ROADWAY DESIGN MANUAL

Mississippi Department Of Transportation

2001

PREFACE

Each year Mississippi invests heavily in highway construction. How Mississippi plans and designs its highway projects is critical to the return received on this investment. This Design Manual provides uniform design criteria and procedures for the Roadway Design Division of the Mississippi Department of Transportation and for other Department personnel to use when approaching highway design projects and preparing contract plans. The information contained in this Design Manual reflects years of experience as well as the most recent national guidelines which have evolved along with the automobile and various types and classifications of roads. In earlier days, while automobiles had more power, they were heavier and traveled at lower speeds on roads that were built along the contour of the terrain. Today vehicles are lighter and faster. Also, earth moving equipment that is larger and more reliable is readily available to build higher classification roads.

With a modern transportation system in place, the on-going challenge for the Mississippi Department of Transportation is to identify ways to improve that system. Of course one way to make the system better is to make it safer. Safety is a primary objective on many of the Department's highway projects and, in all instances, is a concern. The Department seeks to improve safety by a variety of measures including, but not limited to, providing greater sight distances, greater clear zones, improved horizontal alignment, improved vertical alignment, and better anticipation of driver expectations.

However, there are other important goals as well. Due to changes in Federal Regulations, the Department of Transportation is increasing the weight it gives to avoiding environmentally sensitive areas (e.g., wetlands, historic districts, archaeological sites, etc.). In some cases, concerns of reducing potential relocatees, minimizing right-of-way acreage acquired, and limited funding are factors influencing the design of projects. Consequently, although the Design Manual is an important guide, its application must be flexible. The Design Manual's design criteria cannot envision every project's unique concerns. Above all, designers using the Manual must apply good judgment and innovation in their approach to highway design.

EXCERPT FROM THE MINUTES OF THE MEETING OF THE
MISSISSIPPI TRANSPORTATION COMMISSION, SEPTEMBER 11,2001

WHEREAS, the Commission proposed to revise and update the Roadway Design Manual adopted on April 15, 1988, and recorded in Book 106, Page 94, of the Commission Minutes. This manual incorporates in one document safety criteria and design standards or reference to such standards recommended for use in highway design; and

WHEREAS, the revised Roadway Design Manual has been reviewed and recommended for approval by the Chief Engineer and approved by the Director; and

WHEREAS, the Commission has considered and determined and now confirms that the publication and adoption of the revised Roadway Design Manual is proper and necessary.

NOW, THEREFORE, upon motion duly made and seconded, and Commissioners Wayne H. Brown, Zack Stewart and Dick Hall, each voting yes, under the authority of the Commission, in conformity with and as spread on its minutes, the revised Roadway Design Manual is hereby adopted as an official document of the commission and further that the design of highways shall conform to policies, procedures, and standards promulgated by said Roadway Design Manual.

STATE OF MISSISSIPPI

COUNTY OF HINDS

I, Linda 0. Ferrell, Secretary, Mississippi Transportation Commission, do hereby certify that the above and foregoing is a true and correct copy of an Order of the Mississippi Transportation Commission of record in Minute Book 8, Page 784, of the Official Minutes of said Commission on file in its offices in the City of Jackson, Mississippi, duly adopted on the 11th day of September, A.D., 2001.

Witness my hand and official seal this the 13th day of September A.D., 2001.

Linda O. Ferrell

LINDA 0. FERRELL, SECRETARY
TRANSPORTATION COMMISSION
STATE OF MISSISSIPPI

Chapter 1

General Design Considerations

Chapter 1

GENERAL DESIGN CONSIDERATIONS

Section		Page
1-1.0	SAFETY CONSIDERATIONS	1-1
1-1.01	Roadside Design	1-1
1-1.02	Roadway Geometrics	1-1
1-1.03	Driver Expectancy	1-2
1-2.0	DESIGN RESPONSIBILITIES	1-2
1-3.0	SOURCES OF INFORMATION	1-2
1-3.01	Technical Assistance	1-2
1-3.02	Department Publications	1-2
1-3.03	AASHTO Publications	1-3
1-3.04	FHWA Publications	1-4
1-3.05	TRB and NCHRP Reports	1-4
1-3.06	Supplier Publications	1-4
1-4.0	PROJECT FILE	1-4
1-5.0	QUANTITY ESTIMATES	1-5
1-6.0	CONTRACT PLANS	1-5

