, structural adequacy of the existing pavement, subdrainage, existing distress conditions to be corrected, future traffic loadings, traffic control problems, clearances to overhead structures, and overall costs.

Asphalt overlays of rigid pavements are used both to correct surface problems and to improve the structural strength. Thicknesses greater than 2.5 inches [65 mm] are needed to provide improvements in strength. The greater the overlay thickness, the higher the possibility of rutting if compaction is not controlled but a significant reduction in reflective cracking can be expected.

Reflective cracking occurs in overlays due to thermal cracks in flexible pavements and joints in rigid pavements. Low temperatures, temperature cycles and traffic loads cause movement in the existing pavement leading to stresses reflecting through the overlay.

Another method to reduce reflective cracking in asphalt overlays of rigid pavements is to fracture the underlying slabs into pieces 2 to 3 feet [600 to 1000 mm] in size. After cracking the slab, a heavy roller is used to ensure the slabs are fully seated before the asphalt overlay is placed.

Reflective cracking leads to increased infiltration of water into the pavement structure that rapidly deteriorates the overlay. This creates potholes and other distresses. Treatments to minimize reflective cracking include the use of reinforcing fabrics, stress-relieving interlayers of rubber-asphalt with aggregate chips, crack-arresting interlayers of large aggregate bituminous material to blunt the cracks, and extensive pre-overlay repairs including determining existing joint and major crack locations, sealing them and saw cutting after paving.

Portland cement concrete overlays of existing rigid pavement may involve extensive pre-overlay repairs, are more difficult to construct and have added initial traffic control costs. These overlays may be fully bonded, partially bonded and unbonded.

Fully bonded concrete overlays require that the existing pavement be in good condition and that a complete and permanent bond be achieved. Fully bonded overlays are thinner and range from 2 to 4 inches [50 to 100 mm] in thickness.

Partially bonded concrete overlays require repair and/or replacement of damaged slabs. Surface cleaning by sweeping and either water blasting or sand blasting is necessary to achieve as much bond as possible. These overlays are usually between 5 to 7 inches [25 to 175 mm] in thickness.

Unbonded concrete overlays are used to improve the structural capacity of a rigid pavement in poor condition. A bond breaking and leveling course is placed between the existing pavement and overlay to assure there is no bonding and to absorb any movement in the base slab to prevent cracking of the overlay. Unbonded overlays are thicker and range from 8 to 10 inches [200 to 250 mm] in thickness.

All types of overlays normally require a fairly detailed design analysis. The analysis involves determining the initial construction and pavement information, a pavement distress survey, existing layer properties, future traffic analysis, existing pavement subgrade properties, overlay design properties, determining effective structural capacity, future overlay structural capacity, desired remaining service-life, and required overlay thickness design.

Chapter Ten

MISCELLANEOUS DESIGN

This chapter contains guidelines and criteria for the design of roadway elements that do not logically fit within the general categories of information covered in other chapters.

10.1 CONTEXT SENSITIVE DESIGN

Context sensitive design is a term used to identify a design process that balances the design features of a project with:

- User safety,
- Transportation system needs,
- Accessibility and mobility,
- Preservation of historic sites and districts,
- Natural and man-made environmental concerns,
- State and local economic needs, and
- Preservation of community values

Context sensitive design recognizes that the application of uniform engineering standards to specific transportation projects is not always possible. Transportation facilities are not constructed through consistent, uniform settings. In reality, landscapes are constantly changing and preventing the easy application of engineering standards. AASHTO and this manual respond to this problem by developing different sets and ranges of standards to apply to the most common types of landscapes encountered. There are standards for rural and urban environments, different types of terrain, and varying functional classes of highways. However, the designer still finds it difficult to always meet these variable standards. Historic sites, sensitive environmental areas, such as wetlands and natural areas and community concerns along with the mixing modes of transportation such as pedestrians, bicycles, and public transit, within the same right-of-way often conflict with the routine application of established engineering standards.

The context sensitive design approach is used to develop projects that achieve protection of community, historic, and environmental values while utilizing scientifically developed engineering standards that will result in a legally defensible, safe facility for the user. This process allows the designer to address community concerns while meeting the intended transportation needs, minimizing adverse impacts and enhancing the project area. Context sensitive design requires careful listening and understanding of the values placed on the project area by citizens, agencies, special interest groups, and local governments. It requires thoughtful and often creative design solutions.

10.1.1 TYPES OF PROJECTS

There are many types of projects in the Department's annual Transportation

Improvement Program. Projects are developed to preserve the integrity of the current system, to restore or increase the capacity of the system, to improve the safety of the system, to maintain suburban streets, and address other concerns in communities and municipalities.

For the highway system, the types of improvements used to expand capacity are new construction on new alignment and reconstruction of an existing roadway in-place, involving adding lanes and other capacity improvements. They also cost the most and have the greatest social, economic and environmental affects on adjacent areas. Projects on new alignment often do not have the land use restrictions or the historical, operational and safety problems associated with improving an existing roadway. This means that higher design standards can be attained. In contrast, reconstructing an existing roadway in-place to address capacity, safety and operational problems is difficult without introducing flexibility into the design. Since significant geometrical changes are not normally viable, historical data, such as traffic growth, traffic patterns, and types and frequency of accidents, become an important part of the decision-making process in assessing and correcting existing problems. Because of the difficulties in making changes, particularly in vertical and horizontal geometry, this analysis will determine if not meeting the highest design criteria is an acceptable alternative.

There are other types of projects ranging from restoring and/or maintaining a facility's capacity to resurfacing existing roadways to maintain their rideability. Many of these projects lend themselves to flexibility in choice of design options.

Ensuring that a project will be designed within the context of the community meeting the expectations of both the motorist and the community at large is the result of an early, continuous and meaningful public involvement process. Through a continuous public involvement process, the designer is assured that the purpose and need of the project is fulfilled, the needs of the community are understood and addressed, and new or additional issues do not arise during the final design or construction phase.

10.1.2 DESIGN STANDARDS

Each project should begin with determining the applicable design standards as published by AASHTO and this manual. In the context sensitive design environment, it is recognized that there are limitations, constraints, community values and other factors that require the designer to look beyond the full standards for a workable solution. Meeting the full standards for an entire project may not be possible but design alternatives that combine multiple substandard dimensions simultaneously are not acceptable. Arbitrarily lowering a project design speed to solve a problem in a sensitive area is also not an acceptable alternative. All major geometric values are based on design speed and lowering this value would preclude the use of appropriate standards where they are attainable. Where a non-standard element, dimension or approach is used, the potential adverse operational affects should be mitigated by wider paved areas, appropriate advanced signing, increased vertical or horizontal sight distance or other treatment.

The selected design criteria should properly reflect driver safety, desires, expectations, comfort and convenience. Of course there are many contraints, including terrain, land use, roadside and community effects, and cost considerations. A project is designed based on accepted design criteria, practices, guidelines, and standards. Appropriate dimensions and values are selected to produce a facility of a given quality, provide a reasonable degree of safety, and consistent expectation (standard design) for the user. Transportation Research Board’s Special Report 214

Designing Safer Roads is a valuable resource document when evaluating the various design parameters in reaching a safe and flexible set of design standards meeting project intent.

10.1.3 OPERATIONAL CONSISTENCY

Although, designs may not meet all of the applicable design standards, they can provide design/operational consistency. Ensuring the continuity of designs means that a motorist can travel to any state, city or town and, depending upon the situation encountered, react in the same driving manner e.g. all ramps are signed uniformly and have deceleration lanes. In responding to the many issues that each project faces, there is a need for flexibility in the design process, while maintaining uniform design practices. Flexibility is achieved by recognizing outstanding issues and making changes while recognizing the tradeoffs incurred. Flexibility in design should not unduly compromise the user's safety. Each project is unique and has its own community values, social, economic, and environmental constraints. Context sensitive designs recognize and address those unique elements that preserve or enhance community values.

AASHTO design standards do have a measure of flexibility, usually stating a maximum and minimum value. Many of these values were established many years ago with assumed conservative variables, some derived theoretically and others empirically. Design features that fall outside normal design criteria and accepted practice should be documented, if necessary, with a formal "Design Exception". The key to minimizing liability is documentation of major design decisions in terms of safety, capacity, route compatibility, time to construct, environmental, historic, and aesthetic considerations, and construction costs throughout the project development process. Many design options have operational experience from past projects, including accident history.

10.1.4 DESIGN CRITERIA

Design criteria have historically provided consistency in the quality, appearance and operational performance of the highway system. They are based on data available for successful operational performance, as well as theoretical modeling and field-tested research. AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book) and periodically issued supplemental reports and guidelines catalogue this data for use by designers. The Green Book contains nationally accepted design criteria for new construction or reconstruction projects that are generally applicable for all 50 states.

This manual utilizes criteria contained in the Green Book to define design standards to address transportation needs and community values as envisioned in this State. Design data in the Green Book is intentionally developed to be conservative because highway alignments are of a permanent nature, and projects are initially expensive to construct and very difficult and expensive to correct if there are safety problems. The public expects the design professional to design it right the first time, meeting the users' operational needs and safety expectations.

Since the criteria is conservative in many areas with a significant margin of safety, there is room for creative yet safe designs when a designer has to look beyond the criteria to meet a constraint or issue that arises. Again, documentation of accepted changes in design criteria is essential.

10.1.5 DESIGN CONTROLS

Design controls include: functional classification, design speed, traffic volumes, traffic mix, terrain and location. The design team, working with the public, can identify constraints that will require flexibility in the design criteria.

10.1.5.1 FUNCTIONAL CLASSIFICATION

A facility's functional classification and design speed are two major factors in setting the design criteria. The functional classification groups streets and highways based on the type of trips, mix of traffic, accessibility to the facility and the role of the facility in the total transportation system. The functional classification system establishes a heirachy based on the level of service to be provided for the users. This service ranges from the high level of service provided by the interstate system to local streets. Local streets provide a low level of service for through traffic movements but support safe local access and address community mobility needs.

The AASHTO Green Book relates the functional classification to design criteria, particularly design speed and geometrics. As a result of variances of design criteria within the functional classification system, there are overlapping ranges of values. This allows flexibility for choosing a design that is most appropriate within the determined functional classification. Land use is very dynamic, thus changing the character and use of a facility leading to a functional classification change. At the same time there may be strong support for the preservation of the existing character of a facility. The development group will become aware of this during the public involvement phase of both the planning and/or design processes.

10.1.5.2 SPEED SELECTION

In design, there are three speed concepts used: running speed, operating speed and design speed. The difference between these three values can be significant. Running speed is the actual speed of a vehicle over a specified section of highway. For evaluating road-user costs and benefits, the average running speed is used. This value varies during the day based on the traffic volume which, depending upon the roadway characteristics may have to be field measured at several locations to truly reflect the average running speed. Operating speed is the speed at which drivers are observed operating their vehicle during free flow conditions. The 85th percentile of the distribution of observed speeds is used to statistically describe the operating speed associated with a particular location or geometric feature. Design speed is a selected speed used to determine the various geometric design features of the roadway.

Design speed usually controls the design features. Selecting a design speed is not a simple task. Using a design speed as high as practical may ensure that drivers can drive as fast as comfort level will permit but may not be the best method in determining the design speed. In practice, design speed is selected to accommodate a high percentile of drivers.

The determination of a design speed is affected by many factors including: the capabilities of the drivers and their vehicles, physical characteristics of the roadway and its roadsides, the weather, the presence of other vehicles, and speed limitations. Selecting a higher design speed imposes more design constraints. Selecting a design speed based on an artificially low operating speed that does not meet the expectations of a high percentage of drivers can significantly degrade the safety of the facility.

Design speed should be logical with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of highway. The design speed chosen should be consistent with the speed a driver is likely to expect on the facility being driven. A lower design speed should not be selected where physical conditions are such that drivers are likely to travel at high speeds. Drivers adjust their speed not to the importance of the facility, but to their perception of the physical limitations and traffic flow. See Section 2.3.3 and Section 3.2.1 for more discussion

on the selection of an appropriate design speed.

Proposing an alternative design speed for a project or a segment of the project is not recommended for inclusion in the design exception process. Instead, individual geometric features should be evaluated within the selected design speed and addressed as exceptions if necessary. As discussed previously, most design features have an acceptable range of values that will meet driver expectation and provide acceptable driver safety.

Using the posted speed as the design speed is also not recommended. The design speed should be a minimum of 5-mph [10 km/h] over above the posted speed. Design speed usually approximates the 85th-percentile speed value as determined by observing a sizable sample of vehicles, but is not the highest speed that might be used by drivers unless reasonably enforced.

10.1.5.3 TRAFFIC CONSIDERATIONS

The importance placed on the operational and safety characteristics of a project is in direct proportion to the traffic volume and composition. Experience shows that the greatest contributor to the risk of an accident is traffic volume. Thus the volume of traffic may be a primary factor in decisions on design exceptions.

The designer uses two traffic volumes, current and projected in setting design controls. Depending upon the type of facility, perceived need, and existing traffic, a five or ten-year projection may be the control. New construction and reconstruction projects where increased capacity is the goal are designed to meet the needs of a projected 20-year traffic volume. Traffic forecasts are based on technical analysis, policy consensus (the State's vision of the future transportation network), anticipated type and intensity of land use, future economic activity, and other factors. Large variances with one or many of the factors used in the modeling process to project future traffic can and have occurred in the past. Some projects have exceeded their traffic projections in a very short time span and become functionally obsolete, some have never reached their projected traffic volumes and others have taken longer then the projected year to reach assigned design volumes. Context sensitive design recognizes the inherent limitations and uncertainties associated with long term traffic forecasting.

For setting design features, the forecasted daily traffic volume is further refined to develop an appropriate design hourly volume, DHV. The DHV initially selected is the 30th highest hourly volume of the year. Exceptions may be made on roads with high seasonal traffic fluctuations or other conditions, where a different volume may need to be used. Available projection techniques lead to estimated volumes. Selection of a design hour volume should reflect other constraints and controls reflecting the project's purpose and need. High DHV’s usually result in wider lanes, more lanes, and greater costs. All of these tend to increase disruption to the community and conflict with other important issues.

Another design control reached through analysis of current and forecasted traffic is the predominant type of driver and vehicle expected to use the facility. With the perception that safety will be greatly enhanced, design controls are conservatively established regarding the capabilities of the driver to react physically and mentally to conditions that may be encountered. The result of this analysis is the selection of a design vehicle. Projects involve a variety of adjacent land uses, types of intersections and alternative modes of transportation and may have several design vehicles. Each design vehicle has physical and operational characteristics that affect the design controls. These include acceleration and deceleration capabilities, ability to climb steep grades, sweep-path dimensions of turning vehicles, and height of the driver's

eye. There is a range of design vehicles and associated characteristics. There is no mandated design vehicle. The choice(s) should be made with the knowledge of the tradeoffs involved and input from the community and public while being cost and space effective.

10.1.5.4 LEVEL OF SERVICE

How the user perceives a design to meet the traffic operational conditions for the selected design hourly volume is defined as the level of service. Level of service can be quantified in terms of running speed, travel time, freedom of movement, traffic interruptions, comfort and convenience. Different types of highways have different attainable or expected levels of service. The various elements that make up each type of highway depend upon the selected level of service. Rural highways are expected to operate at higher levels of service and facilities in urban areas at lower levels with acceptable congestion. The level of service selection in context sensitive design recognizes the need to tailor the level of service to other design controls and constraints, within the context of the project's purpose and need.

10.1.6 HIGHWAY GEOMETRIC ELEMENTS—DESIGN AND SAFETY CONSIDERATIONS

Geometric design controls the horizontal, vertical, and cross sectional elements throughout the project including intersections. Selection of geometric criteria should recognize the characteristics of the potential users and the physical and operational characteristics of their vehicles. Using absolute design criteria values can lead to conflicts with community values and local constraints. In developing a context sensitive design, it may be necessary to design using the flexibility that lies within the standards while maintaining acceptable and predictable safety and operational characteristics.

10.1.6.1 HORIZONTAL ALIGNMENT

Horizontal alignment is comprised of tangents and circular curves used to adjust the alignment laterally to meet land use and terrain constraints. Horizontal curvature is a controlling feature of a design. Curve selection is based on the project's design speed and resulting superelevation rate. The transition sections between tangents and circular curves are also important elements in establishing horizontal alignment. AASHTO criteria in the Green Book are based on the amount of friction needed for the vehicle to track properly and the comfort perceived by the driver at the design speed under poor pavement conditions. Current design tables for friction are based more on driver tolerance than what is actually necessary to prevent loss of control.

Knowing that the assumptions in establishing the relationship of design speed to curvature are driver comfort and poor pavement condition will allow the designer to determine the risks in accepting a non-standard design. Reliable data on actual speeds, accident types and probable causes, and roadside conditions along the curve area will help the designer in this decision-making process. Studies show that safety is related to curvature and length of curves on higher speed facilities. On lower speed streets, there is little indication that there are safety affects associated with marginal curvature. However, where there is a predominance of higher profile vehicles with higher centers of gravity there is an increased need to meet or exceed the criteria.

In considering the safety risk of a design exception for horizontal curvature, AASHTO suggests evaluating the following factors:

- Risk is proportional to traffic volumes increase;
- Risk may generally be acceptable if the effective or nominal speed of the proposed curve is within 5 to 10 mph

[10 to 15 km/h] of the appropriate design speed of the curve;

- For roads with significant truck traffic, the risk increases due to a truck's tendency to overturn at lower speeds than passenger cars and the acceptable difference between design speed and operating speed is less;
- Risk varies proportionally with the length of curve;
- Risk of accidents within a curve is a function of the curve geometry, cross section, sight distance, and/or presence of intersections and driveways within the curve.

In providing a context sensitive design, the use of a range of curvature may be needed to mitigate various issues that arise on a project. Although it is desirable to use one minimum value for a design radius, it may not be practical. Substandard alignment decisions will require written documentation. Inclusion of other design elements to mitigate any adverse safety impacts should be considered. These elements could include additional signing, increased delineation, improved superelevation, lane widening, improving or widening the shoulder through the curve, improving roadside safety, increased clear zone, relocating or closing intersections or driveways within the curve, or providing higher surface friction pavements.

10.1.6.2 VERTICAL ALIGNMENT

The vertical alignment design elements are grade and sight distance. Grades are related to terrain and functional classification. Stopping sight distance governs vertical curvature.

Long or continuous steep grades can contribute to operational problems and safety risks. Flat grades can cause drainage problems leading to increased maintenance costs and wet-weather safety problems. Combining steep grades with sharp horizontal curvature creates a high-risk safety condition.

10.1.6.3 SIGHT DISTANCE

Sight distance is a function of the roadway alignment and cross section provided. Sight distance is the distance ahead for which a line of sight is provided for the driver under certain design conditions and assumptions. There are four types of sight distance—stopping sight distance, intersection sight distance, passing sight distance and decision sight distance. Stopping sight distance and intersection sight distance usually control the design.

Passing sight distance needs to be considered on two lane facilities as it affects operational quality and capacity but does not necessarily directly impact safety. Decision sight distance may be desirable in situations where there is a need to provide the driver with more time to make movement decisions.

Stopping sight distance criteria are based on operational models that reflect a driver's ability to see an object at given height above the roadway and react appropriately to avoid a collision. Sight distance can be limited by the vertical alignment of the road but also can be affected by a combination of horizontal alignment and sight obstructions beyond the edge of pavement. The model assumptions include size and placement of the object, the height of the driver's eye, driver perception time and braking reaction. The model applies to all levels of traffic volumes and all highway types equally. Since there are several assumptions that are affected by the evolution of the vehicle fleet, the age of the driving population and research on driver behavior, the range of acceptable stopping sight distance values is intended to be conservative. The model is not based on directly measurable safety values, the probability of a driver encountering the assumed conditions, or the severity of such an encounter. In other

words, the design values for stopping sight distance and vertical curvature are unrelated to direct measures of safety, and generally provide substantial margin of safety against the actual risk of a crash. Sag vertical curve values are based on night operations where the controlling feature is the headlight beam on the pavement ahead.

The designer can assess the risk of a location with limited stopping sight distance using the following guidance:

- The risk of a sight distance restriction is related to the traffic volume exposed to it.
- The risk of a sight restriction is greater where other features such as intersections, narrow bridges, high volume driveways, or sharp curvature occur within the sight restriction.
- Where no high-risk features exist within the sight restriction, nominal deficiencies may not create undue risk of increased accidents.
- Horizontal restrictions such as buildings, signs, tree lines, etc. affect all types of vehicles equally.
- When possible, designers should use shorter sag vertical curves in favor of providing the longest crest vertical curves possible.

Lower ranges for stopping sight distance may be applicable for reconstruction projects where speeds and volumes are observable, accident records are available and the constraints of providing the desirable value may be cost prohibitive and adversely impact adjacent land uses.

10.1.6.4 CROSS SECTION ELEMENTS

Cross section elements of a facility are the most visible and have the greatest physical affect on constructing or reconstructing a roadway. These include the lane and shoulder widths, median treatment, border areas, side slopes and ditch sections. In selecting these elements one must consider traffic volume, traffic mix, transportation modes, functional classification, available right of way and adjacent land use. Cross sectional elements become increasingly important as the severity of the alignment increases. Traffic operation and safety are very closely related to lane widths, shoulder availability, shoulder width and shoulder surface type.

The use of wider lane widths should be considered on higher speed facilities or those with high truck volumes. Lane widths are also influenced by horizontal curvature, as vehicles tend to move off-track and require room to avoid on-coming vehicles. In urban areas, narrower lanes may be considered as speeds are lower and the driver expects to encounter normal congestion and slower travel speeds.

Shoulders are a very important element of the roadway cross section. The shoulder serves as a part of the clear zone, improves capacity, accommodates drainage, provides an area for disabled vehicles, enables collision avoidance maneuvers, provides structural support for the traveled way and accommodates bicyclists and pedestrians. Where a full shoulder can not be provided, the designer should attempt to provide as wide a shoulder as possible that meets functional requirements.

The area adjacent to the roadway plays a major role in a facility's safety particularly with respect to run-off the road accidents. In applying context sensitive design philosophies, treatment of this area is affected by the established clear zone, costs, available right-of-way and probable impact on safety. Depending upon the roadside design provided, encroachment may not be a problem. When designing a project, overall safety benefits may be achieved by providing spot roadside improvements that may not necessarily be applicable throughout the length of a project.

Ideally a facility's roadside area should be a wide, firm, flat, hazard-free area that will permit the errant driver to recover and safely return to the highway. Early in the design process the designer normally establishes an area outside the travelway (edge of running lane) termed "clear zone." The effort and cost required in providing this area varies significantly from location to location. Deciding how much clear area to provide is proportional to the design speed, the projected traffic volume, historical accident data and the cost.

The most serious and obvious hazards are fixed objects that when impacted produce a sudden or instantaneous deceleration. These would include man-made objects, such as utility poles, overhead sign structures, buildings, retaining walls, large drainage inlets and outlets, exposed headwalls, and utility and traffic control boxes. Some objects only present a serious hazard at high-speeds while others are potentially serious hazards at any speed. The latter may require a more durable and effective barrier system.

The most hazardous natural fixed objects are trees. In context sensitive design, rather than clear the entire right-of-way, a designer needs to identify those trees most likely to present a significant hazard to the driver. Tree removal is a sensitive environmental and community issue. This perception must be considered when identifying trees to be removed. Where possible an analysis of accident history and visual evidence may indicate if there is a tree safety problem. If there are numerous trees, removing selective trees may not improve safety. Isolated trees lying close to the roadway present the most probable safety risk and should be removed or protected. Community interest, speed, volumes, and past operational history would all affect this decision.

Natural and created topography does play an important role in roadside design. Created topographical features enter into the design of a safe roadside throughout the state. These include steep backslopes in cut areas, steep sideslopes in embankment areas and unsafe median and ditch sections. The goal is to minimize the need and use of unsafe road sections when designing these elements.

Most projects involve some type of improvement to an existing facility. Fixed objects encountered can include utility poles, sign and lighting supports, mailboxes, fences, gates, commercial signs, unsafe rock outcroppings, bridge abutments and piers. In creating a safe roadside environment, the designer has five options to address fixed objects: (1) removal, (2) relocation, (3) modification, (4) shielding and (5) delineation. Although it may not be practical or cost-effective to treat all obstructions, studying and mitigating (where possible), each obstruction should be viewed as an opportunity to improve roadside safety.

The concept of a wide hazard-free clear zone is usually not practical in urban conditions where many roadside features come into play, such as pedestrian areas, utilities, streetlights, signs, ornamental structures, benches, on-street parking, and limited right-of-way.

Roadside safety should not be compromised in context sensitive design. To reach this goal the designer should:

- Avoid setting an artificially high or low design speed;
- Apply a consistent roadside treatment;
- Not be arbitrary in selecting a clear zone;
- Encourage the removal or relocation of fixed obstacles;
- Use safe landscaping and grading treatments; and
- Provide safe sight distances at intersections and all planted areas.

On multi-lane highways the median treatment is an important part of cross

section design. The primary purpose of a median is to separate traffic, thus reducing the occurrence of head-on collisions. However, there are other benefits associated with medians that improve overall safety, traffic flow (vehicular and pedestrian) and aesthetics. The median may be raised, depressed or flat, and includes paved or unpaved shoulders, swales, or ditches. Medians also offer the opportunity for providing landscaping. Median width selection is a function of the available right-of-way, the border area needs and providing area to allow safe maneuvers and room for traffic devices. Median width meeting one or more of these needs has a large range of values. Wide medians are not always attainable but even narrow medians can provide positive benefits.

10.1.6.5 INTERSECTIONS

Most projects will involve the treatment of intersecting roadways. Intersection design is important to a facility's efficiency, safety, speed, capacity, and user cost. Safe intersection design includes the following:

- Provision for adequate sight distance;
- Accommodation of appropriate traffic control devices;
- Provision of safe and efficient handling of turning traffic (left turns have the most conflicts);
- Avoidance of non-standard or confusing geometry, signing or other out of the ordinary treatments prior to or within the intersection; and
- Optimization of the capacity of the intersection.

The design features of an intersection are controlled by selection of the design vehicle, the larger the design vehicle the larger the intersection. Since space is limited, it may be better to accommodate the design vehicle that is anticipated to use the intersection most frequently. The available turning path templates were developed using relatively conservative dimensions. Computer programs are available to help the designer test the tracking of a design vehicle. When constraints do not permit meeting the full geometry and lane widths of the design vehicle, consideration should be given to moving roadside objects, such as signal poles, highway lighting, signs, etc. farther from the edge of pavement. When not providing the full standards, it can be expected that there will be occasional encroachment outside the designated lane area. These areas should be evaluated for type of curb, particularly if there are fixed objects. In addition, aesthetic and traffic calming improvements should not be placed in anticipation of encroachment. If the intersection can not accommodate the desired turning lane arrangements, different traffic control methods may be used including turn prohibitions, special signal phasing, advanced signing or other measures.

10.1.7 MAINTAINABILITY

All designs must be continually evaluated for maintenance needs. The frequency, ease, cost, accessibility and safety of maintaining selected design elements are essential to measuring a successful project. Much of the design effort, particularly for context sensitive projects that use innovative and unique concepts or construction materials, can be negated if they can not be properly maintained.

10.2 TRAFFIC CALMING

In communities where speeding and/or cut-through traffic has been identified as a problem, it may be appropriate to consider the feasibility of one or more traffic calming measures. Such measures are intended to encourage adherence to posted speeds and/or discourage inappropriate routing of non-residential or non-local traffic through communities.