Chapter 1

GENERAL DESIGN CONSIDERATIONS

The basic objective of roadway design is to arrange the physical elements of the facility to meet the typical characteristics of drivers and vehicles. The geometric design of a highway addresses the elements of location, functional class, alignment, profile, cross section, intersections and control of access. Roadside safety, drainage, traffic control and aesthetic qualities are also incorporated into roadway design. The use of these elements in proper combination is the task of the roadway designer. The design should properly reflect driver safety, comfort and convenience. Inadequate design may cause early obsolescence or other financial loss through remedial reconstruction or maintenance.

1-1.0 SAFETY CONSIDERATIONS

1-1.01 Roadside Design

Special attention must be directed to the safety characteristics of the roadside to ensure a deliberate design rather than an unpredictable by-product of construction.

1. Slopes. Fill and cut slopes should be as flat as practical so that a vehicle leaving the roadway will have some chance for recovery. Where economically feasible, 6:1 or flatter slopes should be provided.

2. Obstructions. The area adjacent to the edge of traveled way should be free of physical obstructions which might prevent safe vehicular recovery.

3. Gore Areas. Avoid heavy structures and unyielding sign supports in the area at the divergence of two roadways.

4. Signs. Avoid unnecessary signs. Use breakaway supports. Review signing locations to ensure maximum effectiveness and safety.

5. Guardrail. The objective of guardrail is to lessen the hazard of run-off-the-road accidents. Guardrail should only be used when the result of striking an object or leaving the roadway would be more severe than striking the rail.

Roadside safety is discussed in detail in Chapter 9.

1-1.02 Roadway Geometrics

Geometric design criteria for most roadway elements, such as lane and shoulder widths, slopes, grades, curvature, clearances, etc., have been established. Tables of geometric values are presented in Chapters 2, 11, 13 and 14 for various functional classes and urban/rural locations.

1. Design Criteria. Design criteria more liberal than the indicated minimums or maximums will often increase safety and in some instances may not significantly increase the cost.

2. Consistency. Geometric design consistency is important to safety For example, a sharp curve at the end of a long tangent or between two relatively flat curves can be hazardous.

3. Sight Distance. Adequate sight distance is critical to safety. Evaluate each vertical curve, intersection, median crossover and driveway to make certain that sight distance is not restricted below the minimum criteria.

4. Critical Areas. Particular attention should be given to the geometric adequacy of certain critical areas. These locations include weaving areas, exit and entrance ramps, at-grade intersections, transitions from new facilities to existing facilities, and any locations where there is potential for wrong-way travel on a one-way facility.

1-1.03 Driver Expectancy

Driver expectancy relates to the readiness of a driver to successfully respond to common situations. Design features of a roadway should be similar to create expectancies related to common geometric, operational and route characteristics. Drivers should be able to perceive information and safely modify the speed and accuracy of their responses.

The FHWA publications *A User's Guide to Positive Guidance*, December 1981, and *Driver Expectancy in Highway Design and Traffic Operations,* May 1986, discuss driver expectancy in detail. It will familiarize the designer with the characteristics of a "typical" driver and how the highway system should communicate with the driver.

1-2.0 DESIGN RESPONSIBILITIES

The basic responsibilities of the Roadway Design Division are to prepare the detailed design for road projects and the roadway contract plans. Individual projects are assigned to specific squad leaders who perform and coordinate the design and the development of plans.

Supplementary expertise is provided within the Roadway Design Division by specialists in photogrammetry, roadside development, environmental considerations, pavement design, roadway lighting, hydraulics and geometric design. During the design process, the Roadway Design Division exchanges information with other divisions of the Department as illustrated in Figure 1-3A.

The principal purpose of this *Manual* is to guide designers in the use of uniform criteria and procedures that will consistently result in good quality road design and contract plans.

1-3.0 SOURCES OF INFORMATION

This *Manual* is a principal source of information for designers. Other sources are described below.

1-3.01 Technical Assistance

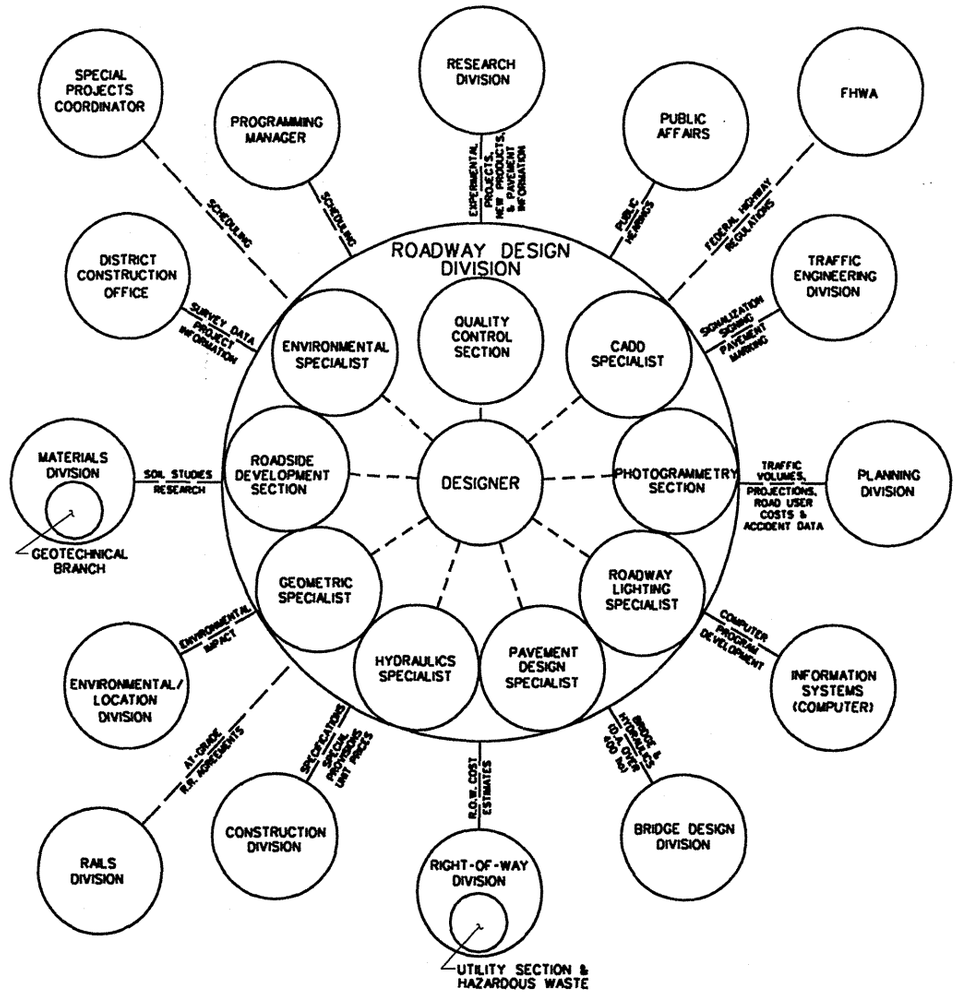
The District Engineers furnish designers with field survey notes and sketches, soils information and recommendations related to the design of specific projects. Designers will receive guidance and assistance from persons with training and competence in specialized fields as indicated by the relationships shown in Figure 1-3A.

1-3.02 Department Publications

Designers will frequently need to refer to the following materials published by the Department:

1. *Standard Specifications*. The Mississippi *Standard Specifications for Road and Bridge Construction* describes the various construction work items and the Department's relationship with contractors.

2. Special Provisions. A file of Special Provisions is maintained to identify current amendments (additions, deletions and revisions) to the *Standard Specifications*.



SOURCES OF INFORMATION FOR ROADWAY DESIGN

Figure 1-3A

3. *Standard Roadway Design Drawings and Bridge Design Drawings*. These are drawings of standardized details suitable for use in contract plans. For reference purposes, they have been bound in two separate volumes.

4. *Standard Operating Procedures*. These volumes present official Department policies and operating procedures for all functions. Roadway Design Division procedures are found in the section entitled "RWD." Additional responsibilities and procedures are found in the sections entitled "ADM" and "ENV."