The DelDOT *Traffic Calming Design Manual* (TCDM) and the joint FHWA-ITE *Traffic Calming-1999* publication provide general guidance regarding the appropriate use, design, signing and marking the traffic calming measures approved for use in Delaware. It also describes the steps necessary to nominate, select, develop and implement traffic calming projects. In the event it is necessary to modify or remove an existing traffic calming measure, the TCDM provides guidance in that matter as well.

10.3 TRAFFIC BARRIERS

The purpose of traffic barriers is to reduce fatalities and injuries by preventing a vehicle from leaving the traveled way and striking a fixed object or terrain feature. However, it is recognized that a protective barrier is, in itself, a roadside obstacle and may not contribute to safety. Therefore, under most situations a barrier is warranted only when the consequences of leaving the roadway are likely to be more severe than a collision with the traffic barrier.

Other factors not to be overlooked by the designer in determining the need for barriers include an analysis of a project's accident history, types of accidents, adjacent land uses, pedestrian and bicyclist use, and adverse geometrics such as sharp curvature combined with poor sight distance.

Barrier performance is based on its ability to contain and/or redirect the errant vehicle. The behavior of a vehicle during impact is very complex. Barrier designs are developed through full-scale crash tests under controlled standardized conditions. Sections 5.1 and 5.2 of the *Roadside Design Guide* provide the details on the testing programs.

Because of the on-going testing programs, the available types of barriers and installation details for barriers are dynamic. The Department's Standard Construction Details, the Standard Specifications, and Special Provisions are continually modified to reflect changes recommended by AASHTO, FHWA, the Transportation Research Board and other agencies that oversee the barrier testing programs. The testing results are published and recommendations made to transportation agencies for selecting approved barrier types based upon the anticipated service needs.

The three most commonly used barriers discussed in this section are:

- Longitudinal barriers,
- Median barriers, and
- Impact attenuators.

10.3.1 DESIGN OPTIONS

A designer's goal is to develop an acceptable design that does not warrant barriers. The designer has several options after an initial determination has been made that a traffic barrier is warranted. Design options for the treatment of the various conditions encountered within a project's limits should be evaluated in the following order:

- Examine the proposed and existing roadside features to determine the feasibility of eliminating the need for a barrier;
- Remove the obstacle or redesign it so it can be safely traversed, that is, design traversable slopes, extend pipes beyond the clear zone, install safety end sections, etc.;
- Relocate the obstacle to a point where it is less likely to be struck;
- Reduce impact severity by using an appropriate breakaway device;
- Redirect a vehicle by shielding the obstacle with a longitudinal traffic barrier and/or crash cushion; and
- Delineate the obstacle if the above alternatives are not appropriate.

10.3.2 GUIDELINES

The criteria and procedures for barrier need and design are outlined in the AASHTO *Roadside Design Guide*. Using the charts and graphs found in the guide in determining the need for barrier is relatively simple. However, this method does not consider the probability of an accident or the costs of either shielding the condition or leaving it as-is. A method has been developed that establishes warrants based on a cost-benefit analysis that considers factors such as design speed and traffic volume. The guide includes a computer program for evaluating the costs and benefits of design options for safe treatment of roadside hazards. Performing this analysis may be a useful tool for determining the preferred safety treatment.

The following are important elements to consider in determining the need for and selection of types of barriers:

- Performance capability,
- Deflection,
- Site conditions,
- Compatibility with other design features,
- Cost,
- Maintenance (routine and collision, storage of spare parts, and simplicity of repair),
- Aesthetics, and
- Past performance.

10.3.3 LONGITUDINAL BARRIERS

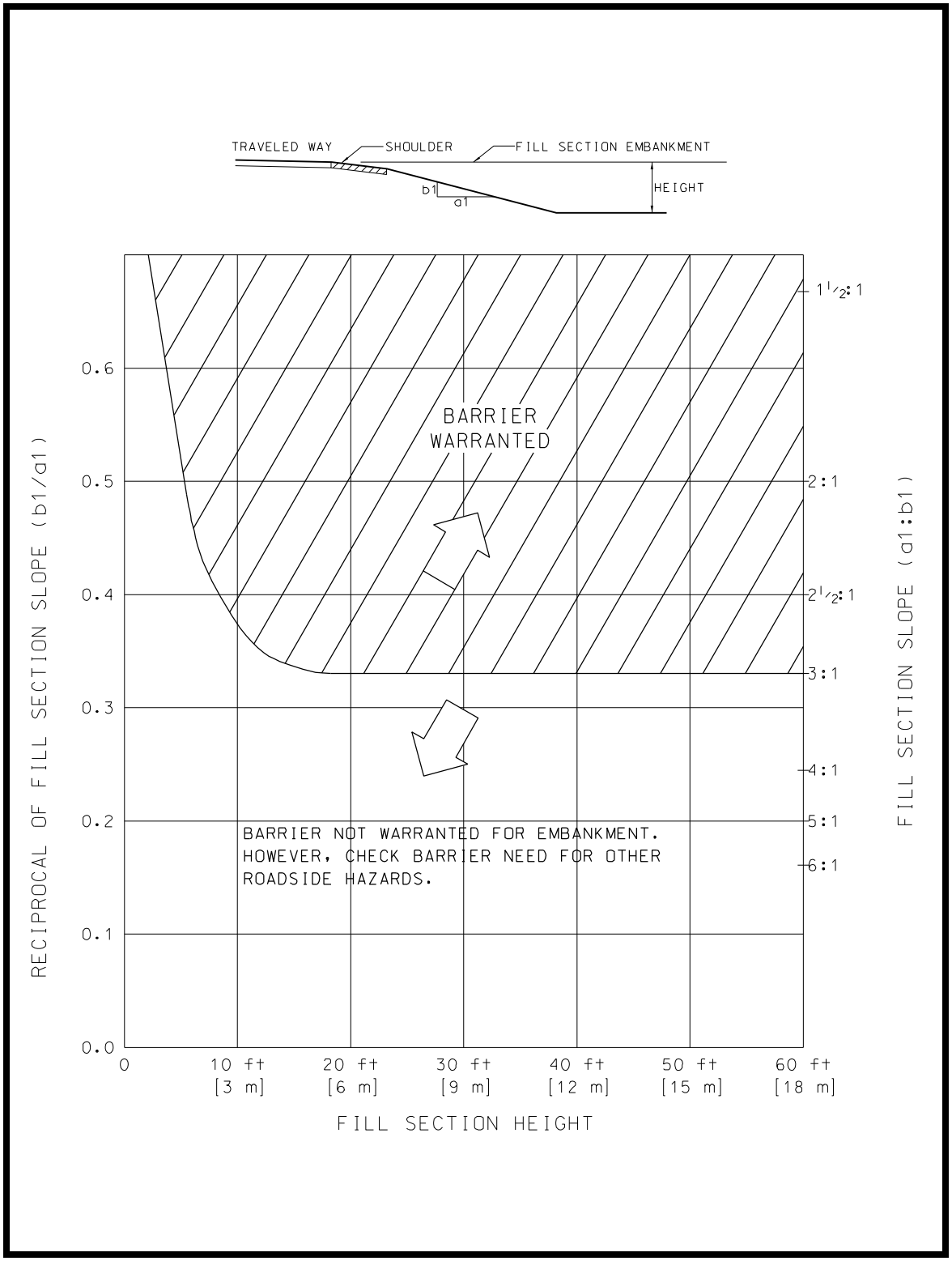
W-beam steel guardrail and concrete safety shape barriers are most commonly used for longitudinal shielding. Geometric criteria for steel beam guardrail, along with illustrative typical installations, are in the Department's Standard Construction Details. The need for barriers is directly related to the selected cross sectional elements as discussed in Chapter Four. This section is directed principally to guidelines and general considerations for designing guardrail installations.

Hazards that may warrant shielding by a roadside barrier can be placed in one of two basic categories, embankments or roadside obstacles. Protecting pedestrians and cyclists from vehicular traffic with a barrier may also be warranted.

ADT, embankment heights and slope rates are the primary factors in determining barrier needs for fill sections. Figure 10-1 shows guardrail warrants for various combinations of embankment height and slope. A barrier installation is warranted where the point of intersection falls above the curve.

Side slopes on an embankment should provide a reasonable opportunity for recovery of an out-of-control vehicle. Flat slopes characterize a traversable embankment and generous rounding of the slope breaks. In instances where construction of flat slopes is not feasible or practical, installing a barrier may be necessary. Where slopes are 3:1 or steeper, designers should first consider the alternative of the adjusting the embankment slopes, thus avoiding the use of barrier.

Figure 10-1
Guardrail Warrants for Embankments



Roadside obstacles are classified as "non-traversable hazards" and "fixed objects". If it is not possible to remove or relocate a hazard, then guardrail may be necessary. However, guardrail should be installed only if it is clear that a barrier offers the least hazard potential and is the most cost-effective in terms of safety benefits.

In the case of non-traversable hazards, a general criterion is that barrier protection should be provided for streams or bodies of water that may constitute a hazard and areas of rough rock cuts.

Warrants for barriers to shield roadside obstacles involve different considerations. A principal consideration is the required clear zone width. Clear zone selection is discussed in Chapter Four. Barriers are normally not required to shield a fixed object outside the specified clear zone. Roadside obstacles in the form of fixed objects include items such as trees, bridge piers and abutments, culvert headwalls, sign posts, light posts, and non-traversable drainage structures. In the case of drainage structures, the designer should consider extending pipes or using traversable end treatments to eliminate the hazard. Guardrail shielding normally is warranted if obstacles or fixed objects are within the limits of the lateral clear zone. Accepted designs of "breakaway types" of poles and posts are not considered fixed objects.

The need for guardrail at a bridge approach is based on the clear zone requirements for fixed hazards. For twin bridges, the length of approach rail on the median side of each bridge should be long enough to prevent an errant vehicle from impacting the bridge rail end of the other bridge. If it is within, or close to, the design clear zone, the guardrail should be long enough to protect the area between bridges at the edge of the clear zone. Consideration should be given to including a transverse berm between the endwalls of the two bridges.

Special treatment is required where side entrances interrupt the installation of guardrail. Additional flares and end treatments may be required, or it may be more economical to eliminate the need for guardrail.

An area of concern is what has been termed the "innocent bystander" problem. In most of these cases, the conventional criteria presented above cannot be used to establish barrier needs. For example, a major roadway or street may adjoin a schoolyard, but the boundaries are outside the clear zone. Since it is outside the clear zone, a barrier would not normally be required. However, if there is a reasonable probability of an errant vehicle encroaching on the schoolyard, a barrier could be considered and installed.

Protection of pedestrians and cyclists is another area of concern. As in the case of bystander warrants, there are no objective criteria for pedestrian and cyclist barrier warrants. On low-speed streets, a vertical curb may be sufficient to delineate pedestrians and cyclists from vehicular traffic. However, at speeds over 25 mph [40 km/h], a vehicle may mount the curb for relatively flat approach angles. Particularly where sidewalks or bicycle paths are adjacent to the traveled way of high-speed facilities, some additional provision may be required for the safety of pedestrians and cyclists.

10.3.4 BARRIER PLACEMENT

After the decision is made that a barrier is warranted, there are several factors that need to be considered for its placement. These are:

- Lateral offset from the edge-of-traveled way,
- Terrain effects,
- Flare rate, and
- Length of need.

10.3.4.1 LATERAL OFFSET

A roadside barrier is normally placed as far from the traveled way as possible, while still maintaining the operating characteristics of the selected type of barrier. The greater the distance the better chance for the driver to recover control of the vehicle. In addition, some barrier installations may obscure a driver's sight distance if placed too close to the traveled way.

Placing the barrier at a uniform offset distance will not only be more aesthetically pleasing but provides the driver with a feeling of security and comfort when approaching a series of protected obstruction areas. The distance from the edge-of-traveled-way, beyond which a roadside object will not be perceived as an obstacle and result in the driver reducing speed or directing the vehicle away from the barrier, is called the shy line. This theoretical distance is different based on design speed and is shown in Table 5.5 in the *Roadside Design Guide*. For long continuous runs of barrier and barrier placed beyond the shoulder, the shy line offset criterion usually does not control barrier placement.

Another consideration in the lateral placement of barrier is the expected deflection of the system selected. The distance from the barrier to a rigid obstruction should not be greater than the dynamic deflection of the system based upon data from actual impact tests under controlled conditions, i.e. vehicle weight, speed and impact angle.

In embankment areas that must be protected, it is important that the width of embankment be sufficient to adequately support the posts to ensure proper operational characteristics of the barrier, see the Standard Construction Details.

There may be considerable deflection in barriers when impacted by a vehicle. Figure 10-2 shows the dynamic deflection of W-beam and concrete safety shape. If a roadside obstacle is too close to the back face of the rail or post, there may be danger that the rail will deflect all the way to the obstacle. Under these conditions, designing the post spacing closer than normal will reduce the potential deflection of the guardrail. If the obstacle is more than 3 ft [0.9 m] behind the back of post, a post spacing of 6 ft 3 in [1905 mm] should be used.

For obstacles located from 2 to 3 feet [0.6 to 0.9 m] behind the back of post, a post spacing of 3-ft 1-1/2 in [952 mm] should be used. If the obstacle is less than 2 ft [0.6 m], a rigid concrete barrier could be used. These deflection guidelines are based on having a proper end anchorage and posts installed in stable soil.

10.3.4.2 TERRIAN EFFECTS

A roadway's cross section is important element in a vehicle's performance when approaching or impacting a barrier. Barrier systems perform best when vehicles have all wheels on the surface and its suspension is in a normal position at the point of impact. The two common features of concern are curb and the approach slope. These features may cause a vehicle to vault over a barrier or strike the barrier too high or too low.

Vehicles striking curbs can change trajectory depending upon the size of vehicle, suspension characteristics of the vehicle, its impact speed and angle, and the height and shape of the curb. Impact testing has shown that the use of any guardrail/curb combination where high-speed, high-angle impacts is not an acceptable practice. If there is not other alternative, than the curb should be limited to 4 in. [100 mm] and the guardrail stiffened.

Vehicles traversing slopes steeper than 10:1, depending upon their impact angle and speed, may go over or impact the selected barrier too low. The *Roadside Design Guide* in section 5.6.22 and Figure 5.22 details the effect of slope rate on vehicle and barrier

performance. The conclusion of the discussion is that roadside barriers perform most effectively when they are installed on slopes of 10:1 or flatter. A slope of 6:1 may be a problem and Figure 5.23 in the *Roadside Design Guide* gives a recommended barrier location when using 6:1 slopes.

Figure 10-2
Dynamic Barrier Deflection

Barrier Type	Post Spacing	Dynamic Deflection (From back of post)
Blocked-out W-Beam Steel Post	6 feet 3 inches [1905 mm]	3 feet [0.9 m]
Blocked-out W-Beam Steel Post	3 feet 1-1/2 inches [952 mm]	2 feet [0.6 m]
Concrete Safety Shape	Continuous	0 feet [0 m]

10.3.4.3 FLARE RATE

A barrier may be introduced by offsetting the beginning of the installation farther away from the traveled way than the normal offset. This allows the terminal section to be located farther away, minimizes the driver's reaction to having an obstacle close to the road, transitions the barrier to an obstacle nearer the roadway or reduces the total length of barrier needed.

There are disadvantages to flaring barriers. The greater the flare rate, the higher angle at which a barrier can be hit increasing the severity of crashes. A flared installation may increase the possibly of a vehicle being redirected back into or across the roadway.

The suggested flare rates for barrier design are shown in Table 5.7 of the *Roadside Design Guide.*

10.3.4.4 LENGTH OF NEED

The length of need *(X)* depends on the runout length *(L_R)*, the lateral extent of the area of concern (hazard) *(L_A)*, the flare rate for the tapered section *(b/a)* and the distance from the edge of traveled way to the face of barrier *(L_2)*. Guardrail is normally placed 1-foot (0.3 m), or more, beyond the shoulder. The runout length *(L_R)* is the theoretical distance needed from the edge of traveled way to the hazard measured along the edge of the pavement. The lateral extent (L_A) of the hazard is measured perpendicularly from the edge of the traveled way to the far side of the hazard or to the clear zone. The variables are shown in Figure 10-3**.** The tangent length of barrier immediately upstream from the area of concern, L_1, is a variable length selected by the designer. Runout lengths *(L_R)* for the various design speeds and traffic volumes are shown in Table 5.8 of the *Roadside Design Guide.*

The total length of barrier, without the end treatment, can be calculated with the following equation:

$$X = \frac{L_A + (b/a)(L_1) - L_2}{(b/a) + (L_A / L_R)}$$

Where,

L_A = the perpendicular distance from the edge of the traveled way to the far side of the hazard, if the hazard is a fixed object or to the outside edge of the clear zone.

L_2 = the distance from the edge of traveled way to the face of the barrier at the location where the rail is parallel to the roadway.

b/a = the flare rate.

L_1 = the length of the tangent section ahead of the hazard.

L_3 = edge of traveled way to front of hazard. L_3 equals, as a minimum, the shoulder width plus the 2 ft [0.6 m] offset for the barrier plus the allowance for dynamic deflection.

Note that for a parallel installation, i.e., no flare rate, the equation reduces to:

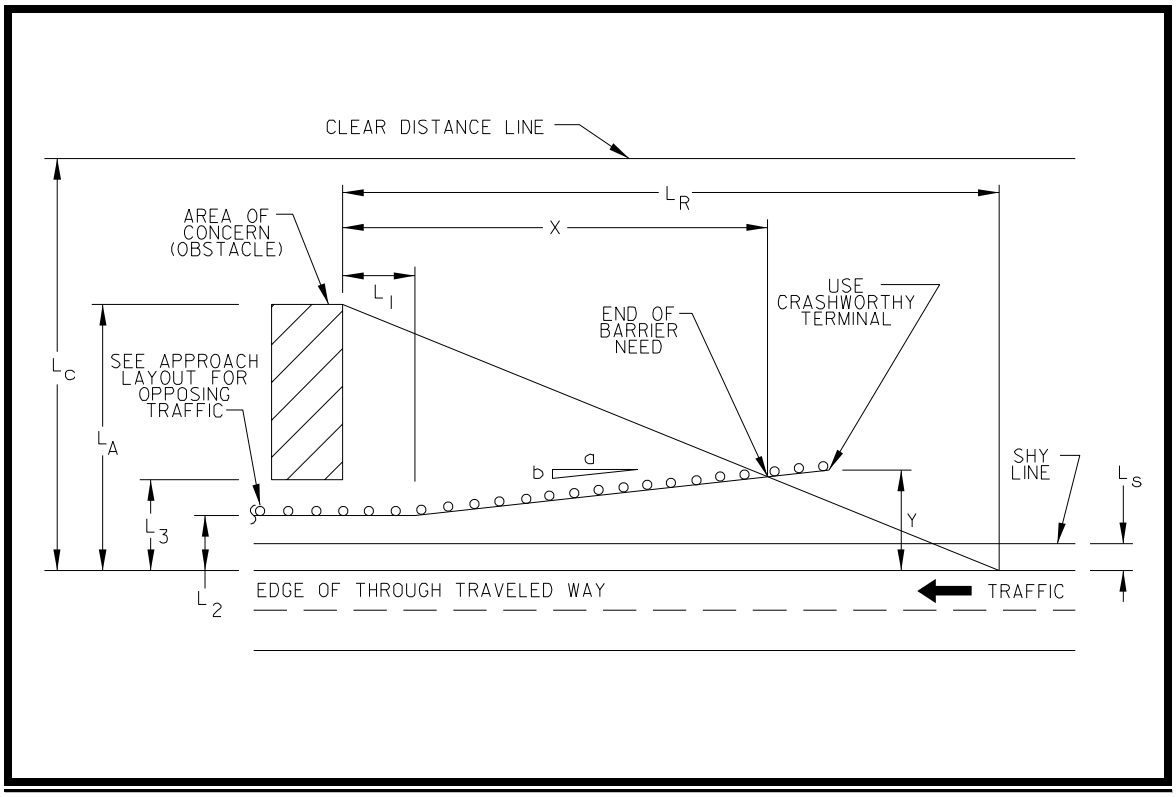
$$X = \frac{L_A - L_2}{L_A / L_R}$$

The lateral offset, Y, from the edge of the traveled way to the beginning of the length of need, can be calculated using the following equation:

$$Y = L_A - \frac{L_A}{L_R}(X)$$

The value of L_1, the length of the tangent section ahead of the hazard, is a variable length selected by the designer. If a semi-rigid railing is connected to a rigid barrier, the tangent length should be at least as long as the transition section. This reduces the possibility of pocketing at the transition and to increase the likelihood of smooth redirection if the guardrail is struck immediately adjacent to the rigid barrier. To determine the length of the transition section, see the Department's Standard Construction Details.

Figure 10-3
Approach Barrier Layout Variables



The final variable for determining the required length of guardrail is the flare rate. Suggested flare rates for various design speeds are shown in Table 5.7 of the *Roadside Design Guide.*

Slopes must be 10:1 or flatter in front of the barrier. It is desirable to place a barrier as far from the edge of the traveled way as possible. Because this may involve a combination of different slope rates, the designer should refer to the *Roadside Design Guide* for proper section and slope treatment.

The shy line offset (L_s) is the distance beyond which a roadside object will be perceived as non-hazardous and does not result in motorists reducing speed or changing vehicle positions on the roadway. This distance varies for different design speeds as indicated in Table 5.5 of the *Roadside Design Guide.* If possible, a roadside barrier should be placed beyond (outside) the shy line offset, particularly for relatively short or isolated installations. For long, continuous runs of barrier this offset distance is not as critical, especially if the barrier is first introduced beyond the shy line offset and gradually transitioned closer to the roadway. Note that flatter flare rates are suggested when the barrier must be placed inside the shy line.

The lengths of barriers can be determined by plotting the barrier layout directly on the plan sheets. By selecting an appropriate runout length and the lateral distance to be shielded, the designer can develop a guardrail installation that satisfies all placement criteria.

This method is most appropriate for determining the length of barrier needed to shield embankments or hazards on curved sections of roadways. It should be noted that the portion of the acceptable end treatment that has full effectiveness can be included in the length of need, (*X)*. The *Roadside Design Guide* provides several figures and examples of procedures for determining the length of need for typical situations.

10.3.4.5 APPROACH BARRIERS FOR OPPOSING TRAFFIC

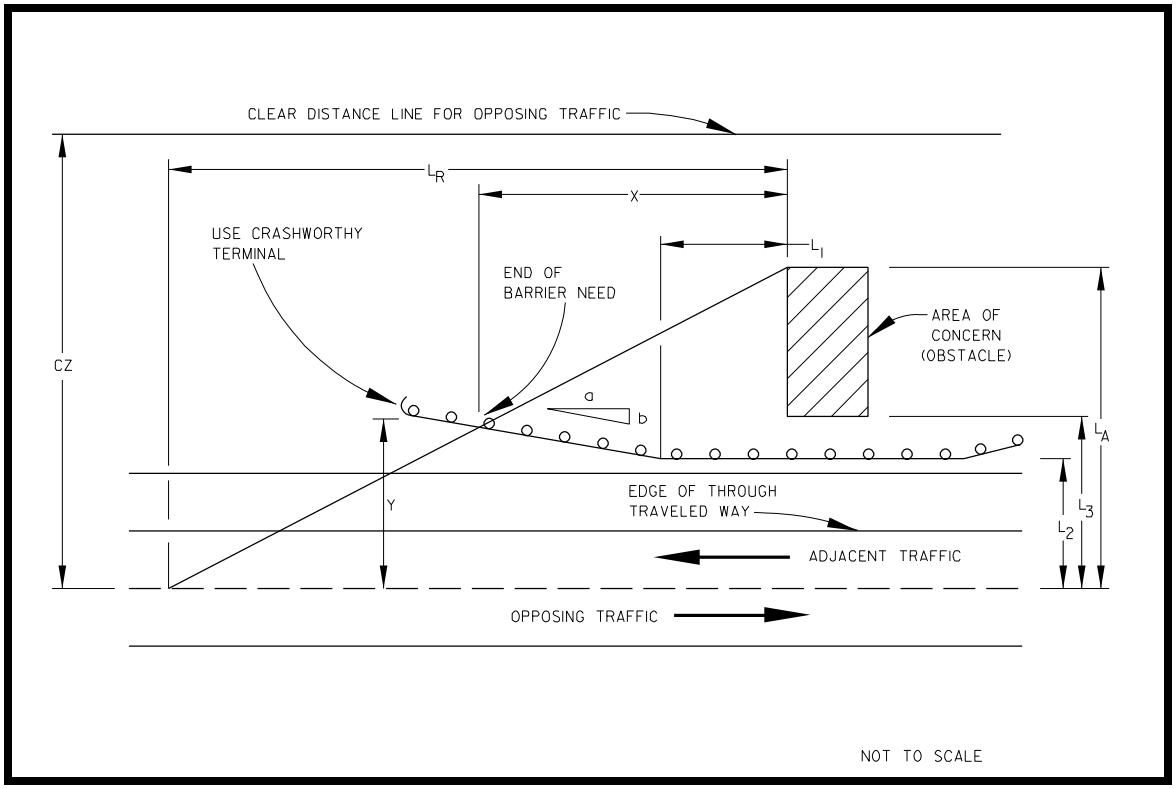
On roadways with two-way traffic, the length of need must be checked from both directions. All lateral dimensions for opposing traffic would then be measured from the centerline of the roadway. Figure 10-4 illustrates the layout variables for an approach barrier for opposing traffic. The length of need and the end of the barrier are determined in the same manner as previously described.

10.3.4.6 ROADSIDE SLOPES FOR APPROACH BARRIERS

The embankment where the flared section is to be placed should have a slope of 10:1 or flatter to prevent errant vehicles from striking the barrier either too high or too low for the barrier to be effective. If the slope is steeper than 10:1, the slope should be flattened so that the embankment criterion is not violated. See Figure 5.28 of the *Roadside Design Guide* for the suggested roadside slopes for approach barriers.

Details of typical guardrail installations at bridge ends are shown in the Department's Standard Construction Details. These sheets also show various applications of guardrail installations.

Figure 10-4
Approach Barrier Layout for Opposing Traffic



10.3.5 MEDIAN BARRIERS

The basic function of median barriers is to prevent out-of-control vehicles from crossing the median and entering opposing lanes. Effective median barriers should be installed on all high-volume, high-speed divided highways with medians where engineering studies establish a need. Figure 10-5 suggests warrants for median barriers on high-speed divided highways that have relatively flat, traversable medians. These criteria are based on a limited analysis of median crossover accidents and research studies and are suggested for use in the absence of site-specific data.