5. Policy Memorandums. Designers will receive copies of memorandums defining new or revised policies and procedures. Periodically, the policies will be incorporated in revisions to the *Design Manual*.

1-3.03 AASHTO Publications

MDOT design criteria and policies adhere closely to policies established by AASHTO. This *Manual* has tailored the design criteria specifically for application in Mississippi, and it has addressed design issues not covered in the national publications. The intent is to provide a single source of information which will answer the vast majority of questions for the designer. However, the national publications provide much of the theory and background which is built into geometric design and roadside safety. Therefore, designers should be thoroughly familiar with the two publications listed below. These sources should be referenced for explanation of the concepts which serve as the basis for much of the design criteria:

1. *A Policy on Geometric Design of Highways and Streets*, 1994 (commonly known as the "Green Book").

2. *Roadside Design Guide,* 1996.

Many other AASHTO publications are available to provide authoritative guides and policies in specific areas such as pavement design, drainage design, landscaping,

rest areas, lighting, utilities, etc. Copies of these publications are available in the Roadway Design Division office.

1-3.04 FHWA Publications

This *Manual* is consistent with applicable FHWA requirements related to geometric design and roadside safety. Many of these directives have been formally published as FHWA Notices, Orders, Technical Advisories, etc., which are kept on file in the Roadway Design Division office.

The *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD), 1988, presents information on the type, location and design of traffic control devices approved by FHWA for highway use. The MUTCD is continually updated, and the designer should be aware of these revisions.

1-3.05 TRB and NCHRP Reports

The National Cooperative Highway Research Program (NCHRP) reports and various Transportation Research Board (TRB) publications, including the *Highway Capacity Manual*, cover the entire field of highway and traffic engineering. TRB publications before 1974 are identified as Highway Research Board (HRB) reports.

Copies of these reports are available in the Department's library.

1-3.06 Supplier Publications

Information on special design problems are often available through the suppliers of various construction materials. Literature may be useful in the design of drainage facilities, retaining walls, traffic barriers, traffic control devices, etc. Upon request, engineers from the Materials Division and the Construction Division will furnish the designer with information related to specific design problems.

1-4.0 PROJECT FILE

The designer will be given a file folder containing pertinent project information compiled to date including:

1. Form RWD-200, "Field Notes and Sketches." This sheet documents the inventory of all field data survey notes, right-of-way notes, incidental notes and miscellaneous maps, plans and sketches.

2. Soils Data. A report on the testing of soil samples and a graphical sketch of the soil profile throughout the proposed project is prepared and submitted by the District.

3. Form RWD-600, "Project Design Data." This sheet documents basic design data instructions for highway construction projects as prepared by the District Engineer and approved by the Roadway Design Division Engineer.

4. Special Location and Design Recommendations. Sometimes there is a need for special consideration of problems unique to a particular project. The designer will be advised of the problem, and recommended procedures and solutions will be documented.

5. Justification for Deviations from Criteria. Designers are obligated to adhere to established design criteria. Chapter 2 discusses the design exception process required when the established criteria cannot be met. The written justification becomes part of the project file.

After receiving the initial file package, the designer should maintain a current project record file with information including the field inspection reports and the miscellaneous project memorandums. The designer should also maintain a project estimate file for quantity calculations.

Upon completion of the design and contract plans, the project record file shall be retained in the Roadway Design Division for later transfer to Central File. The project estimate file may be discarded after final contract payment.

1-5.0 QUANTITY ESTIMATES

The quantity of all contract bid items shall be estimated in terms of the units of measurement in the *Standard Specifications.* Uniform criteria for degree of accuracy and rounding of computations are shown in Table 1-5A.

All computations should be documented with quantity calculations in the project estimate file.

Detailed analysis of estimated quantities are included in the contract plans for certain pay items. Examples are shown in the model plans in Chapter 15. The Summary of Quantities in the contract plans shall include all pay items to be included in the contract.

1-6.0 CONTRACT PLANS

To ensure an accurate and consistent interpretation of contract plans, it is important that the format, content and placement of information be consistent. Model contract plans are described and illustrated in Chapter 15. Designers should adhere to this format when preparing the drawings and assembling the plans.