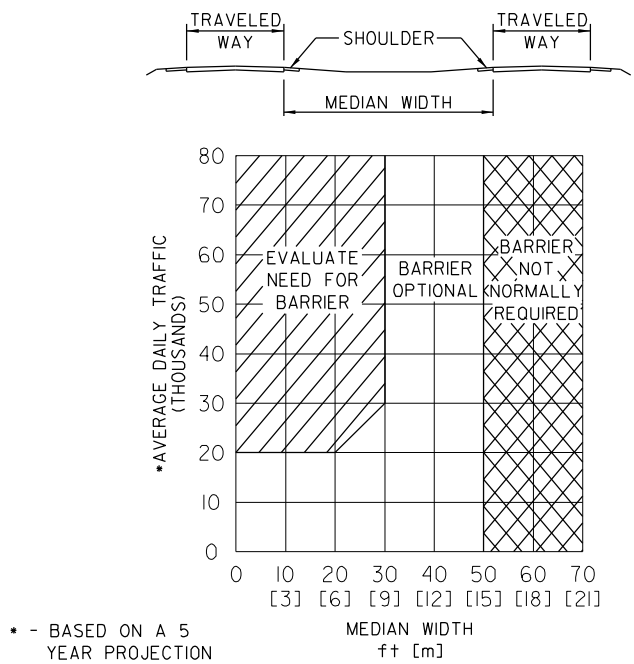
On divided highways, obstacles in the median should be treated the same as obstacles outside the roadway. If the slope is traversable and there are no other obstacles, guardrails on the exit side can be terminated opposite the hazard with a standard guardrail terminal. It need not be flared or have crashworthy end treatment unless it is in, or close to, the clear zone for traffic in the opposing lanes.

Barriers are typically considered for combinations of average daily traffic (ADT) and median widths that fall within the darkened area. In the "Barrier Optional" and "Barrier Not Normally Considered" area of Figure 10-5, a barrier is warranted only if there has been a history of cross-median accidents. When designing for the installation of a median barrier the median must be at least 10 ft [3 m] wide to provide adequate horizontal clearance between the barrier and the edges of traffic lanes.

The most common types of median barriers used in Delaware are the blocked-out W-beam guardrail and the concrete safety barrier. Concrete safety barrier is preferred in narrow medians where regular

maintenance is difficult, or where deflection of the barrier would affect opposing traffic. Temporary concrete median barriers are used for traffic control in construction areas. They are placed to separate opposing lanes of traffic on detours and to separate work areas from traffic.

Figure 10-5
Median Barrier Warrants for High-Speed Divided Highways



10.3.6 IMPACT ATTENUATORS

Impact attenuators or crash cushions are designed to prevent vehicles from impacting a fixed object by redirecting a vehicle if impacted from the side and stopping the vehicle at a rate of deceleration that is tolerable to the vehicle occupants when impacting head-on. The two most commonly used concepts used are absorption of the kinetic energy or transfer of momentum.

When designing new facilities the need for these devices should be limited to gore areas, principally on elevated structures at ramp exits.. For other projects they may be the most logical choice to shield the ends of median barriers or longitudinal barriers without crashworthy end terminals. Impact attenuators must be properly installed to provide the desired performance. The approach grading must be flat and the attenuator placed level with the approach ground.

The type of the impact attenuator depends on the expected impact speed and width of the fixed obstacle. Manufacturers provide charts that can be used to determine the appropriate device for the specific location.

The selection of a particular impact attenuator depends on several factors including cost, site compatibility, periodic maintenance needs, extent and ease of maintenance after impact, anticipated performance characteristics, and structural effect of impacts. For additional information and guidance, including the design of the size of the device for the proposed impact speed, refer to Chapter Eight in AASHTO's *Roadside Design Guide* and the manufacturer's literature. It should be emphasized that in frequent impact locations the impact attenuator should be delineated to reduce the number of hits. At these locations a cost/benefit analysis may justify the additional expenditure for the installation of a self restoring attenuator.

10.4 CURBS

Curbs serve a variety of positive functions including drainage control, roadside delineation, reduction in right-of-way needs, aesthetics, reduction in maintenance operations, protection of pedestrian traffic, and control of existing and future roadside development. Even with all of these positive attributes, a curb remains a longitudinal fixed object that can cause loss of control and vaulting of errant vehicles. Therefore, curbs must be used sparingly and only after careful evaluation of other alternatives. In most cases, a curb is to be used only on urban facilities with limited right-of-way.

10.4.1 TYPES OF CURB

Curbs are described as either barrier or mountable. A barrier curb is any curb with a total vertical rise greater than 6 inches [150 mm] no matter what face or overall shape it may have. Therefore, mountable curbs are those with a vertical rise of 6 inches [150 mm] or less. Both barrier and mountable curbs may have a variety of cross sections. The Department's Standard Construction Details should be used in making the appropriate selection for the function to be served.

10.4.2 PLACEMENT OF CURB

The placement of curb has very serious effects on the behavior of drivers and vehicles. As indicated before, the vehicle can become uncontrollable and may even become airborne when a curb is hit. On rural facilities, the use of curb should be limited to channelization and delineation at intersections, crossovers, and entrances. On rural highways mountable type curb is preferred. On urban highways, curb use, while not preferred, is very common. On urban highways, mountable curb should be used for design speeds 50 mph [80 km/h] and above. Barrier curb may be used with design speeds of 45 mph [70 km/h] or less.

The typical effect on most drivers is to shy away from a curb because it is perceived as a danger and visually restricts the travel lane width. This poses the danger of vehicles shying into adjacent lanes. Barrier curbs should be offset 2 ft [0.6 m] from travel lanes and mountable curb offset 1 ft [0.3 m] to minimize this effect. The introduction of curb in itself poses a possible serious hazard and distraction to the driver. Newly introduced curbs should be offset and flared at least 2 ft [0.6 m] beyond the normal offset. In addition, the forward edge (facing traffic) of barrier curb should be gradually depressed at a 12:1 ratio until flush with the pavement or adjacent area. This type of installation will provide a safe end treatment for the driver and not interfere with snow plowing operations.

Curbs used for island channelization should be mountable and offset from the required pavement width by 1 ft [0.3 m]. The islands are to be located in order that they will not, in any way, restrict the travel lane or shoulder area.

The use of guardrail/curb combination should be discouraged where high-speed, high-angle impacts are likely. Where guardrail is used in conjunction with curb, the following shall be adhered to:

1. For design speeds 50 mph [80 km/h] or greater, guardrail shall be offset 10 ft [3 m] minimum from the front face of curb to the front face of guardrail.
2. For design speeds of less than 50 mph [80 km/h], guardrail shall be offset 6 ft [1.8 m] minimum from the front face of curb to the front face of guardrail.
3. Curb height shall be a maximum of 4 inches [100 mm] when curb is placed at the face of guardrail for all design speeds.

10.4.2.1 CURBS AT DEVELOPMENT ENTRANCES

If it is deemed necessary that a center island must be built, then the island should have mountable curb at a development entrance. Curb used in developments should be transitioned to the appropriate type of curb at the radius point where joining state right-of-way. The Department's manuals entitled *Standards and Regulations for Access to State Highways* and *Rules and Regulations for Subdivision Streets* should be referred to in making curb use determination.

10.4.2.2 CURBS AT COMMERCIAL ENTRANCES

The Department's *Standards and Regulations for Access to State Highways* and *Rules and Regulations for Subdivision Streets* manuals should be referred to for detailed information on commercial entrance design. Commercial entrance designs are to be coordinated with the Subdivision Section. In general, due to traffic volumes, it is normally desirable to have a curbed center island of the proper length at commercial entrances to separate traffic movements. The use of center islands will ensure that parking spaces or other internal traffic configurations will not be placed too close to the intersection with the state route. Allowing internal movements close to the main roadway could cause queuing onto the main roadway and substantial reduction in the operational efficiency of both the entrance and the roadway. The center-island should be built with mountable curb. The entrance radii and curb sections parallel to the state road should conform to those on the state road. Barrier curb should be used adjacent to sidewalks. The use or introduction of barrier curb adjacent to the higher-speed roadway is not recommended unless design consideration necessitates such use. For more detailed information see the Department's manuals entitled *Standards and Regulations for Access to State*

Highways and *Rules and Regulations for Subdivision Streets.*

10.4.3 ACCESS FOR THE DISABLED

For the disabled, it is the policy of the Department to provide accessible curb cuts and adjoining sidewalks at all crosswalk locations and at existing or planned mass transit service locations. The access consists of depressed ramps through curbs suitable for baby strollers, walkers, wheelchairs, etc. Such ramps are also to be provided in refuge islands where a crosswalk crosses a channelized island. Typical details of curb ramps are shown in the Department's Standard Construction Details.

10.5 RIGHT-OF-WAY

The designer is responsible for defining the limits of right-of-way necessary to construct, operate, and maintain the highway project. The following discussion is general in nature and the designer is to refer to Project Development Manual, for a detailed description for preparing right-of-way plans. In designing the project right of way, the designer must first identify actual construction limits from the cross sections to define the toe of fills or the top of cuts. Second, the designer must identify, through input from the Utilities Section, the reasonable real property needs of public or municipal utilities required to relocate facilities as a result of the proposed project.

When right of way needs are being defined, space for current and future bus stop improvements should be considered. Depending on the type of improvement to be installed, a width of up to 14 ft [4.3 m] may be needed (measured from back of curb). The minimum right-of-way needed at each stop whether existing or proposed is 8 ft [2.4 m].

The right of way limits should be of consistent width and wide enough to accommodate the construction limits and appropriate border areas necessary for ditching, utilities, and their maintenance. In urban or other highly developed areas where open ditching is inappropriate, desirable right-of-way border areas should be minimized to reduce negative impact on adjoining private property.

To minimize right-of-way design time and avoid plan changes, the designer should consult with personnel from the design support sections early in the design process, preferably at an on-site scoping meeting before completing the preliminary plans.

10.5.1 RIGHT-OF-WAY CONFIGURATION

Generally, right-of-way lines should be designed with as few breaks and changes as possible. The following general criteria should apply:

- Where there is need for a width change close to the P.C. or P.T. of a horizontal curve, the breaks should be made coincident with the P.C. or P.T. if possible.
- Where there is need for a change of width close to a property line between two different owners, the break in widths should be made at the property line. Where utilities exist or will be located adjacent to the right of way line, the right-of-way should be tapered rather than jogged, if possible. This will ease the installation of utilities along the alignment.
- Where width changes are required both right and left of the roadway, the break points should be made at the same station, if possible.
- Breaks in widths should not occur in drainage channels, roads or drives where installation of right of way monuments would be impractical.
- Where right-of-way may only be needed from a portion of a parcel, consideration

should be given to acquiring right of way through the entire parcel for future connecting projects, so that the owner will not have to be contacted again. This determination is made in coordination with other supporting sections.

- Do not define right-of-way lines with spiral curves. Where centerlines follow spiral curves, right-of-way lines should be described using a circular or compound circular curve of similar shape.
- Proposed right-of-way lines must be described in order for surveyors to lay them out in the field. Dimensions showing survey stations and pluses with offset distance right or left of construction centerline are required in addition to coordinates, bearings and distances.

10.5.2 EASEMENTS

Under certain conditions it is preferable to obtain an easement rather than to purchase right-of-way. There are two types of easements: (1) temporary and (2) permanent easements. The type of easement should be identified on the plans.

A temporary easement should be obtained where it is not necessary to obtain permanent possession of the land or permanent right of access to the land. Temporary easements are appropriate:

- For any areas where the Department will have no maintenance responsibility after the completion of the proposed project construction;
- Where relatively flat cut or fill slopes extend beyond the right-of-way line and the lateral clear zone or for grading purposes that may benefit the property;
- To obtain proper grade on private driveways and approaches;
- For channel changes and inlet and outlet ditches at drainage structures where future maintenance is not anticipated; and
- For construction working areas.

A permanent easement should be obtained where it is not necessary that the State own the land, but where perpetual interest is necessary. Examples are where the Department needs to access the property for future maintenance, repair, or replacement of the highway facility, its drainage systems or appurtenances and as provided for in a project's right of way or railroad agreement.

10.5.3 RIGHT-OF-WAY MONUMENTS

Right-of-way monuments may be placed to provide a permanent reference for re-establishing the centerline and right-of-way line. Right of way monuments should be located and punched so the center is on the right of way line. Details of a standard right-of-way monument are shown in the Department's Standard Construction Details.

10.6 FENCING

It is Department policy that installation of fences should normally be considered under one or more of the following conditions:

- For access restrictions on interstate or other designated controlled access highways,
- Replacement fence where an existing fence was removed because of right-of-way widening, or
- For locations where there is a documented need for fencing i.e. pedestrian and bicyclist safety or right-of-way negotiations.

The location of fencing depends on who will assume the ownership and maintenance responsibility, 1 ft [0.3 m] inside the right of way for DelDOT and 1 ft [0.3 m] outside if by others. The responsibility for installing fencing varies. Fencing required for

DelDOT purposes will normally be shown on the contract plans and included as a bid item for the contractor. The type of fencing will depend on the characteristics and use of the adjacent property.

Installation of replacement fencing or new fencing as the result of negotiating easements or property takings are normally included in the right-of-way settlement agreement. This agreement provides for the affected property owner to be responsible for construction and maintenance of any new fence, with installation reimbursement by the State. Occasionally, the right-of-way agreement will specify that this fencing be included as a contract item. Locations and quantities of fencing to be constructed in this manner are determined by Real Estate and coordinated with the designer.

10.7 UTILITY ADJUSTMENTS

This section is a general discussion of a project's relationship with utilities. The specific details are covered in Chapter 11, Plan Development. In addition, the Department has published the *Utilities Design Manual*, which clearly sets forth policies and procedures regarding the relationships among the Department, the public, and private utilities in Delaware. Much of the manual is related to the accommodation of utilities within the highway right-of-way and adjustments initiated by the utilities. Other sections define responsibilities and procedures related to needed utility adjustments resulting from proposed highway improvement projects. It is this second category that is of concern to designers.

In storm drain design, it is often possible to avoid conflicts with underground utilities by making minor adjustments in the line or grade of the storm drain. The designer should consider conflicts with any utility in making the final design to minimize relocations and conflicts. Relocations of utilities frequently delay the progress of construction and so should be avoided where possible. As further described in the manual, costs for relocating and adjusting utilities may either be the affected utility's or the Department's responsibility. The designer needs to recognize that no matter which party is responsible, the cost will be directly or indirectly passed on to the user.

The procedures and responsibilities for utilities adjustments set forth in the *Utilities Design Manual* are briefly summarized below in terms of the four phases of plan development.

10.7.1 SURVEY PLANS

The designer prepares survey plans showing the approximate project limits, existing detail, and project number(s) assigned to the project. Two sets of survey plans are provided for each utility and are transmitted by the Utilities Section.

The Utilities Section returns marked-up plans showing complete information on existing and abandoned facilities in the area. The information provided includes whether the facilities are aerial, surface or underground, sizes of pipes, numbers of conduits, approximate depths, and any private or commercial easements.

10.7.2 PRELIMINARY PLANS

The designer prepares preliminary plans showing the proposed alignment, profile, drainage, signal pole location, clear zones, right-of-way taking, existing utilities from survey plan data and other details. Two sets of plans are submitted by the Utilities Section to each potentially affected utility company, who reviews these plans and returns them showing their proposed work plan.

The designer, assigned utility coordinator and, in some cases, the utility company's representative review these plans to determine if the exact horizontal and vertical

location of certain facilities are an important part of the design considerations. Where major conflicts with underground utilities appear possible, the Utilities Section arranges for determining the exact field location of the existing underground installations, typically with taking test pits. The Utilities Section submits this information to the designer. The designer and utility coordinator determine which underground or aerial conflicts cannot be avoided and discuss alternatives with the affected utility company. It is very important that the potential impact on scheduling, current project estimate and the responsibility for preparing the design plans and quantities as well as the actual construction be identified at this time.

10.7.3 SEMI-FINAL PLANS

Semi-final plans are prepared showing the final centerline, profile, drainage, right of way and other details.

Two sets of semi-final plans for each utility are provided to the Utilities Section. These plans are delivered to each affected utility. A representative of the Utilities Section coordinates a review of the project's affect on each utility's facilities.

The utilities plot their proposed underground relocation work on the plans, and the information is returned to the designer through the Utilities Section. The Utilities Section arranges with the utility for any needed relocation work and formal agreements. The scope and schedule for the work is included in the project's utility statement that is furnished to the designer for preparation of the P. S. and E. package.

10.7.4 P. S. AND E. PLANS

When the final plans are completed and the project is advertised, one set of final plans is forwarded to each utility. If it has not previously been notified to start the adjustment, the utility is directed by letter from the Utilities Section to order materials and begin making the changes or alterations to their facilities.

10.8 SIDEWALKS

10.8.1. GOALS AND OBJECTIVES

Sidewalks are an integral part of the Department's transportation infrastructure program. They facilitate and encourage safe and convenient pedestrian travel within communities and among different land uses. They provide safe and reasonable access to public transportation and other alternative modes of transportation, thereby helping alleviate vehicular traffic and reduce emissions. They also reinforce the Americans with Disabilities Act (ADA) by increasing the access opportunity for mobility impaired individuals.

The incorporation of sidewalks and pedestrian amenities also provides better land use and transportation connections, thus encouraging more trips on foot, improve access to transit, improving access to adjacent land uses and communities, conserving energy, and enhancing the Department's vision for multi-modal and inter-modal transportation systems.

In addition, by providing this transportation option, the installation of sidewalks can be an effective means in reducing automobile dependence and use. It will assist Delaware's mission toward cleaner air under the requirements of the Clean Air Act Amendments of 1990.

With the addition and installation of proper sidewalks and pedestrian amenities, safety, accessibility, ridership, and more favorable perceptions of public transit service can be increased. Communities will also be able to safely link to other land uses and transportation modes, resulting in better and more desirable neighborhoods and land development practices.

The need for sidewalks in urban or suburban areas is most noticeable at points of community and urban development that result in pedestrian concentrations near or along the state's or municipality's transportation system (roads, rail, air, and water transport facilities). Such examples include: public and private transportation depots, local businesses, industrial receiving and distributing plants, corporate centers, shopping centers, malls, schools, medical centers, religious centers, hotels and places of residence.

The design of sidewalks should reflect the community or context in which they are to be placed through the choice of materials or proximity to a traveled way. Buffer zones between curb and sidewalk are preferred to enhance pedestrian perception of safety and convenience.

It is the Department's goal to ensure that all efforts have been undertaken in determining the need and justification for installing, reconstructing, improving, requiring, or extending sidewalks for its transportation projects as well as for other initiating parties in public or private development.

10.8.2 REGULATORY REQUIREMENTS

The specific state law that governs the installation of sidewalks is Title 17, Section 132(f), of the Delaware Code. In essence, the Department is to reconstruct disturbed sidewalks or install new sidewalks when constructing in an urbanized area, if there is a demonstrated present need or a reasonably anticipated future need. The Project Development Process determines whether such need for sidewalks does or will exist for all or any part of a project. Before arriving at a decision as to the need for sidewalk construction, the Department consults with the affected county planning department, Department of Education and the local school district. Within incorporated municipalities, the Department has a town/city maintenance agreement in force that should be reviewed as to its affect on the decision to reconstruct or install sidewalks.

Since the original law was enacted, additional requirements have been initiated that affect the decision making process on when and where to construct sidewalks, the design and construction standards, and the review and field inspection procedure. The need for and encouraging the use of sidewalks are important parts of the Americans with Disabilities Act, the Clean Air Act, federal legislation funding transportation programs and the State's Long-Range Plan for Transportation. Sidewalks shall be determined as part of the Department's Project Development Process and scoped as part of the handoff prior to design initiation.

The interest of the disabled on state and federally funded projects are represented by the State's Architectural Accessibility Board. This board adopts standards and criteria to address service and accessibility for the disabled and handicapped and is the regulatory agency for ensuring compliance with all applicable standards and criteria during design and in construction.

The Department applies accessibility standards during the design and construction of transportation facilities based on project initiation, scope, and funding authorization. As a part of the initiation process, sidewalk facilities and connector points for new sidewalks, at, along, or between a community's public roadways and transit service are also considered to ensure that the elderly, disabled, and handicapped have access and use of our highway and transit facilities. The Project Design Checklist provides guidance for submission of plans to the Architectural Accessibility Board for review.

The appropriate guides to ensure conformance include: the current State of

Delaware Architectural Accessibility Standards; Part II of the Federal Register's Architectural and Transportation Barriers and Compliance Board (36 CFR Part 1191) - dated July 26, 1991; and Part IV of the Federal Register's Department of Transportation (49 CFR Parts 27, 37, 38) - dated September 6, 1991.

10.8.3 DESIGN APPROACH

Determining the need for sidewalks is a standard component of the Department's process to plan, design, construct, and reconstruct transportation projects and all multi-modal and inter-modal networks.

During the project development process, all projects are evaluated for consideration of the removal and/or relocation of roadside appurtenances, street furniture, vegetation, mailbox posts, landscaping, public or private utilities, poles, guardrail, and/or traffic signs in order to satisfy and encourage safe pedestrian movement. The installation of any obstruction directly within a sidewalk buffer strip (area between edge of curb or shoulder and edge of sidewalk facility) which prohibits or blocks safe pedestrian movement should be avoided. If prohibitive objects are placed directly within/along a sidewalk location, consideration should be given to relocating or repositioning that conflicting sidewalk section.

Certain circumstances such as geographic design, engineering, environmental, safety constraints, or the extent and scope of the project itself, may require sidewalks to be constructed along one side of a roadway, transit corridor, or land use area. For the most part, transportation projects involving a roadway or transit corridor should have sidewalks on both sides. Sidewalks may be omitted on one side of the street where that side clearly cannot be developed and where there are no existing or anticipated uses that would generate pedestrian trips on that side.

Because of legal and design criteria differences, sidewalks are not to be designed, signed, and accommodated for other than pedestrian use. Therefore, a sidewalk/pedestrian facility should not be identified, signed, or striped for some other transportation or recreational use (such as a bicycle). If shared facilities (such as both pedestrian and bike use) are desired, policies and design guidelines reflective of the joint use shall be followed.

The incorporation of sidewalks should be consistent with all other state, county, city, and/or town sidewalk policies, ordinances, or mandates, including the Department's Long-Range Transportation Plan. The Department's Division of Planning will ensure sidewalk provisions and requirements are coordinated with other transportation and land use projects in the state.

Future maintenance responsibility for sidewalks is an important consideration and should be clearly defined before installation.

10.8.4 GUIDELINES FOR ASSESSING THE NEED AND CRITERIA

Sidewalks are considered during the project development phase when evaluating a roadway or transit facility for construction, reconstruction or rehabilitation. The Department, another party or a different agency, regardless of any roadway or transit improvement may initiate sidewalks as independent projects. Therefore, sidewalks should be considered for all Department transportation networks unless it is specifically determined that:

- Sidewalk construction conflicts with public safety.
- Sidewalk cost and economic impact of construction is prohibitive in relation to the need, the benefits realized, or their probable use.
- Specific land use factors indicate there is no current or future need for sidewalk.

During the planning and project initiation phase, facilities for pedestrian movement

warranting sidewalk are considered for their impacts on project costs, right-of-way needs, additional environmental constraints, highway or bridge geometrics/standards, and public and legislative input. When planning, initiating, or recommending the location or relocation of transit service, the Department considers installing sidewalks as well as other forms of pedestrian accommodations to address safety and accessibility. More specifically, the location and environmental study report identifies the initial need for the construction, reconstruction, improvement, or extension of sidewalks.

When pedestrian facilities are provided, special needs of the young, elderly, disabled, and handicapped are to be considered. This may include extending a project's limits for reasonable distances to accommodate better access and safety.

Within the context of multi-modal and inter-modal systems (i.e. linking various land uses and destinations with accessible transportation systems), the location and provisional extent of pedestrian facilities are studied to logically connect their termini.

During the project development process, several agencies, and departments are involved ensuring reasonable consideration of pedestrian needs are met. Where appropriate, these groups help assist in identifying, justifying, securing funding and determining the extent of the sidewalk and pedestrian amenities.

Representatives from the Department of Transportation include the Bicycle and Pedestrian Coordinator, the appropriate District Engineer, the Traffic Engineering and Management Section and the Delaware Transit Corporation. Representatives from outside DelDOT include the Architectural Accessibility Board, county and local planning agencies, the appropriate Metropolitan Planning Organization, the local school district affected by the project, and the Department of Education Transportation Supervisor.

The public and other entities involved in pedestrian and transportation safety may identify and suggest sidewalk and pedestrian accommodations. This is usually realized during or after a project's public workshop/hearing. The local citizenry may also identify problems overlooked.

Projects being initiated or reviewed by the Department consider several factors in determining the appropriateness of constructing, reconstructing, extending, or requiring sidewalks. The following is a discussion of these factors.

Existing and expected land use patterns, growth areas, and generators of pedestrian movements are considered as follows:

- Land use—residential (high/cluster, medium, low), business/commercial, mixed uses, industrial, recreational, educational, agricultural, and open space.
- Growth areas—targeted, expected, or recommended areas of urban growth, usually around corridors of current or planned highway capacity improvements, sewer, or water upgrades.
- Specific generators of pedestrian movement—major employment areas (more than 100 employees), schools, entertainment special events, shopping centers/malls, residential neighborhoods, medical centers, religious centers, colleges and universities, bus stops, depot and transit locations, public and private parking garages/facilities, parks and recreational areas.
- Whether or not the Department, county, city, municipality, or other public or private organization plans on some type of capital improvement involving transportation infrastructure.

Existing and anticipated pedestrian characteristics are analyzed including:

- Special user groups—children, adolescents, elderly, disabled, handicapped, commuters, and those dependent upon or utilizing public transit.
- Trip purpose—shopping errands, commuting to school, work, or place of destination, visiting friends, recreational/entertainment, child care, vacation.
- Frequency of use—daily, weekends, seasonal, as needed.
- Other factors—weather conditions with season, time of day, holidays, school and college schedules, safety, convenience, transit routes.

Existing site characteristics affecting pedestrian use are reviewed including available parking (free, metered, hourly, monthly), roadway surface condition, shoulders, pavement markings, crosswalks, street lighting, phased signals, type of transit accommodation, accessibility to destination, access control, intersection links, safety factors, location of signs, channelization, slopes, and drainage.

An analysis of current or proposed land uses along or near a roadway corridor is made to determine if there is a failure to provide or include a design that incorporates sidewalks and pedestrian amenities. The effect of adding new sidewalks in the area is reviewed to ensure user safety. Environmental constraints such as wetlands, floodplain, steep slopes, historical properties or archaeology, hazardous contamination sites, rare or endangered species and farmlands are considered.

A final consideration is whether or not an intersection, roadway or transit corridor, subdivision or land use area can be effectively redesigned or retrofitted with sidewalks and pedestrian facilities.

Information not readily available can be obtained through the following methods:

- Observation,
- Discussion with local governments, planning groups, property owners, civic organizations, and task forces,
- Public information meetings/brochures,
- Accident reports,
- Questionnaires,
- Architectural Accessibility Board,
- Local School Districts,
- Delaware Bicycle Council,
- DNREC's Parks and Recreation, Soil and Water, Technical Services, and Air and Waste Management sections,
- State Historic Preservation Office, or
- Soil Conservation Service-U.S.D.A.

10.8.5 WARRANTS

Warrants based on pedestrian volume have not been established for sidewalks. Actual volumes counted at any one time may not reflect the demand for pedestrian use. Factors such as poor existing facilities (which discourage use), weather conditions, school schedules, holidays, proposed land use changes, growth areas, proximity of transit and depot locations/stops, and other factors affect current pedestrian use. Therefore, many of the benefits from the construction, reconstruction, or the extension of sidewalks and pedestrian facilities are not quantifiable with the actual magnitude of the safety benefit unknown. This is partially because individuals tend to walk where there are sidewalks and sidewalks tend to be installed where people are walking. In addition, pedestrian use and volumes are not regularly collected by planning and transportation agencies and cannot be easily forecasted, modeled or predicted.

The need for sidewalks should be related to the functional classification of streets. For example, collector streets are more likely to

have greater pedestrian use and volumes than residential streets. Collector streets are normally used by pedestrians to access public transit, commercial developments or other various land uses on the arterial to which they feed. Sidewalks should definitely be provided along developed frontages of arterial streets in land use zones that promote pedestrian activity.

Sidewalks should be considered whenever there is regular or periodic pedestrian travel along an existing roadside, street or transit corridor. Sidewalks should also be considered along any street, highway or transit corridor in developed areas not provided with shoulders even if existing pedestrian activity appears light.

10.8.6 DESIGN GUIDANCE FOR SAFE PEDESTRIAN CIRCULATION

Whenever a project is being initiated or planned where pedestrian movement exists or is anticipated, the initial scope and planning involved with the project should provide suitable space within the current or future right-of-way for safe pedestrian circulation. Sidewalks to be financed and maintained by others may also be included in a project.

When there are existing shoulders or walkways intended for pedestrian use, sidewalks should be evaluated at the project development stage for condition, suitable width, continuity, and limits. Where shoulders are being utilized for pedestrian movement, installing a parallel sidewalk may be considered because shoulders must be carried through intersections where turning lanes and pedestrian areas should not be combined.

Deteriorated sidewalks need to be evaluated for rehabilitation or reconstruction and additional width as necessary. Incomplete systems should be considered for connection to new sidewalks and end at logical terminations.

During the subdivision review process often dedicate right-of-way for transportation corridors and/or open space. The dedication of open space may often include areas close to a roadway edge, providing a buffer zone. County sidewalk policies provide for limited pedestrian circulation within a development. The maintenance of such pedestrian facilities becomes the responsibility of the development, local ordinance, adjacent property owner, or governing entity.

Pedestrian and sidewalk projects at intersections or along highway/transit corridors may include design, redesign, or traffic calming measures. These could include tightening of turn radii, channelized islands, medians and curbs, refuge islands, roundabouts, bulbs, neckdowns, signing, striping, transit shelters, and other features.

The extension of project limits beyond related highway or transit improvements for reasonable or short distances may be considered in order to include necessary pedestrian facilities at nearby intersections, provide safe access to public transportation facilities, or to avoid short sidewalk gaps. This decision can be reached anytime during the project development or design stage, and can be adjusted or deleted as needed. The project scope and funding may have to be revised accordingly.

10.8.7 PEDESTRIAN ACCIDENT HISTORY

An important factor in defining the need and locating and designing a sidewalk is accident history. Keep in mind that not all accidents result in a police accident report; supplement data from accident reports with field observations and discussions with stakeholders. The plan development process should include a study to define:

- Where and how did the accident occur: i.e. at an intersection or median, along the road, shoulder or existing sidewalk,

off the shoulder, or at a transit accommodation?

- Who was responsible?
- What was the pedestrian's origin and destination?
- What are the normal pedestrian movements in that area?
- Were there any existing pedestrian accommodations, lumination devices, warning signs, safety or traffic control devices, alcohol involvement or other contributing circumstances?

10.8.8 EXISTING SITE ACCOMMODATIONS

Typically included in the project development and design process is the assessment of the condition of the existing sidewalk network within the project and adjacent area. This assessment would include:

- The location of existing walkways, shoulders, worn paths, and greenway links;
- The location of incomplete walkway systems that adjoin or are within existing right-of way;
- The condition of existing facilities and how well they function or accommodate pedestrian movement;
- Any limiting geographical or architectural features that enhance or reduce feasibility of constructing pedestrian facilities;
- Any major or minor modifications in road or transit design that may enhance or reduce feasibility of constructing pedestrian facilities;
- Existing transit or depot stops with pedestrian or roadside amenities; and
- Whether surrounding or adjacent residential subdivisions, commercial or business land development, mixed land uses, or other developing land uses have provided or include a design that incorporates sidewalks and pedestrian amenities for school bus stops, greenway or walkway links to other land uses, and transit stop access for safe pedestrian movement and circulation.

10.8.9 PEDESTRIAN FACILITY LAYOUT

New and upgraded pedestrian facilities must conform to ADA standards; the information herein does not include all ADA standards. Contact DelDOT's ADA Coordinator and refer to DelDOT's *Standard Construction Details* for additional information.

Avoid warped surfaces within the pedestrian accessible route as much as possible. Warped surfaces can cause a wheelchair to become unstable by causing a wheel to lose contact with the ground.

The minimum vertical clearance for pedestrian space is 80 in [2 m]. Objects protruding into this space such as signs and utility boxes present a hazard for the visually impaired.

10.8.9.1 SIDEWALK REQUIREMENTS

For new sidewalks a minimum width of 5 ft [1.5 m], not including the width of the top of curb, is required. For new sidewalks at underpasses or overpasses, or where there is an obstruction for which the sidewalk must be narrowed, a minimum width of 4 ft [1.2 m] is permissible. If the sidewalk is narrower than 5 ft [1.5 m], then passing spaces at least 5 ft by 5 ft [1.5 m by 1.5 m] shall be provided at intervals not to exceed 200 ft [61 m]. Wider sidewalks may be preferred or required by local ordinance depending upon the volume and nature of pedestrian traffic. Minimum sidewalk thickness can vary according to materials, but be at least 4 in [100 mm] for Portland Cement Concrete. A minimum thickness of

6 in [150 mm] is required at entrance and driveway areas. The maximum cross slope is 2%, even where the sidewalk crosses a driveway entrance.

Small planting strips between the sidewalk and curb may not be practical unless the property owners, civic associations or volunteer programs can make provisions for maintenance. For increased user safety, sidewalks should be as far away from travel lanes as practical. Where possible a buffer width of at least 3 ft [0.9] m between the edge of a sidewalk and the edge of a shoulder, back of curb, or traveled way is preferred. A 3 foot [0.9 m] wide strip would improve safety, driver comfort, and provide an area for snow removal, signage and mailbox posts.

In central business districts, commercial areas, apartment complexes and generally where buildings or parking areas lie near or on the right-of-way, consideration should be given to pave the entire width from curb to building, property, or right-of-way line. The minimum desirable width of sidewalk between curb line and building face is 8 ft [2.4 m]. This permits space for utilities and other roadside appurtenances, and limited snow.

Standard material for any sidewalk or walkway is usually Portland Cement Concrete. However, sidewalk or walkway materials are not limited to Portland Cement Concrete. Upon approval and when funding is available, more aesthetic materials such as brick, asphalt, or other stable, firm, slip-resistant material surfaces may be used. This may be appropriate for traffic calming areas and in certain circumstances to address the concerns expressed by land use planners and/or communities that concrete sidewalks are aesthetically unpleasing.

When constructing, reconstructing, or extending sidewalks at or near intersections, the design should consider enhancing accommodations for pedestrians and vehicles throughout the intersection. Such elements may include refuge islands, separate pedestrian signal indications and phases (with pedestrian button in close proximity to the sidewalk) and offset room for traffic signs/poles and utilities.

In establishing the location of sidewalks, consideration will also be given to the need for or relocation of conflicting drainage facilities, side slopes, new traffic control devices and signing, intersection crossovers, striping, utility appurtenances, mailboxes with posts, and transit stops.

10.8.9.2 CURB RAMP REQUIREMENTS

Curb ramps shall be installed/upgraded for a project where the scope of work includes roadway reconstruction, resurfacing or any sidewalk construction. Examples of projects that do not require curb ramp installation are those that only consist of signing, pavement marking, lighting, and patching. For projects that fall into the qualifying categories, the following shall apply:

1. Curb ramps shall be provided where an accessible route crosses the curb line. An accessible route is a continuous unobstructed path, which includes sidewalks and other exterior pathways with an improved surface, intended for pedestrian use.
2. Curb ramps shall be provided at the termini of all existing and proposed crosswalks within the project limits if there is existing sidewalk or the project includes proposed sidewalk. Crosswalks may be marked or unmarked.
3. Curb ramps at marked crosswalks shall be placed within the crosswalk lines.
4. The curb ramp shall not be compromised by other highway features such as guardrail, utility poles, signs, drainage inlets, manholes or other obstacles. All options for working around obstacles must be considered. If an accessible route cannot be provided

around an obstacle, then the obstacle should be relocated. If the obstacle cannot be relocated then the Project Manager should work with his or her supervisor and assistant director to determine the best solution for the location. Curb ramps must be installed at all required locations when new sidewalk is constructed, existing sidewalk is reconstructed or a road is repaved. All curb ramp locations must be addressed and cannot be skipped when the road is repaved. Some obstacles may not be able to be relocated in order to make a pedestrian facility accessible, including structural elements such as bridge piers and buildings. In these limited cases where it may be technically infeasible to install a curb ramp at a specific location, the reason(s) must be documented.

5. Parking spaces shall not obstruct the accessible route associated with a curb ramp.

6. Positive drainage must be provided to keep water from ponding on curb ramps and in the street along the accessible route.

7. The accessible route shall be along a safe path for pedestrians. Curb ramps shall not be installed where it would be unsafe for any pedestrian to cross due to conditions such as uncontrolled intersections with high vehicle speeds, unsafe sight distance, or physical obstacles that cannot be crossed such as median barriers on multi-lane highways. If it is determined that a safe crossing cannot be provided, then pedestrians shall be routed to a safe, accessible crossing, and the unsafe location shall be constructed with a barrier that will direct pedestrians (including the visually impaired) to a safer crossing. Examples of barriers that could be used at unsafe locations include shrubbery, a grass strip, railings, bollards, a double-posted informational sign with two horizontal struts (one between 36 to 42 inches [0.9 to 1.1 m] and the other between 4 to 12 inches [0.1 to 0.3 m] above the ground), etc. Whatever is used would have to consider detectability by the visually impaired as well as safe roadside design considerations for the clear zone. Locations thought to be unsafe shall be reviewed by DelDOT's Traffic Section. If the crossing cannot be made safe then the Project Manager should work with his or her supervisor and assistant director to determine the best solution for the location and document the reason(s).

8. The designer needs to consider how curb ramp locations may negatively affect the operation and capacity of vehicular traffic.

Diagonal curb ramps (those placed along the curb radius, DelDOT Type 3) are not preferred when pedestrians will have to change direction in the roadway in order to complete their crossing. While it is recognized that there will be locations where they are the best option (such as at intersections with large turning radii), perpendicular curb ramps (DelDOT Types 2 and 4) are preferred. Diagonal curb ramps are often a good treatment when there are channelizing islands at right turn slip lanes. When diagonal curb ramps are used, a 5 ft [1.5 m] long bottom landing area must be provided between the curb radius and the curb line extensions, which is outside the direction of vehicle travel. If this clear distance cannot be provided, diagonal curb ramps shall not be used and another type of curb ramp must be provided. This clear distance is not necessary for right turn slip lane and channelizing island configurations or where there is a landing at the bottom of the ramp within the sidewalk.

The maximum running slope of curb ramps is 12:1. Flatter slopes should be provided where possible. Curb ramps in alterations where a 12:1 slope is not technically feasible may have a maximum running slope of 10:1 for a maximum rise of 6 inches [150 mm].

The maximum cross slope of a curb ramp is 50:1 (2%).

DelDOT's minimum curb ramp width is 60 inches [1.5m].

Where a 12:1 maximum curb ramp slope will not meet the sidewalk grade within a length of 15 feet [4570 mm] due to a steep adjacent roadway, the last ramped section transitioning back to existing pedestrian facilities may be limited to 15 feet [4570 mm], and the slope of that section allowed to exceed 12:1.

If turning or maneuvering is required within the curb ramp or landing area, a maximum slope of 50:1 (2%) in any direction must be provided. The minimum landing area dimension shall be 60 inches [1525 mm] x 60 inches [1525 mm].

Transitions from curb ramp to gutter and streets should be flush and free of level changes. There should not be a bump at the gutter flow line or where the curb meets the pavement. Changes in level up to ¼ inch [6 mm] may be vertical. Changes in level between ¼ inch [6 mm] and ½ inch [12 mm] must be sloped no greater than 2:1. Changes in level above ½ inch [12 mm] must be treated as a ramp and must be sloped at 12:1 or flatter.

Given the constraints and varying field conditions, each curb ramp must be evaluated and designed using these guidelines and the *Standard Construction Details*.

10.8.9.3 DETECTABLE WARNING REQUIREMENTS

Detectable warnings shall be installed at sidewalk curb ramps and at uncurbed sidewalks at the following locations:

- Crosswalks (marked & unmarked) and designated places where pedestrians cross public roadways (including medians and refuge islands);
- Signalized entrances;
- High volume entrances with ADT greater than about 400;
- Entrances with an operating speed of 25 m.p.h. or greater through the pedestrian area;
- Railroad crossings.

Detectable warnings may also be installed in other areas when determined necessary by engineering judgment. Factors which present a potentially hazardous situation may also be considered, including blind spots, complicated turning movements or other situations in which pedestrians with visual impairments should be signaled to stop. They should not be used at all entrances without consideration of the above criteria, since overuse can cause confusion for pedestrians with visual impairments.

A 24 inch [600 mm] long strip of detectable warnings (truncated domes) must be placed along the fully depressed width of the curb ramp at the transition to the street. Detectable warnings must also be placed at cut-through medians and cut-through islands.

10.8.10 MAINTENANCE RESPONSIBILITY

As a policy, DelDOT does not normally maintain sidewalks. Depending upon their location, applicable laws, local ordinances and the current town or city maintenance agreement, sidewalks are the maintenance and upgrade responsibilities of the property owner, homeowner's association, municipality, town, city, suburban or non-suburban area, incorporated or unincorporated area, or governing body which bears jurisdiction. Delaware Transportation Corporation usually contracts out maintenance of sidewalks and other passenger amenities immediately adjacent to transit facilities.

Projects involving pedestrian or sidewalk amenities proposed under the Transportation Enhancement program have a formal agreement with the responsible party that includes a description of maintenance standards to be upheld and assigns the responsibility for the associated costs for those amenities. Maintaining a pedestrian or sidewalk facility involves several items including snow removal, trash and debris removal, control of vegetation, reconstruction, graffiti removal, resigning or re-striping (specifically related to the sidewalk), avoiding general neglect and deterioration for whatever cause and alterations of the surface or subsurface level required to improve the appearance.

Projects under the Suburban Streets and Resurfacing Program may include repairs, replacement or general maintenance to existing deteriorated sidewalks. For all new construction and replacement, accessibility guidelines and standards are to be followed.

DelDOT repairs or replaces any existing sidewalk surface that has been damaged or altered by DelDOT forces or its contractors. Repair or replacement of sidewalks follows all accessibility guidelines and standards.

10.8.11 REMINDERS

When sidewalks are proposed or initiated without a formal agreement to the contrary, DelDOT may not assume any maintenance responsibility.

Although not to be addressed as part of the project, it may be beneficial to consider the need for future sidewalk and reserve the right-of-way.

Delaware's Department of Education does have busing rules affecting the busing rights for children. These may come into play if certain communities and schools are connected with continuous sidewalk access. The most significant affect is that public busing privileges may be revoked.

Local governments or jurisdictions may adopt land use or subdivision ordinance regulations to protect transportation facilities, corridors, and sites for their identified functions. This could include, but is not limited to, facilities providing safe and convenient pedestrian or bicycle access within and from new subdivisions, planned unit developments, transit stops, greenways, and neighborhood activity centers such as schools, parks, and shopping areas.

If not required under county, city, or local jurisdiction, DelDOT may request or require sidewalks and pedestrian facilities to be installed prior to subdivision entrance permit approval. DelDOT may also request the installation of sidewalks and pedestrian facilities along roadway or transit corridors as part of the mitigation plan under a traffic impact study.

As a part of improving the transportation network, DelDOT initiates and designs sidewalk projects to connect existing and future transit routes, transit facilities, park-and-ride lots, public/private parking areas, bus stops for schools, businesses, shopping centers, industrial parks, residential communities, or any other public or private institution. This enhances multi-modalism while decreasing vehicular traffic and automobile emissions.

10.8.12 FUNDING ALTERNATIVES

Although specially defined on the Project Initiation Form, the designer should be aware that the construction, reconstruction, or extension of sidewalks can be funded by several different methods and funding sources.

Projects initiated by DelDOT are usually 100% funded by the Department and include:

- Removal and replacement of existing sidewalk caused by the construction, reconstruction, improvement, or extension of any highway, transit,

safety, or pedestrian related improvement;

- Projects facilitating State or DelDOT transportation purposes. This may include, but is not limited to, improving or expanding transit facilities, walkway/pedestrian corridors, greenway links, or safety improvement projects.

Projects initiated by others for design and/or construction by DelDOT have various matching fund ratios. These include:

- Projects may be 100% funded under the Suburban Streets and Resurfacing Program for new sidewalk projects that have been approved and initiated by DelDOT in recognition of meeting the needs of the public, town officials, or other governing bodies.
- Projects may be funded and initiated by a school district or other agency with a 50% match by DelDOT. These would involve new or reconstructed sidewalks within a project's termini or short distances outside a project area to connect sidewalks to existing pedestrian or transit generators from or to educational facilities.
- Projects may be 100% funded and initiated by a party or agency for removal and replacement of deteriorated sidewalk.
- Projects may be 50% jointly funded by a party or agency for utility adjustments, drainage, signals, pedestrian barriers, retaining walls, crossovers, etc. required solely for sidewalk safety and enhancement. This does not include projects initiated for ADA conformance.
- Projects may be 10% funded by an initiating party or agency, 10% by DelDOT and 80% funded by the Federal Highway Administration. These projects meet the criteria for funding under the federal Intermodal Transportation Efficiency Act. Qualified applicants can initiate projects to be included in DelDOT 's Transportation Enhancement Program. These projects must first gain approval from DelDOT's Technical Advisory Board. The projects are then subject to final review and approval by the Secretary of Transportation.

10.9 BICYCLE FACILITIES

There is a wide range of facility improvements that can enhance bicycle transportation. Suitable accommodations for bicyclists shall be determined as part of the DelDOT's Project Development Process and scoped as part of the project handoff prior to design initiation. Improvements can be simple involving minimal design considerations such as changing drainage grate inlets, or they can involve a detailed design such as providing a bike path.

Facility improvements for motor vehicles through appropriate planning and design can enhance bicycle travel or should at least avoid adverse impacts on cycling. Unless access is specifically denied, some level of bicycle use can be anticipated on most roadways. All new roadways, except those where bicyclists will be legally prohibited, should be designed and constructed under the assumption that they will be used by bicyclists. Guidelines are presented here to help design and construct roadway improvements and separate facilities that accommodate the operating characteristics of bicycles. Additional information including signing layouts, striping, and design details can be found in AASHTO's *Guide for the Development of Bicycle Facilities.*

Because most highways have not been designed with bicycle travel in mind, there are often many ways in which roadways should be improved to more safely accommodate bicycle traffic. Roadway conditions should be examined and, where necessary, safe drainage inlets, safe railroad crossings and smooth pavements should be provided. Drainage inlets and utility covers are potential problems to bicyclists. When

designing a new roadway, these types of appurtenances should be kept out of the bicyclists' expected path.

Parallel bar drainage inlet grates can trap the front wheel of a bicycle causing loss of steering control. Often, the bar spacing is such that it allows narrow bicycle wheels to drop into the grates, resulting in irreparable damage to the bicycle wheel and frame and/or injury to the bicyclist who could be thrown from the bicycle. These grates should be replaced with the bicycle-safe grates in the DelDOT's *Standard Construction Details*. Parallel bar grates are not to be used where bicycles may be present.

Railroad-highway grade crossings should ideally be at a right angle to the rails. The more the crossing deviates from this ideal crossing angle, the greater is the potential for bicyclists' front wheels to be trapped in the flange way causing loss of steering control. It is also important that the roadway approach be at the same elevation as the rails.

Consideration should be given to the materials of the crossing surface and to the flange way depth and width. If the crossing angle is less than approximately 45 degrees, consideration should be given to widening the outside lane, shoulder, or bicycle lane to allow bicyclists adequate room to cross the tracks at a right angle.

Pavement surface irregularities can do more than cause an unpleasant ride. Gaps between pavement slabs or drop-offs at overlays parallel to the direction of travel can trap a bicycle wheel and cause loss of control; holes and bumps can cause bicyclists to swerve into the path of motor vehicle traffic. Thus, to the extent practicable, pavement surfaces should be free of irregularities and the edge of the pavement should be uniform in width. On older pavements it may be necessary to fill joints, adjust utility covers or, in extreme cases, overlay the pavement to make it suitable for bicycling. Longitudinal joints in pavement and between pavement and gutter pans should not be more than ½ in [12 mm] wide. Longitudinal drop-offs between pavement and gutter pans or between travel lane pavement and shoulder pavement should not exceed ¾ in [18 mm]. Ridges used to create "rumble strips" in pavements should not be more than ¾ in [18 mm] when perpendicular to bicycle travel. Properly located warning signs should precede these locations.

Roadway treatments intended to accommodate bicycle use must address the needs of both experienced and less experienced riders. One solution to this challenge is to develop the concept of a "design cyclist" and adopt a classification system for bicycle users such as the following:

<u>Group A-Advanced Bicyclists</u> - Experienced riders who can operate under most traffic conditions. They comprise the majority of the current bicycle users of collector and arterial streets.

<u>Group B-Basic Bicyclists</u> - Casual or new adult and teenage riders who are less confident of their ability to operate in traffic without special provisions for bicyclists. The basic rider is comfortable riding on neighborhood streets and shared use paths but prefer designated facilities such as bike lanes or wide shoulder lanes on busier streets.

<u>Group C-Children</u> - Pre-teen riders whose roadway use is initially monitored by parents. Eventually they are accorded independent access to the system. Provisions should be made to allow access to key destinations without encouraging them to ride in the travel lane of major arterials.

In the design of bicycle facilities two broad classes of bicyclists are used: Group A riders and Group B/C riders.

Generally, the Group A bicyclist will be best served by designing all roadways to

accommodate shared use by bicyclists and motor vehicles. This can be accomplished by:

- Providing wide outside lanes or bike lanes on collector and arterial streets built with an urban section (i.e., with curb and gutter);
- Providing useable shoulders on highways built with a rural section (i.e., no curb and gutter).

Group B/C bicyclists are best served by a network of neighborhood streets and designated bicycle facilities that can be provided by:

- A network of designated bicycle facilities (e.g., bike lanes, separate bike paths, or side-street bicycle routes) through the key travel corridors typically served by arterial and collector streets.
- Useable roadway shoulders on rural highways, 4 ft [1.2 m] wide.

10.9.1 FACILITY SELECTION

The most significant factors affecting compatibility of roadways for bicycling are motor vehicle traffic volumes, operating speed, and the width of the travel lane and shoulder. The selection of facility type should be determined by an analysis of these factors in addition to the following:

1. State and local bicycle master plans;
2. Proximity of schools, parks and other destinations where a child bicyclist may be expected;
3. Presence of a regionally significant or locally designated bicycle route;
4. Potential turning movement conflicts; and
5. Environmental, historical and right-of-way constraints.

In general, additional travel lane or shoulder width can increase the suitability of a roadway for bicycling. Designation of bicycle lanes with appropriate signs and pavement markings will help increase the predictability of both bicycle and motor vehicle movements. Additional separation of bicycle traffic from motor vehicle traffic on shared use paths may be desirable on high speed, high volume roadways, where an increase in child bicyclists can be expected or along regional pathway networks. Development of a shared use path does not preclude the need to accommodate more experienced bicyclist on the roadway.

10.9.2 FACILITY TYPES

Four basic types of facilities are used to accommodate bicyclists:

1. Shared Roadway (No Bikeway Designation) - Shared lanes are streets and highways with no special provision for bicyclists. Shared lanes typically feature 12 ft (3.6 m) lane widths or less with no shoulders, allowing cars to safely pass bicyclists only by crossing the centerline where permissible or moving into another traffic lane. In residential areas with lane widths of at least 12 ft [3.6 m] low motor vehicle traffic volumes and average motor vehicle speeds of less than 30 mph [50 km/h], shared lanes will accommodate group A riders, and will normally be adequate for group B/C bicyclists. Where the existing lane width is less than 12 ft [3.6 m], additional lane width is called for. For higher speeds and traffic volumes, shared lanes become less attractive routes, especially to group B/C riders.
2. Shared Roadway, Signed - These roadways are designated by bike route signs and either provide continuity to other bicycle facilities or designate a preferred route through high demand corridors. In designating a roadway as a shared roadway, DelDOT must assure there are advantages to using this route, the route is suitable, and it will be

maintained in a manner consistent with the bicyclist's needs. The use of signing and striping will advise the motorist to expect bicyclists and be prepared to react safely.

3. Bike Lane - Bike lanes are established with appropriate pavement marking and signing along streets in corridors where there is a significant bicycle demand. In order to accomplish this, space must be provided or created for preferential use by bicycles. Adequate space, pavement markings, traffic control, pavement conditions, surface hazards etc. must all be addressed. This will give both the bicyclist and the motorist a level of predictability of how each will maneuver. Roads marked with bike lanes indicate that the road is recommended for safe cycling. This designation should not be made unless the bike lane will be a continuous safe route for the length of the bike lane (e.g. through intersections and turn lanes). In area where bicycle and motor vehicle traffic cross paths, pavement markings and signs should indicate the location of the bicycle lane and warn of potential conflict areas.
4. Shared Use Path - Shared use paths are on exclusive right-of-way and with a minimum of cross flow by motor vehicles. Users are non-motorized and may include bicyclists, in-line skaters, roller skaters, wheelchair users, and pedestrians, including walkers, runners, people with baby strollers, people walking dogs, etc.

10.9.2.1 DESIGN APPROACH

Depending upon the information provided on the project initiation form and/or as a result of the project-scoping meeting, the designer might have to evaluate one or all of the four types of facilities as a part of the context sensitive design approach. Bicycles are a part of the available transportation system. If properly addressed, increased bicycle use can contribute to the reduction in air pollution and motor vehicle use.

10.9.3 SHARED ROADWAY

Most bicycle travel occurs on streets and roadways that have no bikeway designation (shared roadway) because the existing street system is adequate to handle safe and efficient bicycle travel. In reviewing a project for shared use, the most important item to consider is whether or not adequate space can be provided to allow safe shared use by the motorist and bicyclist. The items to be considered include adding paved shoulders, increasing lane widths, eliminating on-street parking, improving pavement surface quality, and providing safe drainage grates, utility covers, traffic control boxes, etc. in the useable area.

Improving or adding paved shoulders may be the best alternative to serve both modes. Other benefits of having shoulders are improved mainline drainage, protection of mainline pavement structure, emergency use, etc. The most obvious project effects are increases in design effort, construction costs, and right-of-way needs.

Paved shoulders should be at least 4 ft [1.2 m] wide to accommodate the bicyclist. However, any shoulder widening will be an improvement to the overall use and safety of the project. For the bicyclist, the useable shoulder width does not include the gutter pan, unless the gutter pan is 4 ft [1.2 m] or greater in width. Where guardrail is used, the shoulder width should be increased to 5 ft [1.5 m]. Greater motor vehicle speeds and volumes combined with increased use by bicyclists require that more shoulder width be provided. Design controls found in other chapters of this manual will usually be applied in these situations and will be adequate to serve bicyclists.

The use of rumble strips or raised pavement markings adjacent to shoulders warning errant drivers or discouraging the use of shoulders by motorist can be a

problem for bicyclist. Rumble strips should be marked with a warning sign. Where they are to be used there should be:

(1) A clear path of 1 ft [0.3 m] from the rumble strip to the outside edge of the traveled way;

(2) A clear area of 4 ft [1.2 m] from the rumble strip to the outside edge of paved shoulder; or

(3) A clear area of 5 ft [1.5 m] to the adjacent guardrail, curb or other obstacle.