To minimize detailing and drafting work, the *Standard Drawings* are used extensively for those design features which may be common to many projects. Each designer is provided with a bound set of currently available *Roadway Design Standard Drawings.* Applicable *Standard Drawings* should be selected during the design process and identified in the plan index so that they may be printed and bound with each set of contract plans.

Table 1-5A
UNIT ROUNDING OF PRELIMINARY QUANTITY ESTIMATES

Measurement	Unit	Description	Nearest Unit	
			Estimated Quantity Sheets (Recap)	*Summary of Quantities
bale	bale	all items	1 bale	1 bale
cubic yard	yd^3	all items except: filter material, timber, structural excavation and masonry; concrete	1 yd^3 0.1 yd^3 0.01 yd^3	1 cubic yard 1 cubic yard 1 cubic yard
each	unit	all items	1 unit	1 unit
acre	acre	all items	0.1 acre	1 acre
pound	lb	all items	1 lb	1 pound
mile	mi	all items	0.001 mi	1 mile
gallon	gal	all items	1 gal	1 gallon
1000 gallons	1000 gal	all items	0.1-1000 gal	1-1000 gallons
linear foot	LF	all items except: pipe culvert guardrail	1 LF 4′ increment 12.5′ increment	1 linear foot 1 linear foot 1 linear foot
ton	ton	all items	1 ton	1 ton
square foot	ft^2	all items	0.1 ft^2	1 square foot
square yard	yd^2	all items	1 yd^2	1 square yard
station	sta	all items	1 sta	1 station
station yard	sta-yd	all items	1 sta-yd	1 station yard
thousand	1000	all items	0.1-1000	1 thousand

**Note: When quantities are smaller than one unit, round quantity to one unit.*

Chapter 2

Basic Design Controls

MISSISSIPPI DESIGN MANUAL

Chapter 2

BASIC DESIGN CONTROLS

Section		Page
2-1.0	HIGHWAY SYSTEMS	2-1
2-1.01	Functional Classification	2-1
2-1.01.01	Arterials	2-1
2-1.01.02	Collectors	2-2
2-1.01.03	Local Roads and Streets	2-2
2-1.02	Federal-Aid System	2-2
2-1.02.01	National Highway System	2-2
2-1.02.02	Surface Transportation Program	2-3
2-1.02.03	Highway Bridge Replacement and Rehabilitation Program	2-3
2-1.03	Jurisdictional Systems	2-3
2-1.03.01	State Highway System	2-3
2-1.03.02	County Road System	2-4
2-1.03.03	Municipal System	2-4
2-2.0	SPEED	2-4
2-2.01	Design Speed	2-4
2-2.02	Running Speed	2-5
2-2.03	85th Percentile Speed	2-5
2-2.04	Posted Speed Limit	2-5
2-3.0	HIGHWAY CAPACITY	2-6
2-3.01	Definitions	2-6
2-3.02	Selection of Design Year and DHV	2-7
2-3.03	Impacts on Geometric Design	2-7
2-3.04	Capacity Analyses	2-8
2-4.0	CONTROL OF ACCESS	2-8
2-5.0	PROJECT SCOPE OF WORK	2-9
2-5.01	New Construction	2-9
2-5.02	Reconstruction	2-9
2-5.03	3R Projects (Non-Freeways)	2-10
2-5.04	3R Projects (Freeways)	2-11
2-5.05	Spot Improvements (Non-Freeways)	2-11
2-6.0	OVERALL BASIS FOR DESIGN CRITERIA	2-11
2-6.01	Based on Functional Classification	2-11
2-6.02	Based on Selected Design Speed	2-12
2-6.03	Cross Section Elements	2-13
2-7.0	GEOMETRIC DESIGN CRITERIA	2-15
2-8.0	EXCEPTIONS TO GEOMETRIC DESIGN CRITERIA	2-23
2-8.01	Controlling Design Criteria	2-23
2-8.02	Procedures for Exceptions	2-23
2-8.02.01	Department Procedures	2-23
2-8.02.02	Additional Procedures for Exceptions Requiring Federal Action	2-24
2-9.0	QUALIFYING WORDS	2-24
2-10.0	REFERENCES	2-26

Chapter 2

BASIC DESIGN CONTROLS

Several overall highway design controls provide the framework which establish the geometric characteristics of a proposed highway facility. These controls will have a significant impact on the operational efficiency, safety and cost effectiveness of the highway investment. This chapter addresses the following basic design controls:

1. highway systems,
2. highway speed,
3. highway capacity,
4. control of access, and
5. project scope of work.