Where shoulders can not be provided, wider lane widths may be attainable. On roadway sections without designated bikeways, an outside lane, whether curbed or not, wider than 12 ft [3.6 m] can better accommodate the motorist, bicyclist and adjacent land use.

A useable lane width of 14 ft [4.2 m] is preferred when curb lanes are to be shared. This width does not include the gutter pan and is measured from the edge stripe to lane stripe or from the gutter pan edge to the lane stripe. Where there is a continuous steep grade, drainage grate interference, pavement reflectors, or on street parking, a curb lane width of 15 ft [4.5 m] is preferred. Too much width can be hazardous by encouraging the operation of two vehicles in a lane intended as a single lane operation.

Wide curb lanes have three widely accepted advantages. They can:

- Accommodate shared bicycle/motor vehicle use without reducing the roadway capacity for motor vehicle traffic;
- Minimize the real and perceived operating conflicts between bicycles and motor vehicles; and
- Increase the roadway capacity by the number of bicyclists capable of being accommodated.

Wide outside lanes require the least amount of additional maintenance of the different facilities. The sweeping effect of passing motor vehicles and routine highway maintenance is usually enough to keep the lane free of debris and in good condition for bicycling.

Wide outside lanes are especially valuable for Group A riders who are competent in sharing the roadway with motor vehicles. The same is not true for Group B/C riders. Except on residential or low-volume streets, wide outside lanes are not generally sufficient to provide the degree of comfort and safety required by less skilled bicyclists or children and will do little to encourage them to ride.

Wide curb lanes will be most applicable in urban areas on major streets where Group A riders will likely be operating. If no alternative exists for Group B/C riders, a bike lane or shoulder should typically be used.

The designer should not overlook the potential for encouraging bicycle use on rural routes. Adding 4 ft [1.2 m] of paved shoulder and a 4 in [100 mm] edge stripe can safely provide this.

Facilities with on-street parking provide greater opportunity for conflicts between cyclist and motorist since the rider will be between moving traffic and parked vehicles. The bicyclist is subject to opening car doors, exiting vehicles, extended mirrors that narrow travel space and reduced visibility. Where this type of operation is to be permitted, a parking lane of a minimum width of 12 ft [3.6 m] is needed.

The shared roadway can also be occupied by existing or proposed surface obstacles to the bicyclist. These include drainage grates, utility covers and traffic control appurtenances. Throughout the design and construction phases these obstacles should be ether eliminated or designed to accommodate bicycle use.

10.9.4 SIGNED SHARED ROADWAY

Signed shared roadways are those that have been identified as bicycle routes and are signed accordingly. The reasons for designating a certain route as preferred for bicycle use include:

- The route provides continuity to other bicycle facilities,
- The road is a common route through a high demand corridor,
- In rural areas, the route is preferred due to low motor vehicle volumes, aesthetics, or availability of paved shoulders, or
- The route extends along local streets and collectors leading to a neighborhood destination such as a park, school or commercial district.

Signing a particular route suggests that there are advantages to using this route rather than some other alternative. This designation implies that certain criteria have been established and are being maintained. Criteria to be considered prior to selecting a route for designation include:

- In high demand corridors, the route is a direct through route;
- The route connects discontinuous segments of other types of bicycle facilities;
- Traffic control devices have been adjusted to reflect increased bicycle use;
- Street parking has been removed or restricted where width is critical to provide safe travel;
- A smooth surface has been provided;
- Maintenance will be provided to remove accumulated debris and keep traffic control devices serviceable;
- Wider curb lanes are provided as compared to parallel alternative routes; and
- Shoulder or wider lane widths meet the established minimums.

10.9.5 BIKE LANES

A bike lane is a portion of the roadway that has been designated by striping, signing and pavement markings for the preferential or exclusive use of bicyclists. Four typical bike lane layouts are shown in Figure 10-6.

Bike lanes are considered when it is desirable to delineate available road space for preferential use by bicyclists. The beginning and end of the bike lane should be clearly signed and marked. Bike lanes should not be designated on roads where the lane must bend in an unsafe location.

Bike lane markings can increase a bicyclist’s (especially B/C riders) confidence in motorists not straying into his/her path of travel. Likewise, passing motorists are less likely to swerve to the left out of their lane to avoid bicyclists on their right. Provision with appropriate signage should be made for a continuous bike lane through turn lanes and at intersections. On high-speed roadways where a left turn lane is introduced, a bike lane should be continued on the right of the high-speed lane.

Bike lanes should be one-way facilities and carry traffic in the same direction as adjacent motor vehicle traffic. Two-way bike lanes on one side of the roadway are not recommended when they promote riding against the flow of motor vehicle traffic. Wrong-way riding is a major cause of bicycle accidents and violates the Rules of the Road as stated in Delaware's Uniform Vehicle Code. Bike lanes on one-way streets should be on the right side of the street, except in areas where a bike lane on the left will decrease the number of potential conflicts (e.g., conflicts with heavy bus

traffic). Contra flow bike lanes may be considered in exceptional circumstances in urban area where appropriate traffic control devices may be used to ensure the safety of users.

For roadways with no curb and gutter, the minimum bike lane width is 4 ft [1.2 m]. Certain edge conditions indicate the need for additional bike lane width. If parking is permitted, the bike lane should be placed between the parking area and the travel lane and have a minimum width of 5 ft [1.5 m].

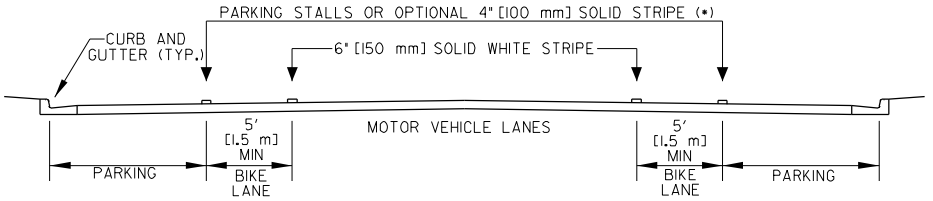
Where parking is permitted but a parking stripe or stalls are not utilized, the shared area should be a minimum of 11 ft [3.3 m] without a curb face and 12 ft [3.6 m] adjacent to a curb face. If the parking volume is substantial or turnover is high, an additional 1 to 2 ft [0.3 to 0.6 m] is desirable. Bike lanes between the curb and the parking lane can create obstacles for bicyclists from opening car doors and poor visibility at intersections and driveways, and they also prohibit bicyclists from making left turns. Therefore this placement should not be considered. Bike lanes are not advisable where angled parking exists.

Bike lanes along the outer portions of an urban curbed street, where parking is prohibited, also require special consideration. Bicyclists do not generally ride near a curb because of the possibility of debris, hitting a pedal on the curb, an uneven longitudinal joint, or a steeper cross slope. Bike lanes in this location should have a minimum width of 5 ft [1.5 m] from the curb face. If the longitudinal joint between the gutter pan and the roadway surface is uneven and falls within 5 ft [1.5 m] of the curb face, a minimum of 4 ft [1.2 m] should be provided between the joint and the motor vehicle lanes.

Bike lanes should be placed within the paved shoulder area at the outside edge in rural areas having a roadway section without curb, gutters and with infrequent parking. Bike lanes should have a minimum width of 4 ft [1.2 m]. Where the shoulder can provide additional maneuvering width, a width of 5 ft [1.5 m] or greater is preferable; additional widths are desirable where substantial truck traffic is present, or where there are excessive vehicle speeds.

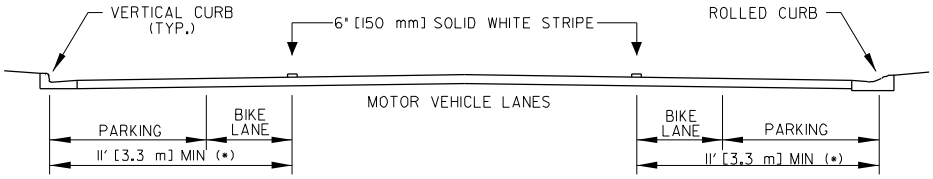
A bike lane should be delineated from the motor vehicle travel lanes with a 6 in [150 mm] wide solid white line. Bike lanes should be provided with adequate drainage to prevent water ponding, washouts, debris accumulation and other potential hazards to the bicyclist. Adequate pavement surface, bicycle-safe grate inlets, safe railroad crossing, and traffic signals responsive to bicycles should always be provided on roadways where bicycle lanes are being designated. Raised pavement markings and raised barriers can cause steering difficulties for bicyclists and should not be used to delineate bicycle lanes.

Figure 10-6
Typical Bike Lane Cross Sections



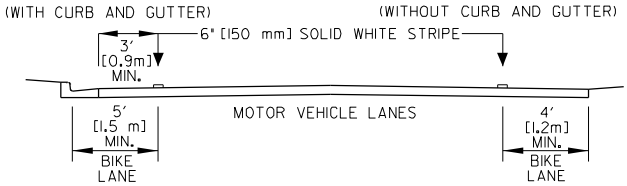
* - THE OPTIONAL SOLID WHITE STRIPE MAY BE ADVISABLE WHERE STALLS ARE UNNECESSARY (BECAUSE PARKING IS LIGHT) BUT THERE IS CONCERN THAT MOTORISTS MAY MISCONSTRUE THE BIKE LANE TO BE A TRAFFIC LANE.

(1) ON-STREET PARKING

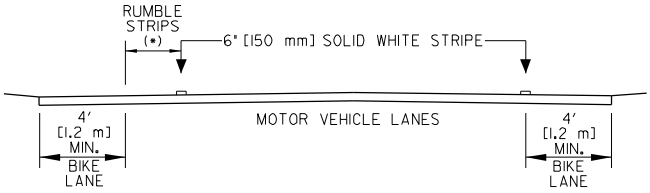


* - 13' [3.9 m] IS THE RECOMMENDED MINIMUM WHERE THERE IS SUBSTANTIAL PARKING OR WHERE TURNOVER OF PARKED CARS IS HIGH (E.G. COMMERCIAL AREAS).

(2) PARKING PERMITTED WITHOUT PARKING STRIPE OR STALL



(3) PARKING PROHIBITED



* - IF RUMBLE STRIPS EXIST THERE SHOULD BE 4' [1.2 m] MINIMUM FROM THE RUMBLE STRIPS TO THE OUTSIDE EDGE OF THE SHOULDER.

(4) TYPICAL ROADWAY IN OUTLYING AREAS PARKING PROTECTED

10.9.5.1 INTERSECTIONS WITH BIKE LANES

Bike lanes tend to complicate both bicycle and motor vehicle turning movement at intersections. Because they encourage bicyclists to keep to the right and motorists to keep to the left, both operators are somewhat discouraged from merging in advance of turns. Thus, some bicyclists will begin left turns from the right side bicycle lane and some motorists will begin right turns from the left of the bicycle lane. Both maneuvers are contrary to established rules of the road and result in conflicts.

At intersections, bicyclists proceeding straight through and motorists turning right must cross paths. Striping and signing configurations that encourage these crossings in advance of the intersection, in a merging fashion, are preferable to those that force the crossing in the immediate vicinity of the intersection.

The design of a bike lane needs to include appropriate pavement markings and signing approaching and through intersections to reduce the number of conflicts. Guidance for signing and pavement marking of bike lanes is shown in the MUTCD and AASHTO's G*uide for the Development of Bicycle Facilities.*

10.9.6 SHARED USE PATH

A shared use path is a facility that is physically separated from the roadway and intended for exclusive use of modes other than motorized vehicles. Initially perceived as bicycle paths, these facilities have grown in popularity serving bicyclists, in-line skaters, roller skaters, wheelchair users, and pedestrians, including, walkers, runners, people with baby strollers, people walking dogs, etc.

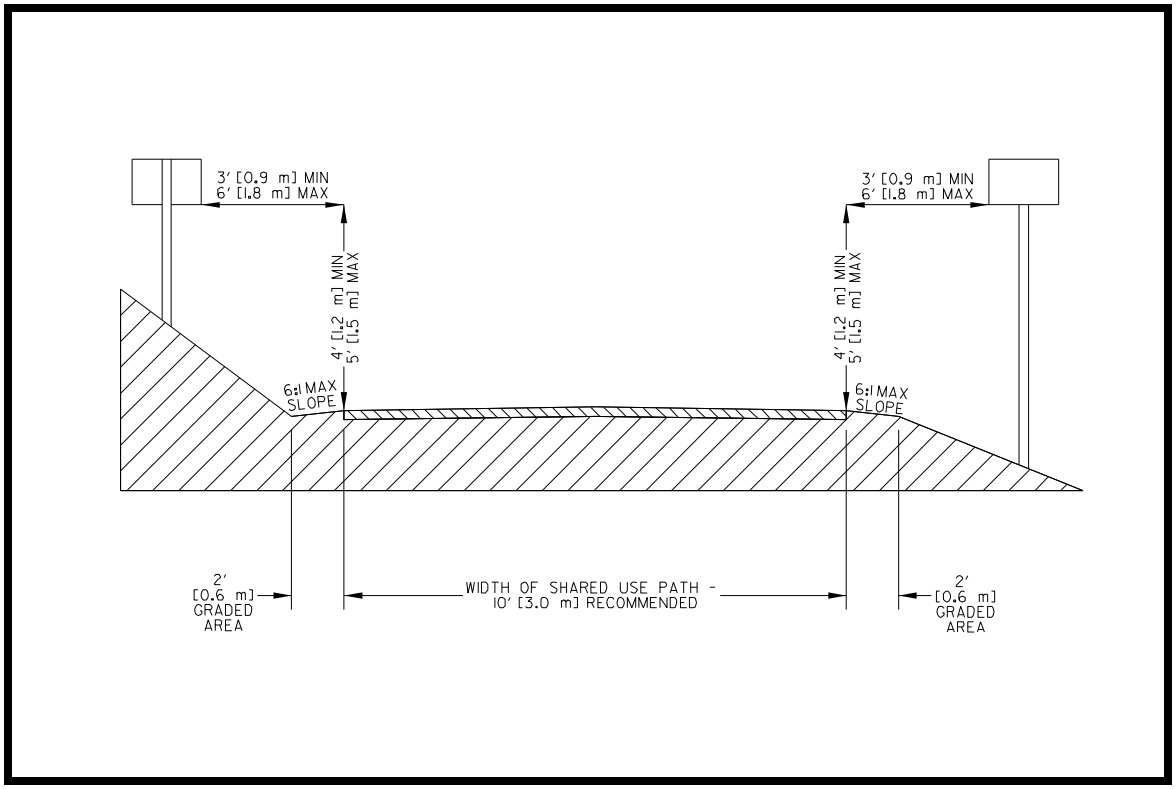
These facilities should be designed in accordance with the Americans with Disabilities Act standards for shared transportation paths. Maximum slope, cross slope and the rate of change in grade should be carefully examined during the design process. Because of their multi-use attraction they are a valuable addition to the highway system and to the range of facilities available to planners and engineers seeking to improve conditions for all categories of travelers. They can serve both a transportation and recreational function and have proven to be significant generators of bicycle use. Groups A and B/C riders (as well as other non-motorized users) can benefit from the absence of motor vehicle traffic on these paths. If bicycle and pedestrian volumes are expected and large numbers of conflicts are expected, a separate pedestrian facility may need to be considered.

The inclusion of a shared use path in a project would be the result of the planning process and be fully described in the project initiation documentation. Figure 10-7 shows the layout for a typical shared use path.

Shared use paths provide users with a shortcut through residential areas, provide enjoyable recreational opportunity, and provide access to areas not accessible by motor vehicle and areas only accessible by limited access highway facilities closed to this type of user.

Separate bike paths may be referred to as “multi-use trails” or “greenways” even though they are slightly different facilities. A trail typically runs along an independent right of way such as an abandoned railroad corridor, and a greenway is a park-type corridor of land that may or may not incorporate a trail within its boundaries.

Figure 10-7
Cross Section of Two Way Shared Use Path on Separate Right of Way



The design criteria categories that apply to shared-use paths are based on those used for highways including horizontal alignment, vertical alignment, vertical and lateral clearances, sight distance, grades and pavement structures. The similarity does not carry over into the actual criteria as the operational characteristics are significantly different.

Even when providing a shared path, street improvements to address this mode of transportation should not be overlooked. Many users will still use the local street system perceiving it to be more convenient, safer and better maintained, particularly for utility trips.

10.9.6.1 SEPARATION BETWEEN SHARED USE PATHS AND ROADWAYS

One of the most important elements in designing a successful shared path is the separation between the path and any adjacent roadways. Unless there is adequate separation there will be operational problems that make it more desirable and convenient for the user to use the roadway. The minimum separation of a bicycle path from a roadway is 5 ft [1.5 m]. When this minimum is not possible a suitable barrier at least 44 in [1.1 m] in height should be provided.

Problems associated with locating shared use paths too close to a roadway include:

- Requires one direction of bicycle traffic to ride against motor vehicle traffic;

- Upon entering or leaving a path bicyclist going against traffic tend to continue on the wrong side;
- Creates nonstandard flow conflicts at intersections;
- Signs posted for roadway users are backwards for the contra flow bicyclist;
- Many bicyclists will perceive the roadway as more convenient, better maintained or safer;
- At intersections and entrances, drivers expect the bicyclist to stop or yield but this does not usually happen;
- Stopped vehicles at cross-streets and vehicles exiting side streets and driveways block the path; and
- The distance between the two facilities may require the use of a barrier creating a hazard to the bicyclist and the motorist.

Because of the problems when shared use paths are located too close to a roadway, other types of bikeways may be more practical for accommodating bicyclists.

10.9.6.2 WIDTH AND CLEARANCE

Two-way shared paths should be a minimum of 10 ft [3.0 m] wide. In high use areas it may be desirable to increase the width to 12 ft [3.6 m].

A minimum graded area of 2 ft [0.6 m] in width should be maintained adjacent to both sides of the pavement. However 3 ft [0.9 m] or more is desirable to provide clearance from trees, poles, walls, fences, guardrails, or other lateral obstructions. A minimum 5 ft [1.5 m] separation from the edge of the path pavement to the top of slope is desirable. Depending upon the height of embankment, it may be necessary to provide a physical barrier.

The minimum width of a one-directional shared use path is 6 ft [1.8 m]. However, without adequate enforcement, it can be anticipated that the path will be used as a two-way facility and designed accordingly.

The vertical clearance to obstructions should be a minimum of 8 ft [2.4 m], however, vertical clearance may need to be greater to permit passage of maintenance vehicles and, under crossings and tunnels, a clearance of 10 ft [3 m] is desirable for adequate vertical shy distance.

10.9.6.3 DESIGN SPEED

The bicyclist controls the design speed and is dependent on several factors, including the type and condition of the bicycle, the purpose of the trip, the condition and location of the shared use path, the speed and direction of the wind and the physical condition of the bicyclist. Shared use paths should be designed for a selected bicycle speed that is at least as high as the preferred speed of the faster bicyclist. In general, a minimum design speed of 20 mph [30 km/h] should be used; however, when the grade exceeds 4 percent a design speed of 30 mph [50 km/h] is advisable.

For most bicycle path applications the superelevation rate will vary from a minimum of 2 percent (the minimum necessary to encourage adequate drainage) to a maximum of approximately 3 percent (beyond which maneuvering difficulties by slow bicyclists and adult tricyclists might be expected). The minimum superelevation rate of 2 percent will be adequate for most conditions and will simplify construction.

For a superelevation rate (e) of 2 percent, the minimum radii of curvature is 80 ft [25 m] for a design speed of 20 mph [30 km/h] and 260 ft [80 m] for a design speed of 30 mph [50 km/h]. When substandard radius curves must be used on bicycle paths due to right-of-way restrictions, topographical or other considerations, standard curve warning signs and supplemental pavement markings should be installed in accordance with the MUTCD. The negative effects of

substandard curves can also be partially offset by widening the pavement through the curves.

10.9.6.4 GRADES

Grades on shared paths should be kept to a minimum, especially on long inclines. Grades greater than 5 percent are undesirable because the ascents are difficult for many bicyclists to climb and the descents cause some bicyclists to exceed the speeds at which they are competent. Where terrain dictates, grades over 5 percent and less than 500 ft [150 m] long are acceptable when a higher design speed is used and additional width is provided. Grades steeper than 3 percent may not be practical for shared paths with crushed stone or other unpaved surfaces because of handling and increased maintenance problems.

If excessive grades must be used there are several alternatives to address this problem. These are:

- Widen the path to allow slower users to walk;
- Provide signing, alerting users of the steep slope ahead;
- Provide recommended speed signs,
- Exceed the minimum stopping sight distance;
- Exceed the minimum horizontal clearances, recovery areas, and if necessary add protective rails; or
- Design the path with a series of switchbacks.

10.9.6.5 HORIZONTAL ALIGNMENT

Unlike a motor vehicle, a bicycle must be leaned when entering a curve to prevent it from falling outward. If the bicyclist pedals through a curve and leans too far, the pedal may strike the surface. Depending upon the ability of the bicyclist, the lean angle may vary. Pedal heights may differ, but generally a 25° lean angle will cause the pedal to strike the surface. In design, a lean angle of $15\text{-}20^{o}$ should be used.

The following equations are used to determine the minimum radius of curvature.

For US Customary Units:

$$R = \frac{0.067V^2}{\tan\theta}$$

For Metric Units:

$$R = \frac{0.0079V^{2\prime}}{\tan\theta}$$

Where:

R = Minimum radius of curvature ft [m]
V = Design Speed mph [km/h]
θ = Lean angle from the vertical (degrees)

When the lean angle approaches 20^0, the minimum radius becomes a function of the superelevation of the pathway surface, the coefficient of friction between the bicycle tires and surface and the speed of the bicycle. For this design condition, the minimum radius or curvature can be derived from the equation:

For US Customary Units:

$$R = \frac{V^2}{15\left(\frac{e}{100} + f\right)}$$

For Metric Units:

$$R = \frac{V^2}{127\left(\frac{e}{100} + f\right)}$$

Where:

R = Minimum radius of curvature ft [m]
V = Design speed mph [km/h]
e = Rate of bikeway superelevation (percent)
f = Coefficient of friction

The equation variables are limited in range. Shared paths are subject to the Americans with Disabilities Act. Cross slopes are limited to a maximum of 3 percent. Friction factors are controlled by the rider's sense of discomfort. Figures 10-8 and 10-9 provide an easy reference for design application.

If the minimum radius is not attainable, warning signs and other supplemental signing as per the MUTCD should be installed

10.9.6.6 SIGHT DISTANCE

The sight distance accommodating bicycle traffic usually controls the design of a shared use path. In some cases, sight distance for pedestrians and wheelchair users should be considered as well.

Figure 10-8
Desirable Minimum Radii for Paved Shared Use Paths[1]

Design Speed (V) mph [km/h]	Minimum Radius (R) ft [m]
12 [20]	36 [12]
20 [30]	100 [27]
25 [40]	156 [47]
30 [50]	225 [74]

[1]Based on a 15^0 lean angle

Figure 10-9
Minimum Radii for Paved Shared Use Paths[1]

Design Speed (V) mph (km/h)	Friction Factor (*f*) (paved surface)	Minimum Radius (R) ft (m)
12 [20]	0.31	30[10]
20 [30]	0.28	90 [24]
25 [40]	0.25	1155 [47]]
30 [50]	0.21	260 [86]

[1]Paths Based on 2% Superelevation Rate and 20^0 Lean Angle

To provide bicyclists with an opportunity to see and react to the unexpected, a shared use path should be designed with adequate stopping sight distance. The distance required to bring a bicycle to a full controlled stop is a function of the bicyclist's perception and brake reaction time, the initial speed of the bicycle, the coefficient of friction between the tires and the pavement, and the braking ability of the bicycle.

The minimum stopping sight distance of bicycles is based on a total perception and brake reaction time of 2.5 seconds and a coefficient of friction of 0.25, to account for the poor wet weather braking characteristics of many bicycles. For two-way bicycle paths, the sight distance in descending direction, that is, where G is negative, will control the design. Sight distance is calculated using the following equation(s):

For US Customary Units:

$$S = \frac{V^2}{30(f \pm G)} + 3.67V$$

For Metric Units:

$$S = \frac{V^2}{254(f \pm G)} + \frac{V}{1.4}$$

Where:

S = Minimum stopping sight distance, m
V = Design Speed, mph [km/h]
G = Grade, ft/ft [m/m] (descending - Neg., Ascending +Pos.)

The minimum length of vertical curve necessary to provide minimum stopping sight distance at various speeds on crest vertical curves is given by the following equation. The assumptions used for the equation are the eye height of the bicyclist is 4.5 ft [1.4 m] and a object height of zero recognizing that impediments to bicycle travel exist at pavement level.

For US Customary Units:

When S>L L = 2S-900/A
When S<L L = AS^2/900

For Metric Units:

When S>L L = 2S-(280/A)
When S<L L = AS^2/280

Where:

L = Minimum Length of Vertical Curve ft (m) ≥ 3 ft [1.0 m]
S = Stopping Sight Distance ft [m]

A = Algebraic difference (%)

Height of bicyclist eye = 4.5 ft [1.4 m]

Height of object = 0 ft [0 m]

Bicyclists frequently ride abreast on bicycle paths. On narrow bicycle paths, bicyclists have a tendency to ride near the middle of the path. For these reasons, and because of the serious consequences of a head on bicycle accident, lateral clearances on horizontal curves should be calculated based on the sum of the stopping sight distances for bicyclists traveling in opposite directions around the curve. Where this is not possible or feasible, consideration should be given to widening the path through the curve, installing a yellow center stripe, installing a curve ahead warning sign in accordance with the MUTCD, or some combination of these alternatives.

Tables and curves for solutions of both sight distance equations, under various design conditions, can be found in AASHTO's *Guide for the Development of Bicycle Facilities.*

10.9.6.7 INTERSECTIONS

Intersections with roadways are important safety considerations in shared path design. There are three basic types of path-roadway intersections: mid-block, adjacent path and complex. If alternate locations are available, the one with the most favorable intersection conditions should be selected.

Mid-block crossings should be located far enough from the intersection to remain outside of the vehicular traffic mix approaching and entering an intersection. It may be preferable to provide a mid-block bicycle path crossing at a minimum distance of 100 ft [30 m] from the vehicular intersection. There are other elements that need to be considered in this type of crossing, including right of way assignment, appropriate traffic control devices, sight distance, refuge islands and pavement markings. The preferred intersecting angle for this type of crossing is 90^0.

Adjacent path intersections occur when the path is parallel to a roadway and it crosses a driveway or other intersecting roadway such as a T-intersection or a simple four-legged intersection. In designing this type of crossing it is important to keep the location close to the intersection. This allows the motorist and path user to recognize they are a part of the traffic mix and to be prepared to react accordingly. In this situation, the user is faced with multiple conflicts.

The major roadway may be the parallel route or the intersecting roadway. It is necessary to clearly definite rights of way and the appropriate traffic control devices adjusted to reflect the addition of the shared path component in the intersection flow. Considerable traffic engineering must be utilized to make this type of crossing safe.

Complex intersections are defined as all other types of intersections of paths with roadways. These intersections are unique and must be designed as site specific. Several alternative treatments are available such as moving the crossing, installing a signal, changing signalization timing, or providing a refuge island.

If these or other safe solutions are not possible, it may be preferable to have the cyclist dismount and walk the bicycle across the intersection. Designers should insure that adequate signing is in place to alert both vehicles and bicycle users of their design intent.

In most cases, the cost of grade separating the shared use path from the highway will be cost prohibitive. However, for crossings of freeways and other high speed, high volume arterials, a grade separation structure may be the only possible or practical treatment.

When intersections occur at grade, a major consideration is the establishment of right-of-way. The type of traffic control to be used (signal, stop sign, yield sign, etc.), and location, should be provided in accordance with the MUTCD.

Sign type, size and location should also be in accordance with the MUTCD. Care should be taken to ensure that shared use path signs are located so that motorists are not confused by them and that roadway signs are placed so that shared use path users are not confused by them.

It is preferable that the crossing of a shared path and a roadway be at a location away from the influence of intersections with other highways. Controlling vehicle movements at such intersections is more easily and safely accomplished through the application of standard traffic control devices and normal rules of the road. Where physical constraints prohibit such independent intersections, the crossings may be at or adjacent to the pedestrian crossing. Who yields the right of way should be assigned and sight distance provided so as to minimize the potential for conflict resulting from unconventional turning movements. At crossings of high-volume, multi-lane arterial highways where signals are not warranted, consideration should be given to providing a median refuge area for the shared path user. Where shared use paths intersect at highway intersections it may be preferable for the bicyclist to dismount and walk the bicycle through the intersection using pedestrian crosswalks. Adequate signing must be provided to alert the bicyclist of this condition.

When shared use paths terminate at existing roads, it is important to integrate the path into the existing system of roadways. Care should be taken to properly design the terminals to transition the traffic into a safe merging or diverging situation. Appropriate signing is necessary to warn and direct both bicyclists and motorists regarding these transition areas.

Bicycle path intersections and approaches should be on relatively flat grades. Stopping sight distances at intersections should be checked and adequate warning should be given to permit bicyclists to stop before reaching the intersection, especially on downgrades.

Curb ramps at intersections should be the same width as the shared use path. Curb ramps should provide a smooth transition between the shared use path and the roadway.

10.9.6.8 RESTRICTION OF MOTOR VEHICLE TRAFFIC

Shared use paths need some form of physical barrier at highway intersections to prevent unauthorized motor vehicles from using the facilities. Provisions can be made for a lockable, removable post to permit entrance by authorized vehicles. The post should be permanently reflectorized for nighttime visibility and painted a bright color for improved daytime visibility. When more than one post is used, 5 ft [1.5 m] spacing is desirable. Wider spacing can allow entry to motor vehicles, while narrower spacing might prevent entry by adult tricycles and bicycles with trailers.

An alternative method of restricting entry of motor vehicles is to split the entryway into two 5 ft [1.5 m] sections separated by low landscaping. Emergency vehicles can still enter if necessary by straddling the landscaping. The higher maintenance costs associated with landscaping should be considered.

10.9.6.9 OTHER DESIGN ISSUES

The preferred pavement surface is a good quality all weather surface. Designing the pavement structure is similar to that of a roadway. Design is based on soil investigation to determine the load carrying capacity of the existing soils. In this case, the controlling load is that of motorized maintenance and patrol vehicles. The pavement selection is influenced by frost damage potential, skid resistance, surface quality, edge support, and surface and subsurface drainage.

Motor vehicle grade separations pose a design problem for the shared use path. The first consideration is whether or not safe continuity can be provided, particularly on existing structures. A clear width equal to the approach width of the shared use path plus a lateral offset of 2 ft [0.6 m] is desirable. Vertical clearances are controlled by the motor vehicle traffic. Railing, fences or barriers on both sides of the path across the structures should be a minimum of 44 in [1.1 m] high. Expansion joints should be "bicycle friendly".

Drainage design for shared paths is similar to that of a roadway. A cross slope of 2 percent in one direction with no crown is preferred and also simplifies the construction. Side ditches, ground cover, erosion and all other drainage design elements are a part of the path design.

Designating a sidewalk as a shared use path is not recommended, even if the sidewalk is wider than normal. The introduction of a diversity of users (bicyclists, rollerbladers, etc.) and their particular operational characteristics will cause safety problems for all users. Sidewalks as a shared use path should be limited to a high speed or heavily traveled area or across long narrow structures where continuity of the path is desirable.

10.10 BUS STOPS

The Delaware Transit Corporation (DTC) establishes policy and design guidelines for bus stops and other transit related facilities. As part of the project development process highways and corridors served by transit will be identified and appropriate facilities included in the project. The following is a general discussion on bus stops. For specifics, the designer should refer to the Delaware Transit Corporation's Policy: *Bus Stop and Passenger Facilities Standards."*

10.10.1 LOCATION CRITERIA

Bus stops are generally located at or near major trip generators, destinations or at regular intervals based on the population density and other related demographic transit related criteria. Stops are located where passengers can board and alight safely and where buses can safely enter and exit the traffic flow. Sidewalks and walkways serving bus stops should separate

pedestrians from vehicular traffic. Whenever, possible, stops in opposite directions on a route should be located directly opposite each other.

Bus stops should not obstruct driveways or entranceways nor cause visual obstruction for motorists or the bus driver merging back into traffic. In areas that have high traffic volumes, significant turning movements and pedestrian crossings, the stop should be placed where it will present the least conflict.

10.10.2 BUS STOP DESIGN

The three basic configurations for bus stops are (see Figure 10-10):

- Far-side (placed immediately after an intersection),
- Near-side (placed immediately before an intersection), and
- Mid-block (placed between intersections or along the side of a stretch of roadway).

All three of these configurations have advantages and disadvantages. Of the three, far-side curb stops, those placed at the far side of the intersection, are preferred in high traffic areas because they allow for better bus operation and traffic and pedestrian flow. After determining the stop configuration, the bus stop zone design type must be selected. Figures 10-11 through 10-17 show the layout and design parameters for the five types of stop zones. These are:

- Curbside—those placed in parking or running lanes adjacent to the curb.
- Bus Bay—a separate lane outside the influence of an intersection, with appropriately designed deceleration and acceleration lanes.
- Open Bus Bay—a separate lane on the far side of an intersection with an acceleration lane. The intersection area is used as the deceleration lane.
- Queue Jumper Bus Bay—a separate lane on the far side of an intersection where the right turn lane is used by the bus to by-pass the signal queue. An appropriate acceleration lane is provided.
- Nub—In a location with on-street parking, the curb is extended toward the travel lane, removing available parking, creating an area large enough to permit the bus to stop at curb side, in the running lane.

Bus bay, open bus bay and queue jumper bus bay designs provide a protected area away from moving traffic and minimize delay to through traffic. However, they present problems for the bus to reenter the traffic flow, are expensive to install, result in the removal of considerable parking spaces and, perhaps the greatest disadvantage, are difficult and expensive to move. The nub type has similar advantages to the curbside type. However, in most situations the disadvantages due to the operational area required, cost to install and loss of parking outweigh the advantages.

The preferred type of stop is the curb-side stop which provides easy access for the bus, results in minimal delay, is simple to design, easy and inexpensive to install and easy to relocate. The disadvantages are, when placed in the running lane, that they cause traffic to back up and may lead drivers to unsafe maneuvers to avoid the stopped bus. With advanced signing and driver expectation these disadvantages are minimized. In areas with permitted on-street parking, removal of parking spaces within the stop zone is required but not to the limits required by the other types of bus stop zones.

Curbside bus stop zones are the portions of the roadway marked and signed for use by buses. Bus stop zones should be a minimum of 90 ft [27 m] for far-side stops, 100 ft [30 m] for near side stops, and a minimum of 150 ft [45 m] for mid-block stops. Bus stop zones lengths may have to be extended depending on the rate of bus

arrivals and passenger service time at the stop. Bus bays, also known as turnouts, are specially constructed areas separated from the travel lanes and off the normal roadway section. The design allows through traffic to flow freely when encountering a stopped bus. Bus bays are usually constructed on high-volume or high-speed roadways or heavily congested downtown shopping areas with large rider usage. On high volume roadways bus bays must be designed properly or they may be avoided by the bus driver due to the extreme difficulty maneuvering and lost time required in returning to the traffic flow.

Bus bay design is based on criteria that considers the established roadway speed, the entering bus speed, the required deceleration acceleration and tapers to allow for entrance and exit of the bus, plus the required stopping area allowed for boarding and alighting. The higher the traffic speeds the greater all these dimensions plus the lane width become. In urban areas far-side intersection placement of bus bays is preferred.

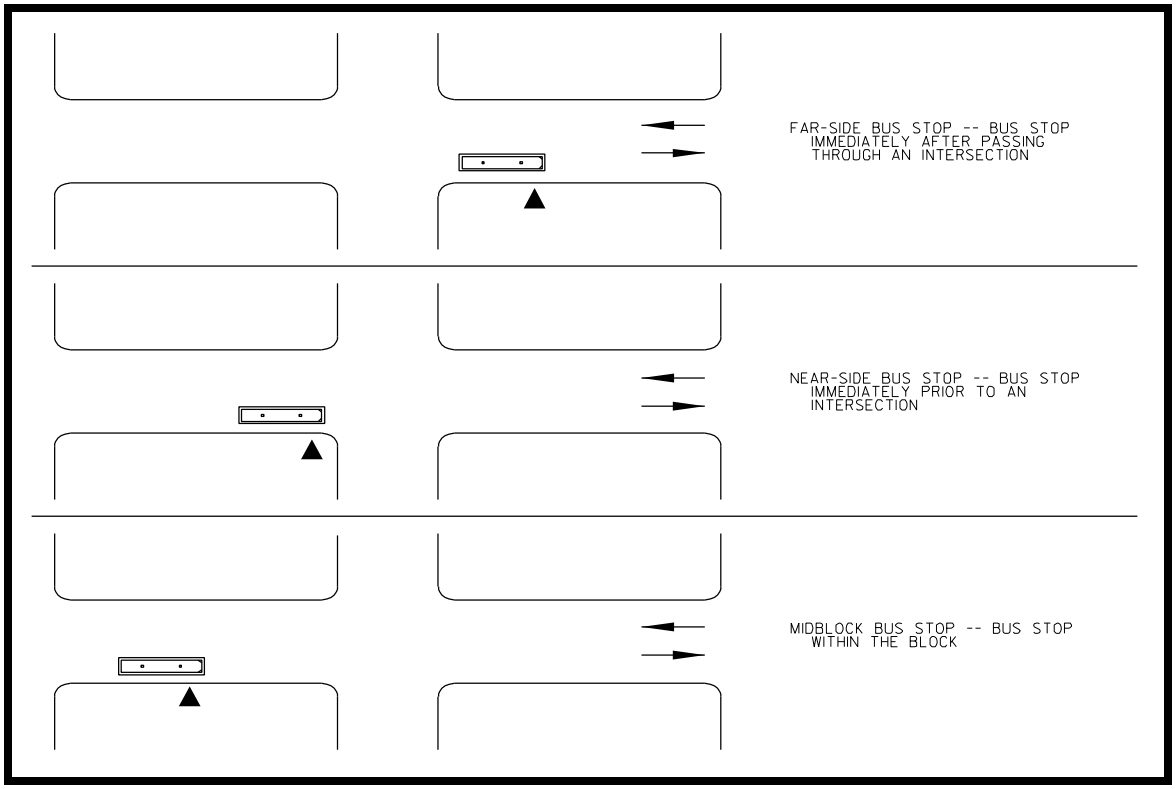
The open bus bay design is a variation of the bus bay design with the entrance end open to the upstream intersection. The bus driver has the pavement width of the upstream cross street available to decelerate and to move into the bay. The advantages of this design are that it allows efficient bus movement and permits free flow of traffic. The disadvantage is to the crossing pedestrians who have a greater distance to cross the intersection and encounter another potential conflict, particularly if the impatient vehicle driver decides to use this lane to avoid the queue. To minimize the disadvantages, a partial-open bus bay design may be considered. This alternative maintains the roadway curb line at the far side of the intersection creating a bubble that allows the buses to use the intersection approach in entering the bay and provides a partial sidewalk extension to reduce pedestrian crossing distance.

Queue jumper bus bays provide priority treatment for buses along arterial streets by allowing buses to bypass the queue at intersections. These bus stops consist of a near-side and far-side open bus bay. Buses are allowed to use the right turn lane to bypass congestion and proceed through the intersection. The right turn lane is appropriately signed to allow for this maneuver. The benefits of queue jumper stops are that they remove the bus from congested intersections and installation costs are affordable. When right turn lane volumes becomes too high an exclusive bus lane may be needed.

Nubs are sections of sidewalk that taper from the curbed parking lane to the edge of the travel lane creating an area prior to and around the intersection radius. The bus operates similar to curbside stops. Nubs are used on low speed and/or low volume streets where right turn lanes are not warranted. They are considered at locations of high pedestrian activity, crowded sidewalks, where pedestrian crossing distances need to be decreased, and where stopped buses in the travel lane can be tolerated. Nubs can be used both as bus stops and traffic calming measures.

Once a bus stop configuration is selected a design vehicle representative of those in the fleet must be determined. The most commonly used bus is the standard 40 ft [12 m] bus. Articulated 60 ft [18 m] buses may be a part of the fleet. Normally the dimensions typical for a standard 40 ft [12 m] bus are used for design. The design features that are affected by the design vehicle decision are lane and shoulder widths, lateral and vertical clearances, vehicle storage lengths, minimum turning radii, and the pavement strength.

Figure 10-10
Bus Stop Placement



Safe pedestrian access to the bus stop is important. Placing stops where there is an existing sidewalk network meeting all the criteria of the Americans with Disabilities Act to accommodate the disabled and physically challenged is the desired location. The accessible path to the stop should be well drained and, where possible, placed in street lighted areas.

The pathway to the bus stop should have the following characteristics:

- Clear width of at least 3 ft [0.9 m], preferably 5 ft [1.5 m] (the minimum width needed to allow passage of two wheelchairs)
- Running slope of pathway can be no greater than 8%
- The surface of the pathway must be firm and well-drained
- The path must comply with accessibility guidelines for curb cut at all street intersections.

10.10.2.1 BUS SHELTER SETBACK

Whether or not to install bus shelters is a decision of the Delaware Transit orporation (DTC). Primarily the number of passenger boardings per day guides this decision. However, consideration is also given to the number of transfers at a stop, the volume and frequency of transit service, the number of disabled and elderly users, the proximity to major activity areas and available space. More detail is available in DTC policy *Bus Stops and Passenger Facilities Standards.*

When DTC determines that a bus shelter is to be included in a project, then the clear zone principles found in the *Roadside Design Guide* are to be applied. Unless designed as a breakaway unit, the bus shelter is considered a fixed object. The bus shelter

location than becomes a function of design speed, traffic volumes, cross slopes and roadway curvature. The following guidance is to be used in locating a bus stop:

- Projects with a design speed of 60 mph [100 km/h] or higher should have any bus shelters located off-site.
- If the design speed is 55 mph [90 km/h], a bus shelter should be off-site or outside the clear zone without curbing.
- If the design speed is 50 mph [80 km/h], or less (such as approaching or in an urban area), a bus shelter should (a) be outside the clear zone without curbing, (b) be setback a minimum of 5 ft [1.5 m] from a barrier curb or (c) be setback a minimum of 8 ft [2.4 m] from the face of curb if the curb is depressed for the disadvantaged.
- Off-site shelters located in park-and-ride lots or transit hubs should conform to the requirements set for sites in areas with design speeds of 50 mph [80 km/h], or less.
- If a barrier curb is used, it should be extended upstream from the shelter a distance as required by Section 5.6.4 of the Roadside Design Guide. This distance is generally between 110 ft [33 m] and 210 ft [63 m] depending on the traffic volume, design speed and shelter setback.
- If the setback can not be met, a design exception with necessary documentation and justification should be processed.

10.11 PARK-AND-RIDE LOTS

DTC is responsible for establishing and maintaining park-and-ride lots. DTC is involved in the project development process to ensure site feasibility and conformance with intended use. This section is a general discussion on park-and ride facilities. The designer should refer to Delaware Transit Corporation's Policy: *Bus Stops and Passenger Facilities Standards*.

Park-and-ride lots are intended to provide a common location for individuals to transfer from low- to high-occupancy travel modes. The overall objective is to maximize the efficiency of the transportation system and to provide commuter options. In addition to serving locations to transfer automobile users to a transit facility, they are created at selected locations to encourage the formation of carpools and vanpools.

In providing this travel alternative, there are five types of park-and ride facilities; (1) remote, (2) local service, (3) peripheral, (4) exclusive use, and (5) shared-use. Remote park-and-ride lots are located relatively far from a major activity center and/or the final destination of users. Remote lots serve residents of rural and suburban areas and community centers allowing them to travel to and from central business districts or other high employment centers in a mode other than a single occupancy vehicle (SOV). To be successful they need to intercept automobiles close to the residential area or home. In order to do this, they are usually located relatively close to heavily traveled corridors.

Local service park-and-ride lots are located at the end of a local transit route. These lots are located closer to desired destinations and serve residential neighborhoods at the end of the routes, as well as those along the route. Peripheral park-and-ride lots are located on the edge of a major activity center. These lots function to expand the available parking and to intercept automobiles before entering the congested area. These types of facilities do not eliminate the commute trip. The last part of the trip into the most congested areas is made by transit. Ideally carpools and vanpools form at peripheral lots to use this facility for transfer to transit serving individual destinations.

Exclusive use park-and-ride lots are planned, designed, constructed and operated specifically to serve as park-and-ride lots.

Remote park-and-ride lots are usually exclusive facilities. However, these types of facilities are more generally associated with rail systems, high occupancy vehicle lanes, or express bus services. These lots have a high use rate, provide passenger amenities, and have frequent peak-hour service. Because they are designed exclusively to serve as a park-and-ride, provisions for adequate parking, efficient bus operation, pedestrian circulation and safety are a necessity. In order to accomplish this, these facilities have a high cost.

Shared-use lots serve several functions with only a portion designated as a park-and ride area. They may be located in a shopping center, church, school, or other activity center parking lot. Shared-use lots are usually located along an existing bus route. The advantage of this type of lot is that it is low in cost and can provide a low cost means of determining the demand for this type of service. The disadvantages include the need to negotiate a formal agreement to cover rent, maintenance, repairs, and termination of use. In addition, the lot may not be laid out or easily adapted to serve transit and its associated pedestrian-automobile conflicts, particularly during peak business seasons.

The need for park-and-ride facilities is influenced by many factors. The following characteristics influence utilization:

- A concentration of work trips to a central location, High levels of traffic congestion on destination routes,
- Limited, inconvenient and/or expensive parking at destinations,
- Frequent transit service available from the park-and-ride facility, and
- Time savings due to preferential treatment of high occupancy vehicles.

In most cases, park-and-ride facilities and the supporting transit services are oriented to a major destination, usually the central business district (CBD). However, in some cases service may be oriented to another major activity center, or service may be provided to more than one destination. General guidelines are discussed in this section. Refer to AASHTO's *Guide for the Design of High-Occupancy Vehicle Facilities* and *Guide for the Design of Park-and-Ride Facilities* and FHWA's *Park-and Ride Facilities: Guide for Planning, Design and Operation* for more detailed information.

Figure 10-11

Street-Side Bus Stop Designs

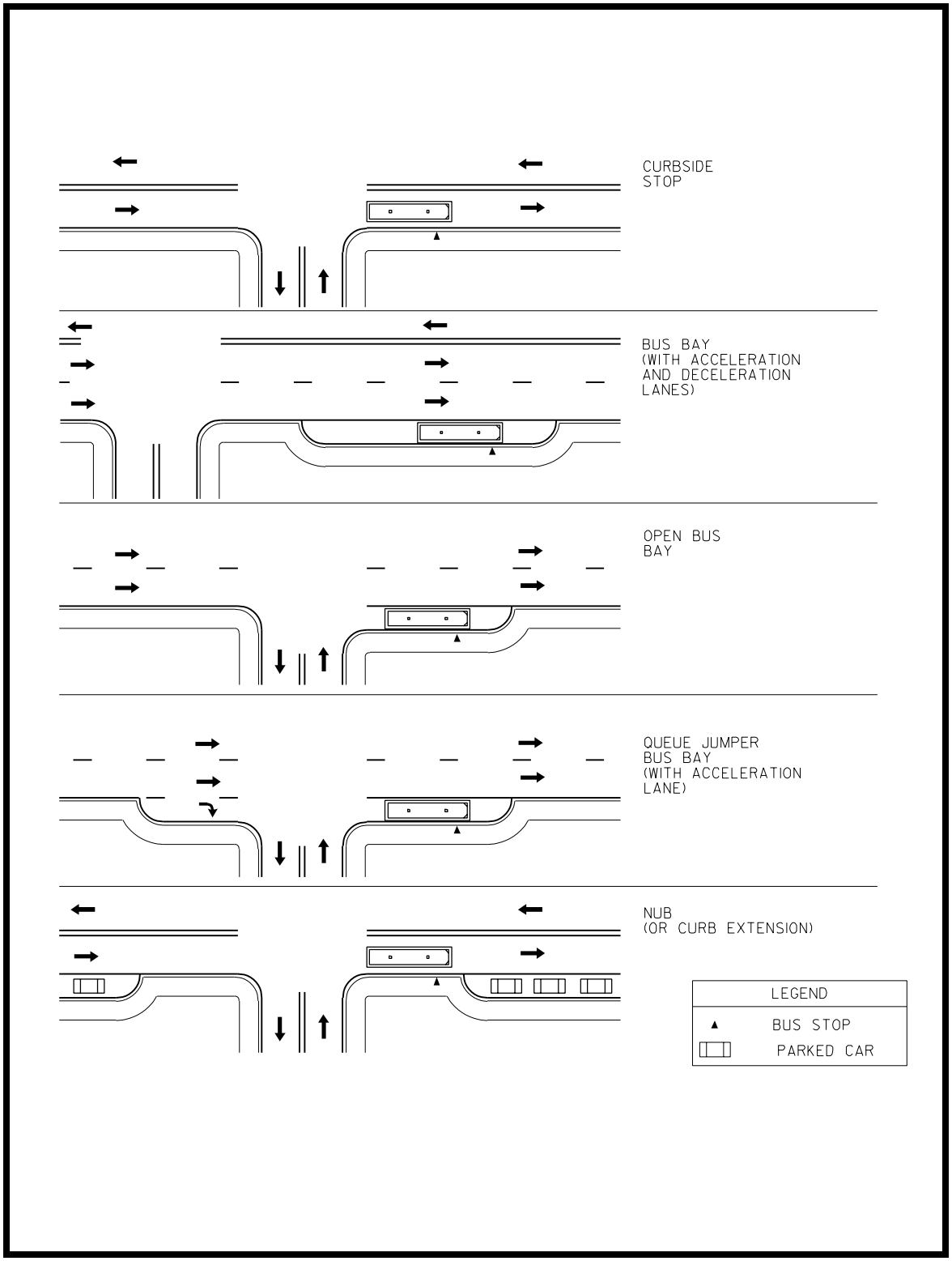
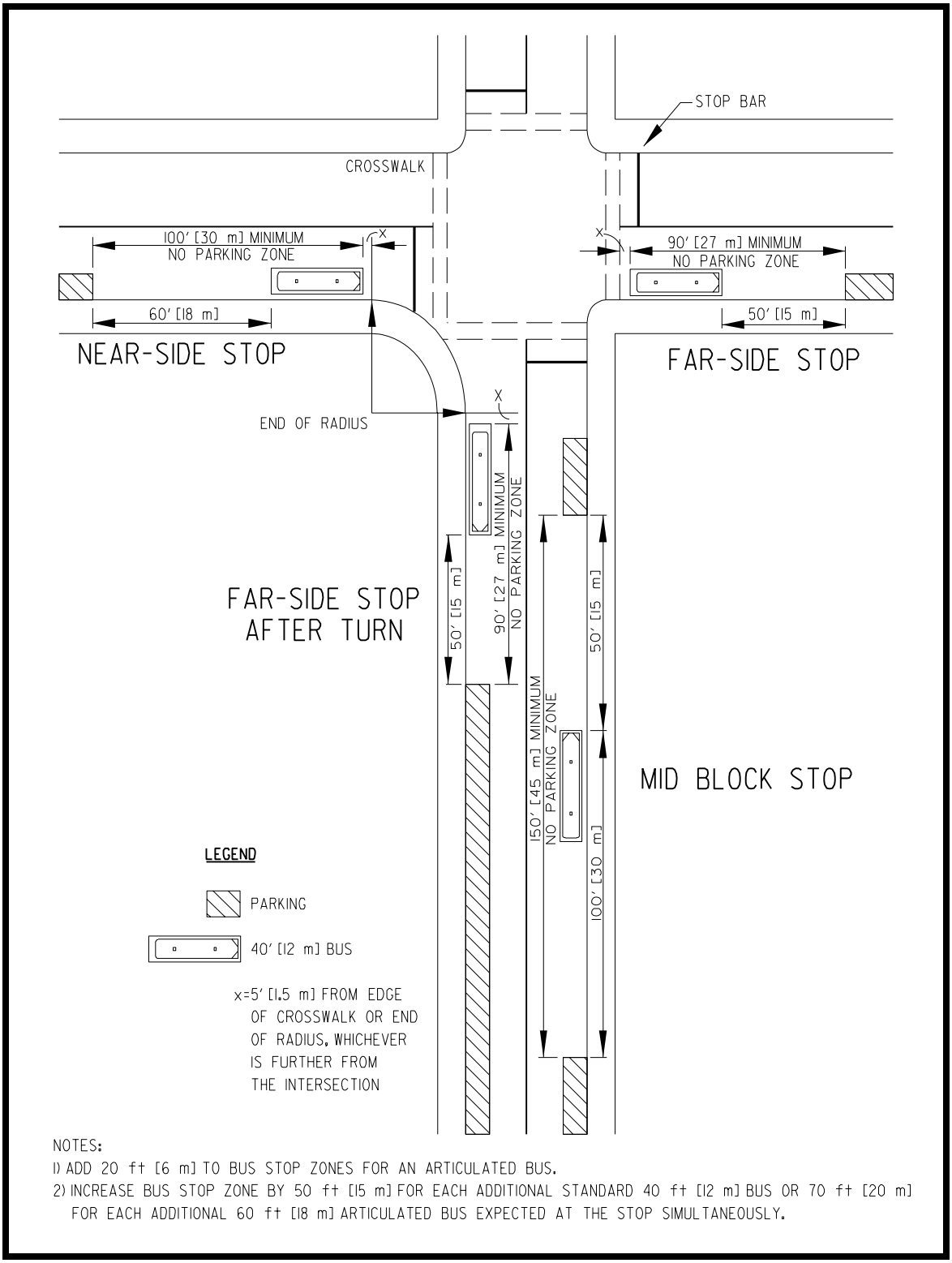


Figure 10-12
Typical Dimensions for On-Street Bus Stops



NOTES:
1) ADD 20 ft [6 m] TO BUS STOP ZONES FOR AN ARTICULATED BUS.
2) INCREASE BUS STOP ZONE BY 50 ft [15 m] FOR EACH ADDITIONAL STANDARD 40 ft [12 m] BUS OR 70 ft [20 m] FOR EACH ADDITIONAL 60 ft [18 m] ARTICULATED BUS EXPECTED AT THE STOP SIMULTANEOUSLY.