This chapter also provides the geometric design criteria applicable to rural highways on the State highway system and to urban freeways. Design criteria for urban streets involves special considerations. These are discussed in detail in Chapter 14. Design criteria for State Aid roads are discussed in detail in Chapter 13.

The American Association of State Highway and Transportation Officials (AASHTO) is the recognized authority on highway design policies and criteria. Since 1938, AASHTO has been developing and publishing design policies for use by highway agencies, and it continues to update policies and criteria to reflect new findings and the current state of knowledge. The AASHTO publication *A Policy on Geometric Design of Highways and Streets* (the Green Book) provides the principal source of information and design criteria contained in this *Manual*. The AASHTO criteria have been tailored to be consistent with the prevailing conditions within Mississippi.

2-1.0 HIGHWAY SYSTEMS

The Mississippi highway network has been divided into several classification systems for applying design criteria and for determining which sources of funds can be used. The Planning Division maintains and updates the maps for the functional classification system, Federal-aid system and the various jurisdictional systems.

2-1.01 Functional Classification

The Department's highway design criteria are based on the functional classification concept. In this system, highways are grouped by the character of service they provide. The two major considerations in classifying the public highway network are access to property and travel mobility. Each highway or street will provide varying levels of these two functions. In the functional classification scheme, the overall objective is that the highway system, when viewed in its entirety, will yield an optimum balance between its access and mobility purposes.

2-1.01.01 Arterials

Ideally, arterial highways are characterized by a limited access to abutting properties and a capacity to quickly move relatively large volumes of traffic. In rural areas arterials provide connections between the major urban areas and provide a level of service suitable for Statewide or interstate travel. In urban areas the arterial system serves the major centers of activity within the urban area, carries the highest traffic volumes and longest trip movements, and serves both major intracity and through

trips. The rural and urban arterial systems are connected to provide continuous through movements at approximately the same level of service. For highway design, arterial highways are divided into the following categories:

1. Freeways. The freeway is the highest level of arterial. These facilities are characterized by full control of access, high design speeds and a high level of driver comfort and safety. Freeways are considered a special type of highway within the functional classification system, and separate design criteria have been developed for these facilities.

2. Principal and Minor Arterials. In both rural and urban areas, the principal arterials provide high traffic volumes and the greatest trip length. Many of these are divided facilities which may have partial control of access. Minor arterials will provide a mix of interstate and intercounty travel service in rural areas and will provide intracommunity connections in urban areas. Minor arterials, as compared to principal arterials, provide relatively lower travel speeds, trip lengths and traffic volumes, but they provide more access to property than the principal arterial system.

2-1.01.02 Collectors

Collector routes are characterized by an approximately even distribution of their access and mobility functions. Traffic volumes and speeds will typically be somewhat lower than those of arterials. In rural areas collectors serve intracounty travel needs and provide connections to the arterial system. In urban areas collectors act as intermediate links between the arterial system and points of origin and destination.

2-1.01.03 Local Roads and Streets

All public roads and streets not classified as arterials or collectors are classified as a local facility. Local roads and streets are characterized by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility. Speeds and volumes are usually low and trip lengths short.

2-1.02 Federal-Aid System

The Federal-aid system consists of those routes within Mississippi which are eligible for the categorical Federal highway funds. The Department, working with the local governments and in cooperation with FHWA, has designated the eligible routes. United States Code, Title 23, describes the applicable Federal criteria for establishing the Federal-aid system.

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, as amended by the National Highway System Act of 1995 and the Transportation Efficiency Act for the 21st Century, implemented a major realignment of the Federal-aid system. Previously, the system had been divided into Interstate, primary, secondary and urban Federal-aid systems, which were based on the relative importance of the highway route and its anticipated functional classification 5 to 10 years in the future. Separate categories of Federal funds were available for eligible Federal-aid projects on each system. The following sections briefly describe the Federal-aid system created by ISTEA.