Figure 10-13
Typical Bus Bay Layout

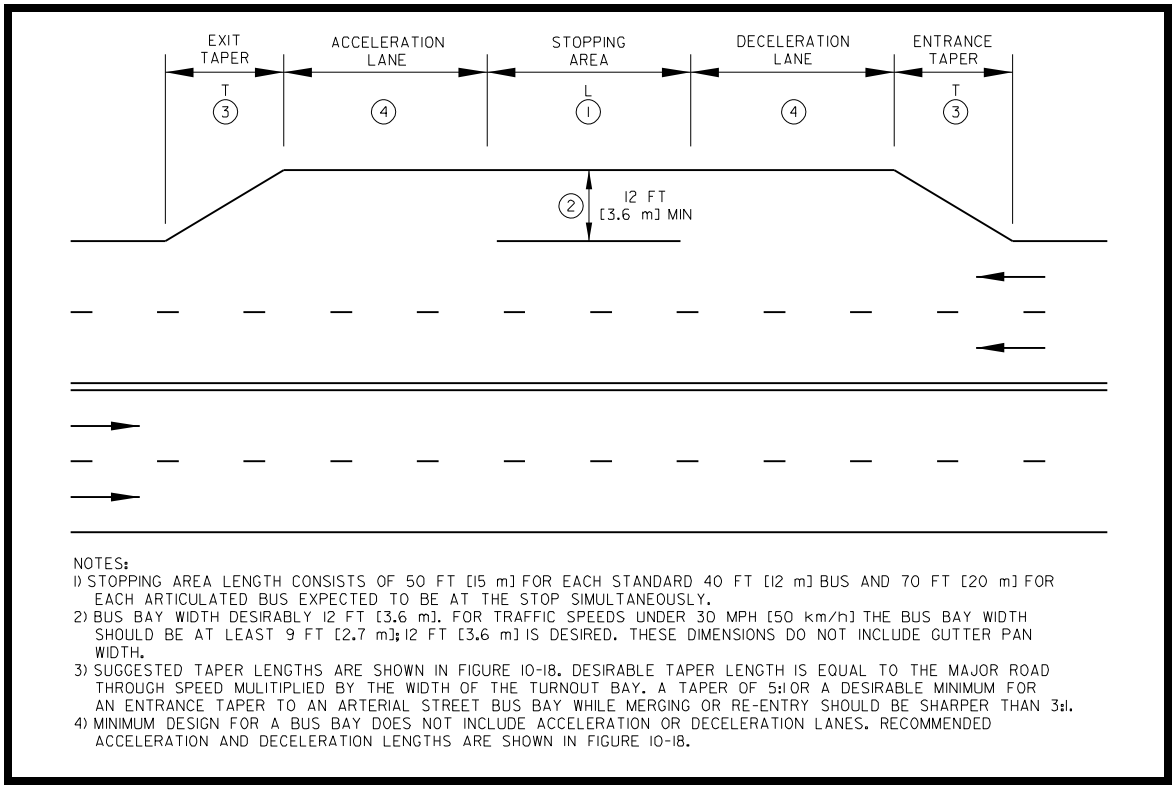


Figure 10-14
Typical Bus Bay Dimensions

Through Speed mph [km/h]	Entering Speed[a] mph [km/h]	Length of Acceleration Lane ft [m]	Length of Deceleration[b] Lane ft [m]	Length of Taper ft [m]
35 [55]	25 [40]	250 [75]	184 [55]	170 [50]
40 [60]	30 [50]	400 [120]	265 [80]	190 [60]
45 [70]	35 [55]	700 [215]	360 [110]	210 [65]
50 [80]	40 [60]	975 [300]	470 [145]	230 [70]
55 [90]	45 [70]	1400 [425]	595 [180]	250 [75]
60 [100]	50 [80]	1900 [580]	735 [225]	270 [80]

[a] Bus speed at end of taper, desirable for buses to be within 10 mph [15km/h] of travel lane vehicle speed at end of the taper.

[b] Based on 2.5 mph/sec [5 km/sec] deceleration rate.

Figure 10-15
Partial Open Bus Bay

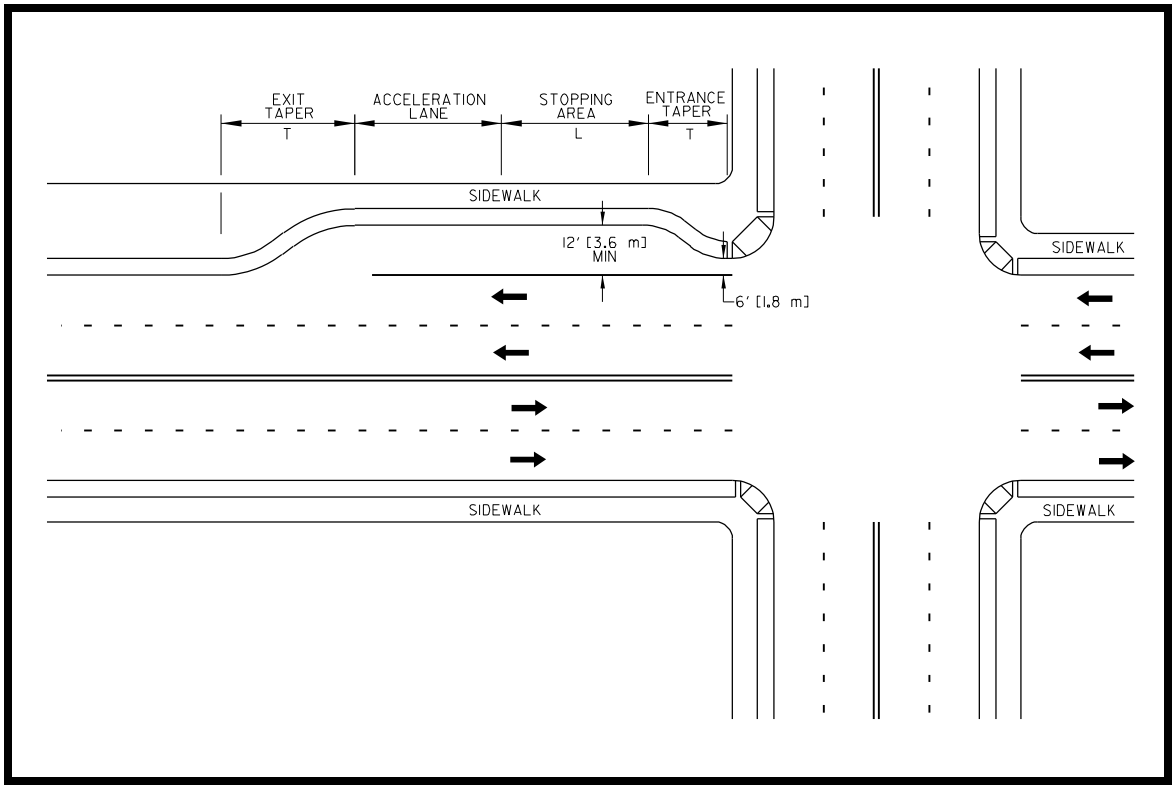
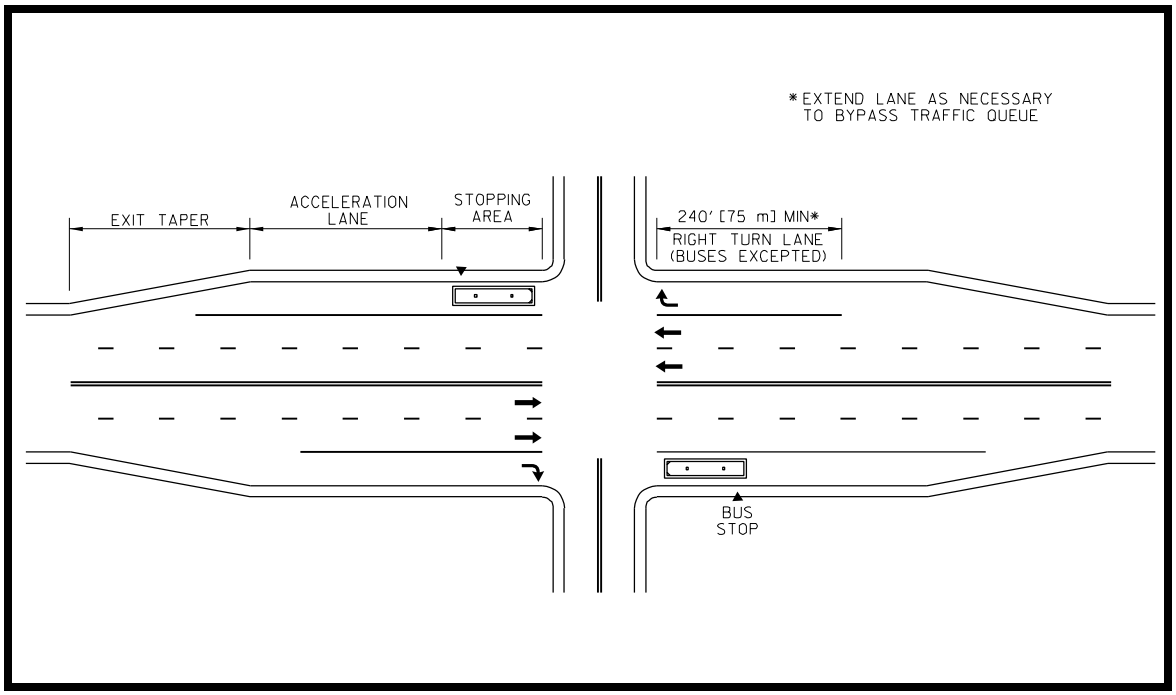
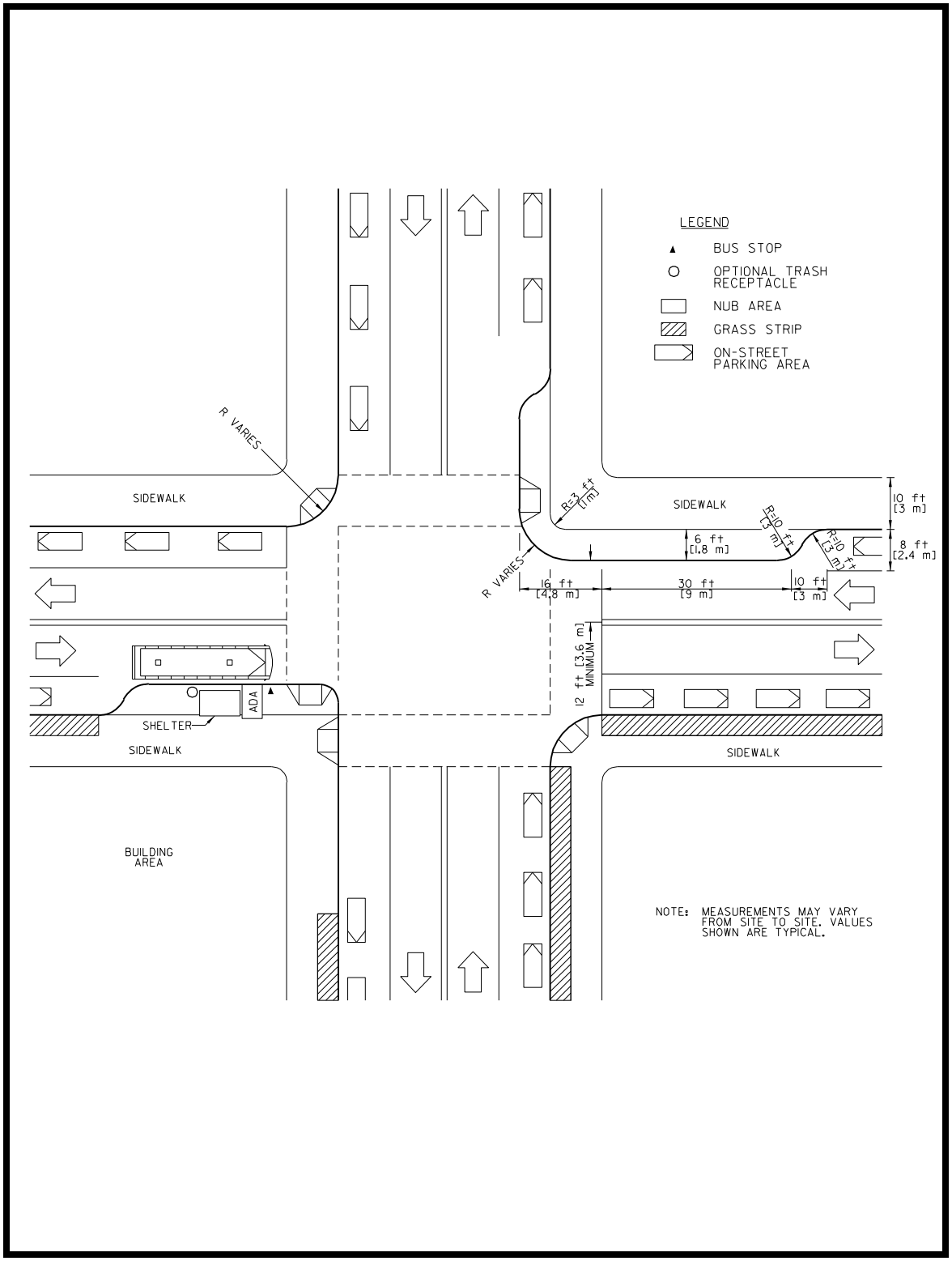


Figure 10-16
Queue Jumper Bus Bay



Notes on Figure 10-13 are applicable to Figures 10-15 and 10-16.

Figure 10-17
Nub Bus Bay



10.11.1 LOCATION

Lot location is a very critical element in the success of a park-and-ride lot. DTC, the project management team, the public, local agencies and communities are involved in this decision. The general rules for locating park-and-ride facilities are (1) locate them where there is easy access to and egress from the lot for both transit and automobiles and (2) locate them where they will be used e.g. in the desired direction of and along the route of the users. Other factors considered are the geographical location, the site conditions, the anticipated site improvement costs, and the potential user costs in out of pocket cost and personnel time.

Geographical considerations are:

- Is it within a densely used transportation corridor and close to the major roadway(s)?
- Is it located with good site visibility from the major roadway(s)?
- Is the distance to activity centers great enough to encourage its use?
- Is it conveniently accessible—on the desired travel path with easy ingress and egress?
- Does it fit into local traffic patterns?
- Does it fit acceptable commuter driving patterns?
- Does it or can it be tied into bicycle routes?
- Is it located prior to congested roadways with easy access?
- Is the site near or can it be served by a transit service?

Site considerations have varied factors such as:

- Impact on the environment and local communities i.e. is it perceived as a community asset?
- Is the site large enough to serve the projected usage?
- If successful, is there room for expansion?
- Does the adjacent area and streets have parking that is more convenient than a park-and-ride site?
- Is or can the site be made secure?

Several cost variables must be considered including:

- Land purchase value.
- Ease of acquisition, a willing seller.
- Site preparation, construction, and on-going maintenance.
- User costs—estimated vehicle operating cost to drive to the site, parking fees, and transit fares.
- User time—time from commuter's home to the site, waiting time for transfer to high-occupancy mode, and walking time to desired activity center.

One of the simplest methods of site evaluation is to observe in the field and inventory sites where people park in groups, usually at intersections or vacant lots. Addressing this need by providing better facilities at or near these areas may increase ride sharing.

Park-and-ride lots and amenities for transfer of passengers to transit routes are coordinated by DTC.

10.11.2 DESIGN

The designer must consider the use to be made of the facility. The design of a park-and-ride lot for car and vanpoolers can be quite simple, a convenient, graveled or paved parking area with safe access to an arterial route (via collector or local roads).

Design considerations are more complex when the lot must serve many users of

various modes including short and long-term commuters, buses, pedestrians, bicyclists and motorcyclists, and maybe even a kiss-and-ride option. The design of the lot should provide for each mode. The primary concern during the design phase is safe, efficient transfer and traffic flow for all potential users and transportation modes. Where possible, the differing arrival modes should be allowed separate access points. In addition, the site should have provisions for adequate numbers of useable parking spaces, facilities that are comfortable, safe, and attractive and accessibility for elderly and disabled patrons.

Parking provided in a central business district is for the convenience of both employees and customers. This creates competition for the use of park-and-rides. Park-and-rides are a user's choice, making them dependent upon perceived benefits to the user either in time and/or cost savings. This requires the designer to address the user's special characteristics and provide cost-effective designs that are safe, convenient, and reflective of other values considered by the patrons in making parking decisions.

10.11.3 ACCESS

Access to the site is an important design feature requiring internal and external traffic analysis and selection of adequate design criteria. The goals are easy maneuverability for the categories of vehicles allowed on the lot, minimizing potential traffic impacts on local roadways and/or major streets, and providing safe operations both on and off the lot.

Access to the lot should not increase congestion on the major highway that it serves nor add a major conflict point to the route. For this reason, direct access by private automobiles from the lot to a major arterial route is usually not provided. However, direct access for buses is often desirable. Usually, the most efficient access point to a park-and-ride lot will be on a collector or local street intersecting the arterial.

Locating the facility on the right side for inbound traffic on a two-way roadway will allow most users to turn right into the lot, thus eliminating the hazard of crossing an opposing traffic stream. It is likely that maximizing the accessibility for inbound trips will be more effective in attracting users than improving the flow for exiting outbound traffic. Figure 10-18 shows four typical entrance and exit configurations. It is desirable to provide separate one-way entrance and exit drives to minimize crossing conflicts. Note that the access points for the lot should be located at least 150 ft [45 m] from an intersection with the cross-street and 150 ft [45 m] between access points. When there are no more than 150 parking-stalls these distances may be reduced to 100 ft [30 m]. As parking stall capacity increases above 300 stalls, all the external and internal design parameters, circulation patterns and recognition of differing modes become more critical and must be carefully evaluated.

10.11.4 INTERNAL CIRCULATION

Major circulation routes in the lot should be located at the periphery of the parking area to minimize pedestrian-vehicle conflicts. For design, the priority sequence is feeder buses, taxis, kiss-and-ride, then park-and-ride with emphasis on the safety needs of bicyclists and pedestrians. The control vehicle in design is the bus. The ingress, egress and internal layout are most influenced by the bus's turning radius and size. Lots designed to serve cars only will probably not accommodate buses and will be avoided by the drivers. Wherever possible, buses and cars should not be mixed. Where transit service is available, circulation routes should be designed for easy bus movement, efficient terminal operation and convenient passenger transfer. A one-way roadway pattern is desirable with

two lanes provided in the bus stop area for passing the stopped bus.

When passenger-waiting areas are needed, they should be located in a central area with the various user modes surrounding the waiting area or at one end of the lot and parking for the various modes extending radially from the waiting area. Internal circulation design should give priority to fast and easy ingress and egress for transit buses, paratransit vehicles and kiss-and-ride vehicles. In shared use facilities, such as shopping centers or churches, the waiting and designated parking areas should located away from main buildings as to not interfere with normal activities.

Some of the general considerations for design of internal circulation are:

- Drivers should not be confronted with multiple choices at the same time;
- Access points should be spaced to provide for maneuvering and minimize conflicts;
- Access points should be designed to meet demand capacity;
- Flexibility to adjust to changes in operation should be available;
- The terminal/waiting area for high use lots should be located off-street but have convenient access to and from the major roadway;
- When transit services are provided and the lot also serves car/van pool formation, the car/van parking should be located in an area removed from the transit operation;
- HOV access and circulation should be separated from car access and circulation whenever possible; and
- Simple, clear signing.

10.11.5 BUSES

As previously discussed, where buses load and unload within the parking lot, traffic flow should be such that buses and automobiles do not conflict. Buses require adequate room for decelerating, proper turning radii, maneuvering into and out of an adequate loading area, and returning to the mainline traffic flow. Refer to Chapter 7 of this manual and the Green Book for design criteria such as required turning radii for bus operation.

10.11.6 KISS-AND-RIDE FACILITIES

A kiss-and-ride facility is located so that transit or commuter passengers can easily and safely access the terminal or loading zone with minimum conflicts with other vehicles; see Figure 10-19. To accomplish this, circulation in the kiss-and-ride facility should be one-way and flank the terminal or loading area. Parking should be at 45 degrees to allow for pull through and face the terminal or loading zone. To operate properly it is usually necessary to enforce kiss-and-ride restrictions.

10.11.7 PEDESTRIANS

Two pedestrian movements must be provided for park-and-ride lots that serve bus routes: a direct and safe approach from adjacent streets to the bus stop and pedestrian access from the parking area. Pedestrian circulation in parking lots is provided by aisles and crosswalks or, in larger lots, by walkways. The pedestrian path from any parking stall to the bus stop should be as direct as possible.

10.11.8 BICYCLES AND MOTORCYCLES

It is important to provide adequate bicycle storage racks at park-and-ride lots where large concentrations of bicycle traffic are expected. Similarly, a special parking area for motorcycles will improve utilization of

space. Motorcycle storage should be on Portland cement concrete to prevent stands from sinking into hot asphalt pavement. Provisions for locking both bicycles and motorcycles to prevent theft are needed. This includes bicycle racks and lockers.

10.11.9 DISABLED

At lots for transfer to buses, the design should consider provisions for safe and convenient access for the elderly and disabled. Design requirements and provisions for disabled parking shall be in conformance with the *Americans with Disabilities Act of 1990* and the State of Delaware *Architectural Accessibility Standards.*

Facilities for the disabled should also be designed in accordance with the following:

- Disabled persons should reach the bus loading zone without crossing any access roads;
- Loading areas must meet ADA space requirements;
- Disabled persons must never be forced to travel behind parked cars; and
- Suitable ramps must be provided.

10.11.10 PARKING DIMENSIONS AND LOT LAYOUT

Parking areas and roadway layout for park-and-rides can be designed in much the same manner as other parking facilities. Facilities that interact with transit, where DTC does not specify a bus size, should use a 40 foot [12 m] transit vehicle as the design vehicle. Standard dimensions for car parking stalls are shown in Figure 10-20.

For design purposes only two size stalls should be considered—standard and intermediate. If compact car parking is to be provided, it should be in a prime location or the driver will select more convenient available stalls. Combining several different types of stalls also creates an undesirable and more complicated signing layout. If there is adequate room, limiting the design to accommodate the standard stall size, 9 by 18.5 ft [2.7 by 5.6 m] is preferred. The minimum bus-parking stall should be 13 by 32 ft [4 by 15.25 m].

Figure 10-18
Typical Car Parking Dimensions

Size	Stall Width ft [m]	Stall Length ft [m]	Aisle Width ft [m]
Standard	8.5-9.5 [2.6-2.9]	18-20 [5.5-6.0]	24-26 [7.5-8.0]
Inter-mediate	8.0-9.0 [2.4-2.7]	16-18 [4.9-5.5]	22-24 [6.7-7.3]
Compact	7.5-8.5 [2.3-2.6]	15-17 [4.6-5.2]	20-22 [6.0-6.7]

Substandard stall and aisle widths are false economy. Although they permit marking more stalls in a given area, vehicles tend to encroach upon adjacent stalls such that one or more spaces are unavailable for use. The end result is no gain in actual space usage.

Vehicles and other objects should be excluded from corners or parking spots where it is necessary to provide adequate intersection sight distances. Islands at the end of rows should be considered when laying out the lot. For pedestrian safety, the parking stalls and aisles should be parallel to the direction of the desired pedestrian flow. For efficient land area use, a row of parking on each side of the aisle is preferred.

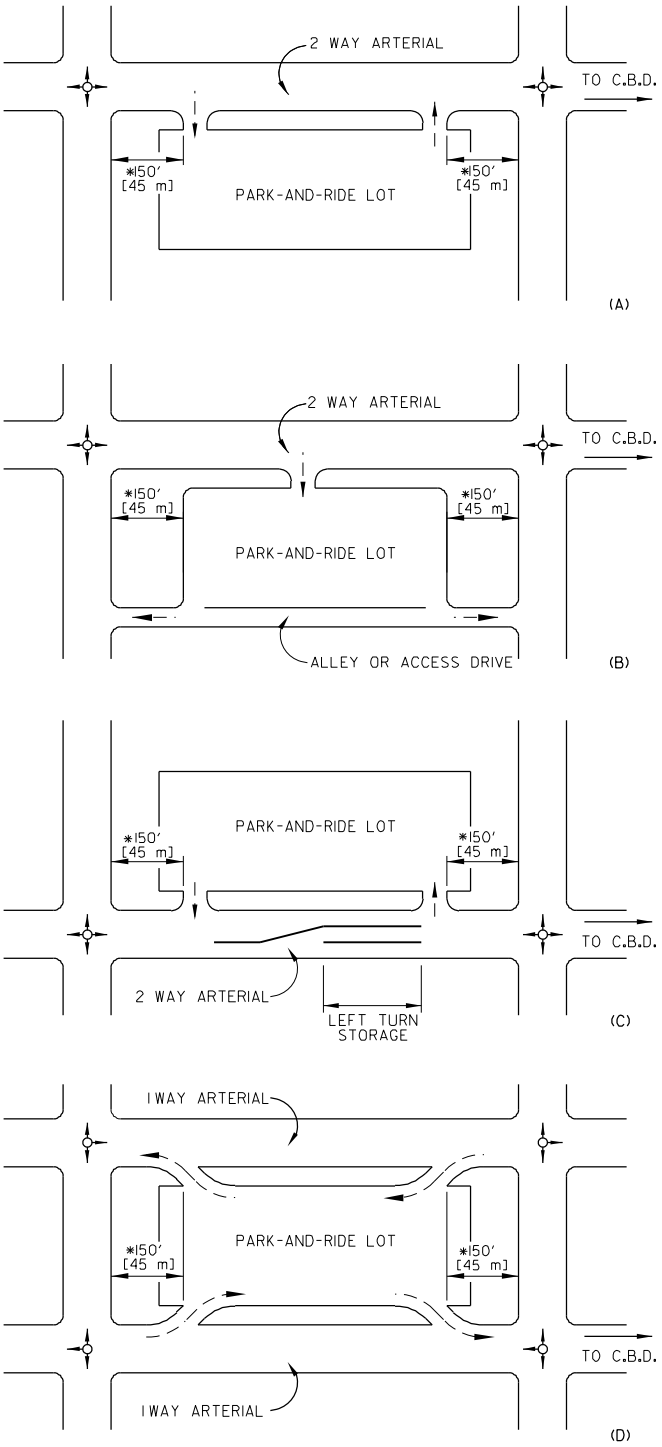
Aisle width is a function of the parking angle and stall width. One-way aisles are generally used with angle parking and two-way circulation is generally used with 90-degree parking. Aisle lengths should not exceed 400 ft [120 m] to limit pedestrian walking distance. One-way aisles should favor counterclockwise circulation with head-in parking only. Due to lower vehicle undercarriage heights, a 6 in [150 mm] curb is recommended where head-in parking is being considered. Sidewalks should be a

minimum of 5 ft [1.5 m] and loading areas should be 12 ft [3.6 m] wide. Pedestrian paths from parking spaces to loading areas should be as direct as possible. All sidewalks and curb areas are to be in conformance with ADA standards.

Figures 10-21 and 10-22 provide data for planning stall layouts for standard stall sizes of 9 by 18.5 ft [2.0 by 5.6 m]. Layouts for intermediate and compact stall sizes are available in the AASHTO guide.

The parking area should be sloped to provide positive drainage. Ponding water in a lot is undesirable for both vehicle and pedestrian movement, particularly where freezing may create icy spots. The recommended minimum grade is 1%, the desirable is 2%, and the maximum is 5%. The designer should provide adequate access and areas for snow removal and/or storage. The pavement selection needs to recognize that a variety of traffic loads, particularly when transit is expected, may be applied to the lot and the pavement type and strength designed accordingly.

Figure 10-19
Park-and-Ride Access Configuration



*MINIMUM, 350′ [107 m] DESIRABLE
C.B.D.=CENTRAL BUSINESS DISTRICT

Figure 10-20
Example of Kiss-and-Ride Parking Lot

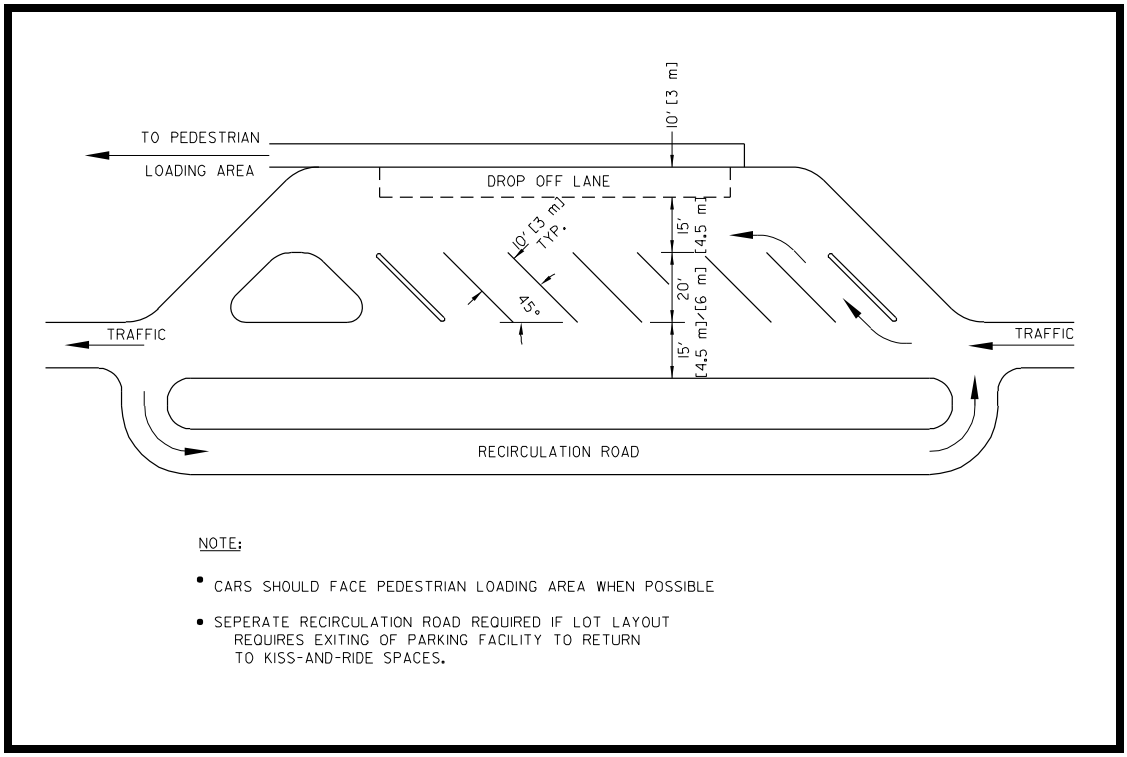


Figure 10-21
Stall Layout for Standard Vehicle (Car)

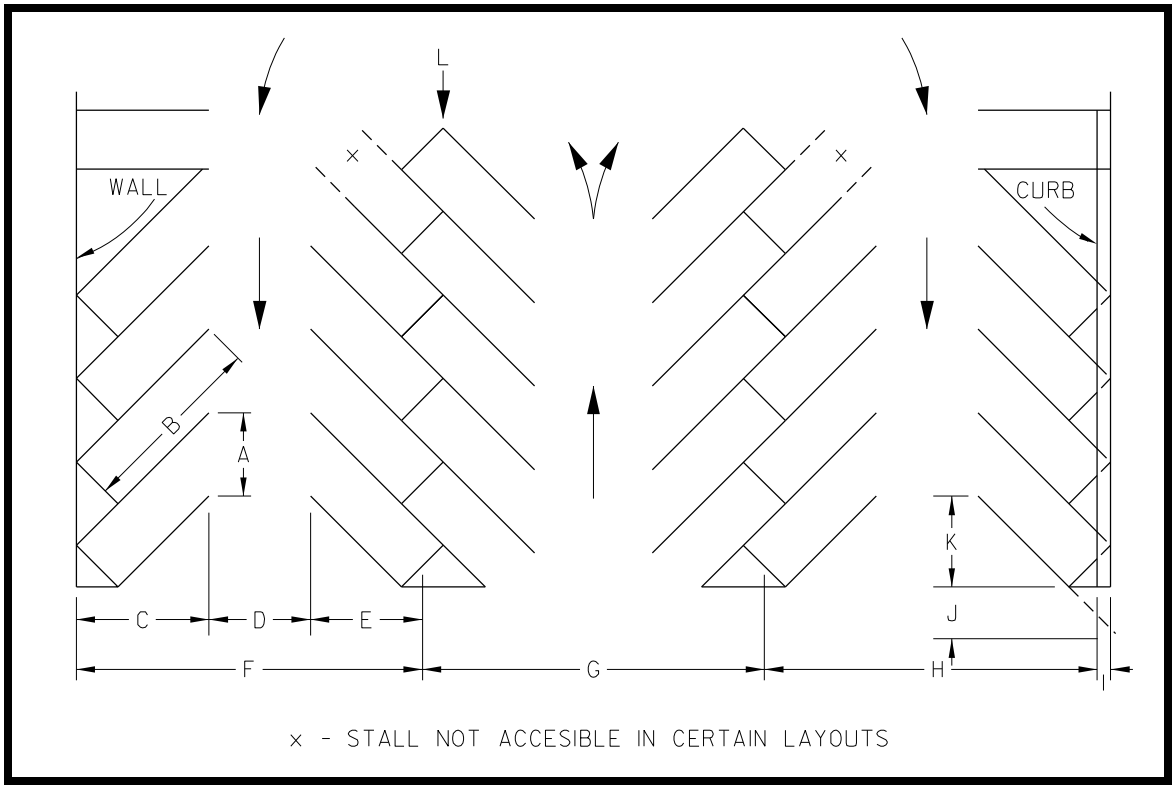


Figure 10-22
Stall Dimensions for Standard Vehicle (Car)

Dimension ft [m]	As shown in Figure 10-25	Angle			
		45^0	60^0	75^0	90^0
Stall width, parallel to aisle	A	12.7 [3.9]	10.4 [3.2]	9.3 [2.8]	9.0 [2.7]
Stall length of line	B	25.0 [7.6]	22.0 [6.7]	20.0 [6.1]	18.5 [5.6]
Stall length of line	C	17.5 [5.3]	19.0 [5.3]	19.5 [5.9]	18.5 [5.6]
Aisle width between stall line	D	12.0 [3.6]	16.0 [4.9]	23.0 [7.0]	26.0 [7.9]
Stall depth, interlock	E	15.3 [4.7]	17.5 [5.8]	18.8 [5.7]	18.5 [5.6]
Module, wall to interlock	F	44.8 [13.7]	52.5 [16.0]	61.3 [18.7]	63.0 [19.2]
Module, interlocking	G	42.6 [13]	51.0 [15.5]	61.0 [18.6]	63.0 [19.2]
Module, interlock to curb face	H	42.8 [13.0]	50.2 [15.3]	58.8 [18.0]	60.5 [18.4]
Bumper overhang (typical)	I	2.0 [0.6]	2.3 [0.7]	2.5 [0.8]	2.5 [0.8]
Offset	J	6.3 [1.9]	2.7 (0.8]	0.5 [0.2]	0.0 [0.0]
Setback	K	11.0 [3.3]	8.3 [2.5]	5.0 [1.5]	0.0 [0.0]
Cross aisle, one-way	L	14.0 [4.3]	14.0 [4.3]	14.0 [4.3]	14.0 [4.3]
Cross aisle, two-way	—	24.0 [7.3]	24.0 [7.3]	24.0 [7.3]	24.0 (7.3)

7

Appendix A

Landscaping and Reforestation Act Implementation

A1.0 INTRODUCTION

During the 2002 Legislative Session, the Delaware State Senate (141st General Assembly) passed Senate Bill Number 324: *"An Act to amend Title 17 of the Delaware Code relating to transportation construction projects requiring landscaping and reforestation activities."* In addition, the Act added a new subchapter to Title 17 that states the following:

"Therefore, this subchapter is intended to more precisely reflect a policy requiring the incorporation of landscaping and reforestation in the projects developed by the Department of Transportation for road construction and improvements in the State. This subchapter is intended to provide minimum standards for the volume of landscaping or reforestation that must take place, for how those activities must be planned, and for how much must be allocated to ensure those activities can take place. The Department of Transportation bears the responsibility for providing specific standards that are needed through regulations developed pursuant to this subchapter."

The Legislative Intent of Senate Bill Number 324 stated that *"...forested land in the State, together with landscape features such as trees, shrubs and ground covers…not only improve the aesthetic value of our State, but carry with them valuable benefits to the health and welfare of our citizens and our environment."* In addition, the Bill continued by stating: *"It is likewise declared that the Department of Transportation is a leader in replacing forestlands that are required to be cleared for such projects and in providing travelers throughout the State with scenic vistas along its roadways while maintaining safe design and construction standards."*

This legislation complements DelDOT's Context Sensitive Design policy of developing transportation solutions by balancing safety, environmental, cultural, and community needs. Further, the legislation works hand-in-hand with DelDOT's Enhancing Delaware Highways Initiative of employing native plants species in a cost effective way.

(For more information regarding the Livable Delaware Initiative see http://www.deldot.net/static/pubs_forms/manuals/livable_delaware/table_of_contents

.html. The Tree legislation can be found on the State of Delaware website: http://www.legis.state.de.us/Legislature.nsf/fsLISArchives?openframeset, under the 141st General Assembly, Senate Bill #324.)

A2.0 PURPOSE

The purpose of this appendix is to provide guidance and "specific standards" for the Department of Transportation's planners, engineers, landscape architects, and consultants' staffs in implementing the requirements of the Landscaping and Reforestation Act. It provides both technical and design guidance to assure responsible landscape and reforestation planting designs while minimizing both capital costs and the Department's perpetual site maintenance costs.

This appendix includes guidance, pursuant to the Act, that will complement previous landscaping requirements developed by the Department. It is intended to assist planners and designers in a rational basis for identifying minimum landscaping and reforestation requirements, as well as the subsequent preparation of landscape planting and reforestation plans. It also addresses tree protection and maintenance activities and procedures that should be performed to save existing trees.

A3. 0 DEFINITIONS

The legislation includes definitions for clarification of the Act. Listed below are additional definitions to assist in implementation.

Afforestation means the establishment of a forest area in an area on which forest cover has been absent for a long period of time or the planting of open areas which are not presently in forest cover. (Definition from Maryland "State Forest Conservation Technical Manual" Third Edition, 1997.) Note: The Landscaping and Reforestation Act does not specifically include the word "afforestation". However, it shall be understood that, for the purpose of this guidance, reforestation and afforestation will be performed to meet the requirements of the Act.

Critical Root Zone is a circular region measured outward from a tree trunk representing the area of the roots that must be maintained or protected for the tree's survival. For the purpose of this guidance document, critical root zone is one foot of radial distance for every inch of tree diameter (DBH) measured at 4.5 feet above the ground, with a minimum radius of eight feet. For specimen trees the critical root zone shall be 1.5 feet for every inch of tree diameter. (Definition from Maryland "State Forest Conservation Technical Manual" Third Edition, 1997.)

Qualified Professional means an individual who has the requisite educational background and technical experience to perform the Mitigation Needs Analysis. A Qualified Professional must be approved by the Department of Transportation prior to the commencement of work related to the Act.

Shade Tree means an individual deciduous or coniferous tree, typically isolated or standing alone, located in front of or adjacent to an existing residential, institutional, industrial, or commercial property, or a park setting. Shade trees have aesthetic value, provide shade and/or shel-

ter, and allow the passage of people, animals, or vehicles beneath them.

Tree, for the purpose of performing the Mitigation and Needs Analysis means an existing woody plant with a trunk that measures at least four inches in diameter at four feet above the ground. (A tree, for the purpose of replacement, mitigation, or enhancement planting, shall mean any woody plant that will eventually achieve the four-inch diameter measurement at four feet above ground. These trees may be planted as seedlings, whips, or caliper stock as typically available from growing nurseries.)

A4.0 PROCEDURES

The requirements of the Act shall be invoked whenever the Department of Transportation performs a construction project. A "construction project" means any activity undertaken, authorized, or required by the Department of Transportation through which any expressway, arterial, or collector road is:

(1) Constructed on a new alignment, or

(2) Widened by adding one or more through travel lanes or turning lanes.

A4.1 MITIGATION AND NEEDS ANALYSIS

The Act requires that an analysis be performed by the Department of Transportation or its consultant prior to the commencement of construction operations for any proposed road construction or improvement project, excluding local roads. This analysis shall identify the anticipated impact of the proposed construction activity and identify the quantity of trees to be removed. The Mitigation Needs Analysis shall be performed by a Delaware Registered Landscape Architect, or a Department approved forester, or other qualified professional. However, the planting design plans shall be in accordance with Section 211 of the Act, performed by a Delaware Registered Landscape Architect, and will require review and approval by the Roadside Environmental Group at each design review submittal level.

The plan reviewer (Delaware Registered Landscape Architect, forester, or other qualified professional) should refer to Figure 2, the "Landscaping and Reforestation Act Decision Making Flow Chart", as a first order of business in the performance of the Mitigation and Needs Analysis.

The Mitigation and Needs Analysis may be initiated with a review of aerial photographs, field surveys, or existing plans. However, it shall be mandatory that the individual performing the Mitigation and Needs Analysis visit the project site to perform a thorough evaluation of the proposed construction impacts and to physically document and count the number of trees that will require removal.

In conducting a Mitigation and Needs Analysis it is necessary to determine the type or class of road being constructed. The Mitigation Needs Analysis is limited to expressway, arterial or collector road construction projects related to new alignments or the widening by adding one or more travel or turning lanes. The road type being constructed can be determined

by referring to DelDOT's Functional Classification Maps. These maps identify road classification for all the state maintained roads within the state and are in accordance with the Federal Highway Administration Functional Classification Guidelines.

The Mitigation and Needs Analysis should be initiated prior to the Semi-Final Submittal by reviewing a set of plans that show the proposed roadway alignment or roadway modifications, including the limits of proposed disturbance (limits of cut and fill). Review of these plans can be supplemented with the review of any available existing aerial photographs, a topographic survey, or existing roadway plans.

The reviewer should identify the number of 4-inch diameter or larger trees that will be removed in the performance of the roadway construction work. (Trees are to be measured at four feet above ground as specified in the "Definitions" portion of the Act.) This quantity identification may be made from the plans (if the topographic survey includes the identification of 4-inch diameter or larger trees) or from a field review. A field visit must be performed in either case to confirm the actual number of 4-inch diameter or larger trees to be removed.

It is recommended that all trees 4-inches in diameter and greater be located on the topographic survey and included as part of the roadway construction Contract Documents. However, the identification/location of each tree to be removed will not be necessary if fifty or more 4-inch or greater trees are to be removed. Please refer to the Landscaping and Reforestation Checklist in Section 5 for tree identification.

Landscaping or reforestation plans should be prepared once the tree removal threshold is identified. These landscaping or reforestation plans must follow the prescribed replacement requirements identified in this section below and shall conform with the Department's *Road Design Manual* and the Planting Standards.

The landscape architect must refer to DelDOT's list of plant materials suitable for roadside plantings. The landscape architect may recommend the use of plants not included on this list, but the concurrence of the Roadside Environmental Group or the Environmental Studies Group will be required. The Environmental Studies Group will make plant selections for reforestation mitigations and will be administered under separate contract.

In addition to identifying the trees to be removed, the plans must identify the trees to remain and other vegetation that will require tree protection and maintenance work as part of the roadway construction contract. These plans should include a tree survey, a tree work schedule, limits of tree protection fencing and signage, tree root pruning, limits of mulching, and other necessary tree protection and maintenance details and notes.

All landscaping, reforestation, and tree protection and maintenance plans should be included for review and comment at the Semi-Final design review submittal.

A4.1.1 NO REMOVAL OR CUTTING OF EXISTING TREES

A landscape planting plan, for aesthetic enhancement, shall be prepared even when a roadway construction project will not require the removal or cutting of any existing trees. Such landscape planting plans shall be prepared by either the Department or its consultants to improve and enhance the right-of-way within the project limits. All landscape planting plans shall be in accordance with Section 211 of the Act, the Department's landscape planting guidelines and specifications, and shall be prepared, signed, and sealed by a Delaware Registered Landscape Architect.

A4.1.2 REMOVAL OF EXISTING SHADE TREES

Any existing shade trees that would be removed to accommodate a proposed widening, construction, or reconstruction of a major arterial, interstate connector, minor arterial, collector road, or a proposed new road in an urbanized area of the State would be required to be replaced on site, whenever possible. Replacement shade trees would have to be balled and burlapped nursery stock with a minimum height of 16 feet and planted in accordance with the latest edition of the Department's Standard Specifications.

Shade trees are not to be included in the tree counts for the "Removal of 10 or Fewer Trees"; "Removal of more than 10 to 49 Trees"; or the "Removal of 50 Trees or More" unless the shade tree cannot be replaced on site. In those cases the shade tree will be counted as a standard tree for tree mitigation purposes.

A4.1.3 TREE MITIGATION

Planting design shall be accomplished on site and in accordance with Section 211 of the Act. The planting design shall be sealed by a Delaware Registered Landscape Architect, and will be reviewed and approved by the Roadside Environmental Group at each design review submittal level. (Note: Refer to Section 4.2 if the planting cannot be accomplished within the project's right-of-way.). The planting design shall be in accordance with Section 211 of the Act, performed by a Delaware Registered Landscape Architect, and will require review and approval by the Roadside Environmental Group at each design review submittal level.

A4.1.4 REMOVAL OF 10 OR FEWER TREES

The removal of 10 or fewer trees for a roadway construction project will require planting at least one new tree for every tree removed. This planting shall be performed within the project's right-of-way, be responsive to site conditions, and include input from the project's stakeholders.

A4.1.5 REMOVAL OF 10 TO 49 TREES

The removal of more than 10 but fewer than 50 trees for a roadway construction project will require planting two trees for each tree removed. The replacement planting should occur within the project's right-of-way, be responsive to site conditions, and include input from the project's stakeholders.

A4.1.6 REMOVAL OF 50 TREES OR MORE

The removal of 50 or more trees for a roadway construction project will require reforestation of at least one acre of land for every acre of trees removed. Once the 50 tree removal threshold is reached, tree mitigation is translated to an area basis as opposed to individual tree ratios. As such, the reforestation requirement will also apply to fractions of acreages where 50 or more trees have been removed; equivalent fractions of acreages shall be replanted. Reforestation planting shall be performed at the rates indicated in Figure 1 below. There may be instances where, after 50 trees to be removed are counted, individual and/or isolated trees, not in woodland areas, may exist. For the purpose of calculating reforestation requirements, a 15-foot by 15-foot area (or 225 square feet per tree) can be assumed for the reforestation planting requirement.

There may also be instances where a standard reforestation planting (seedling plantings) would not be appropriate for a particular project or site condition. An example would be where the project site was in an urban or suburban environment/setting that warranted a more developed landscape planting plan to reflect the character of the community and design input by the stakeholders. In this case, a landscape planting plan could be substituted for a standard reforestation planting.

A4.2 INABILITY TO MEET REQUIREMENTS FOR LANDSCAPING OR REFORESTATION

A4.2.1 OFF-SITE PLANTING

A hierarchy has been established for landscaping and reforestation when the Department is unable to perform planting activities within the rights-of-way of the roadway construction project:

The Department must:

- First complete the balance of those landscaping and planting activities within other rights-of-way it owns or controls, beginning with rights-of-way which are along the same road and geographically closest to the construction project requiring landscaping and planting…"

- If landscaping and planting activities cannot be performed as described above, then the Department shall move outward from the construction project location "...in all directions until it has met the minimum requirements for landscaping and planting activities…."

- If landscaping and planting activities cannot be performed as described in either condition above "…within any rights-of-way it owns or controls in the State, it shall determine the number of acres of landscaping and planting activities it has been unable to mitigate and perform landscaping and planting activities in the State (or cause the same to be performed) at locations recommended by the Department of Agriculture, Forest Service, or the De-

partment of Natural Resources and Environmental Control."

A4.2.2 FEE-IN-LIEU

As a last resort to accomplish the requirements of the Act, "if the Departments of Agriculture or Natural Resources and Environmental Control are unable to identify any locations in the State for the performance of landscaping and planting activities, [the Department of Transportation shall] transfer to either the Department of Agriculture, Forest Service, or the Department of Natural Resources and Environmental Control an amount equal to the dollar value per square yard necessary to plant seedlings in the area it has been unable to mitigate to be used by either Department for conducting landscaping and planting activities in the State under programs administered by those Departments."

A4.3 EXISTING TREE PROTECTION AND MAINTENANCE

Tree protection and maintenance should be included in every road construction and improvement project when there is existing vegetation on the site. Retention of the existing vegetation provides an opportunity that most new landscape plantings cannot match.

In addition to preserving existing vegetation, every effort should be made to preserve and protect large, specimen, or champion trees that lie within a road construction or improvement project's limits. These trees should be included in the project's tree survey and the requisite tree protection and maintenance work should be incorporated into the construction documents. The landscape architect should refer to *Big Trees of Delaware (Second Edition)*, prepared by the Delaware Department of Agriculture's Forest Service. This reference document identifies Delaware's largest trees' critical measurements (trunk circumference, tree height, and crown spread) and their location within the state.

Tree protection and maintenance may include the following work items:

- Protective Fencing and Signing
- Root Pruning
- Crown Pruning
- Tree Fertilization
- Tree Watering
- Tree Wall/Well
- Mulching

Some projects may require the use of a Certified Arborist during design to assist in the development of the tree protection and maintenance plans and specifications, as well as assist the Resident Engineer during the construction period.

Figure A-1
Reforestation Site Stocking

Size	Number Required (per acre)	Approximate Spacing (ft on center)	Survivability Requirement (At the end of the third season)	
Bare root seedlings or whips	700	8x8	55%	385
Container Grown Seedling Tubes (minimum cavity width 1.5")	450	10x10	65%	290
Container Grown 1,2&3 Gallon	350	12x12	75%	260
Container Grown 5&7 gallon or 1" Caliper B&B	200	15x15	85%	170
Container Grown 15&25 Gallon or 1.5-2"Calpier B&B	100	20x20	100%	100

Notes:

1. These stocking and survival requirements are the minimum number estimated to meet the definition of forest from bare land.
2. In certain circumstances, any combination of the above mentioned stocking options, dry seeding, tree shelters, transplants, and/or natural regeneration may be appropriate strategies to fulfill the requirements of the Act. They will be evaluated on a case-by-case basis by the Department.
3. Spacing does not imply that trees or shrubs must be planted in a grid pattern.

Source: Adapted from the Maryland Department of Natural Resources' *State Forest Conservation Manual, Third Edition, 1997.*

Figure 2 - Landscaping And Reforestation Act Decision Making Flow Chart

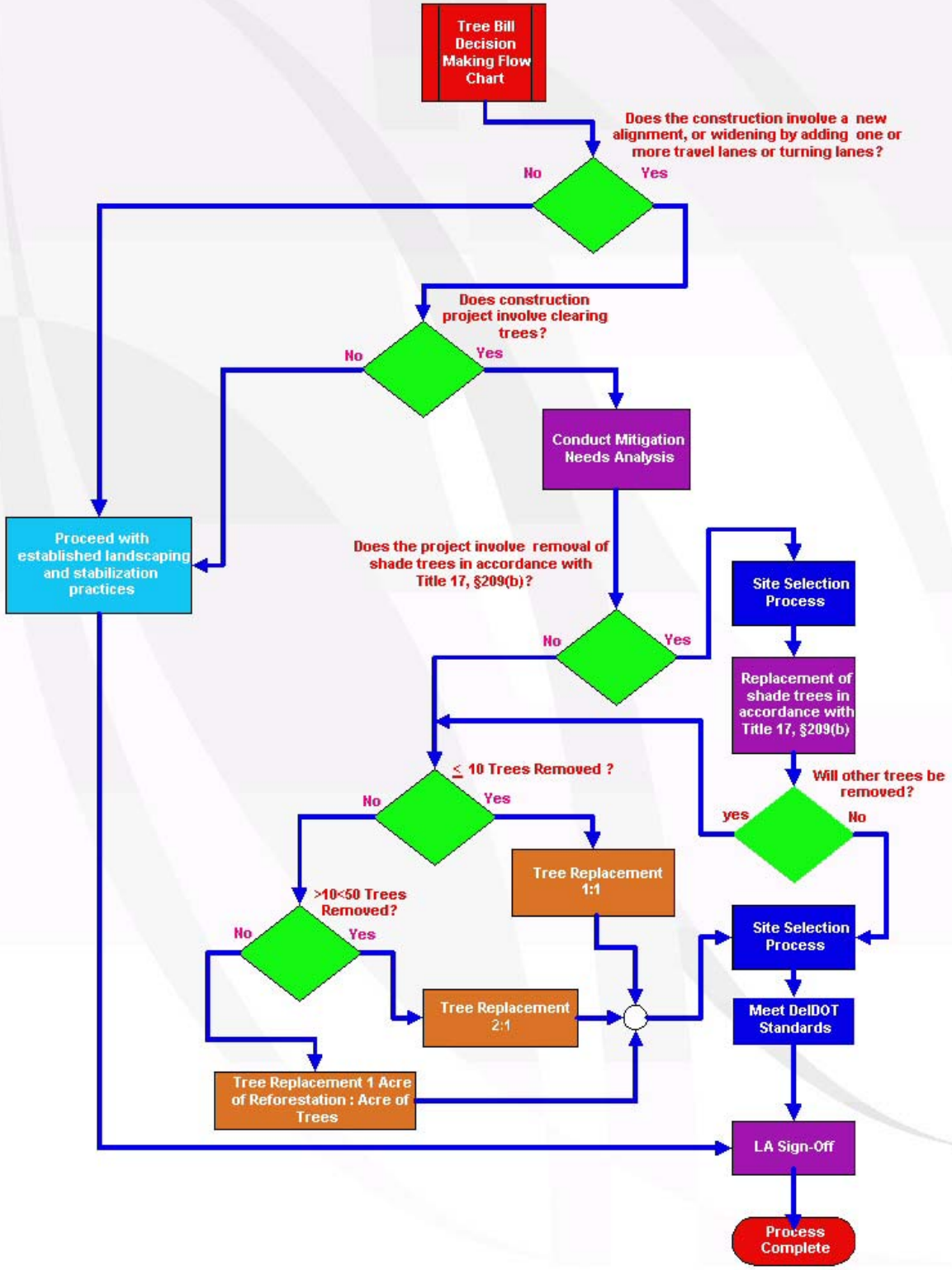


Figure A-3
Landscaping and Reforestation Checklist

Tree Number	Common Name	DBH Inches	Spread Feet	Shade Tree	Non-Shade Tree	Forest Tree	Condition
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
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