2-1.02.01 National Highway System

The National Highway System (NHS) is a system of those highways determined to have the greatest national importance to transportation, commerce and defense in the United States. It consists of the Interstate highway system, logical additions to the Interstate system, selected other principal arterials, and other facilities which meet the requirements of one of the subsystems within the NHS. The NHS represents approximately 4% - 5% of the total public road mileage in the United States. Specifically, the NHS includes the following subsystems (note that a specific highway route may be on more than one subsystem):

1. Interstate. The current Interstate system of highways retains its separate identity within the NHS. There are also provisions to add miles to the existing Interstate subsystem.

2. Other Principal Arterials. These are highways in rural and urban areas that provide access between an arterial and a major port, airport, public transportation facility or other intermodal transportation facility.

3. Strategic Highway Network. This is a network of highways that are important to the United States' strategic defense policy and which provide defense access, continuity and emergency capabilities for defense purposes.

4. Major Strategic Highway Network Connectors. These are highways that provide access between major military installations and highways which are part of the Strategic Highway Network.

To properly manage the NHS, ISTEA initially mandated that each State highway agency develop and implement several management systems for those facilities on the NHS. These include management systems for pavements, bridges, traffic congestion, safety, public transportation facilities/equipment and intermodal transportation facilities/systems. However, the NHS Act of 1995 has relaxed the requirements for these management systems.

2-1.02.02 Surface Transportation Program

The Surface Transportation Program (STP) is a program which provides Federal-aid funds for any public road not functionally classified as a minor rural collector or a local road or street. The basic objective of the STP is to provide Federal funds for improvements to facilities not considered to have significant national importance with a minimum of Federal requirements for funding eligibility. The STP replaced a portion of the former Federal-aid primary system and replaced all of the former Federal-aid secondary and urban systems, and it includes some collector routes which were not previously on any Federal-aid system. Collectively, these are called Federal-aid Roads. In addition, bridge projects using STP funds are not restricted to Federal-aid roads but may be used on any public road or street. Transit capital projects are also eligible under the STP program.

2-1.02.03 Highway Bridge Replacement and Rehabilitation Program

Because of the nationwide emphasis on bridges, the Highway Bridge Replacement and Rehabilitation Program (HBRRP) has retained its separate identity within the Federal-aid program. HBRRP funds are eligible for work on any bridge on any public road or street regardless of its functional classification.

2-1.03 Jurisdictional Systems

The State of Mississippi contains approximately 72,000 miles of public roads. The network has been classified into several systems based on the responsible organization for highway and street improvement, for maintenance and for traffic enforcement.

2-1.03.01 State Highway System

The State highway system consists of all highways under the jurisdiction of the Mississippi Department of Transportation. The system equals approximately 15 percent, or 10,500 miles of all public highways in Mississippi. In general, these routes are the most important highways in the State, carry the greatest traffic volumes and operate at the highest speeds. Six State highway districts conduct maintenance work on the State system.

2-1.03.02 County Road System

Mississippi has 82 counties. The county governments are responsible for all rural roads within their boundaries which are not on the State highway system. There are 53,000 (±) miles of county-maintained roads in Mississippi. Of the county-maintained roads, approximately 50 percent are paved.

The Office of State Aid Road Construction is responsible for administering both State and Federal-aid funds, which are available for highway improvements on part of the county road system. The Office has adopted specific design criteria for these projects. Chapter 13 discusses in detail the State Aid program and the design criteria used for these projects.

2-1.03.03 Municipal System

The municipal system consists of most urban roads and local city streets within the corporate limits. The extension of these routes outside the corporate limits, but still within the urban area, are generally the responsibility of the county.

2-2.0 SPEED

2-2.01 Design Speed

For each proposed highway project, a design speed is selected which establishes basic criteria for certain design elements. These include horizontal and vertical curvature, superelevation and sight distances. Design speed, perhaps more so than any other design control, will have a major impact on geometric design. The designer should evaluate the following information when determining the project design speed:

1. Project Scope of Work. Section 2-5.0 identifies the various project scopes of work. For new construction and reconstruction, the design speeds are determined from the tables of geometric design criteria in Section 2-7.0 for State highways; Section 13-3.0 for State Aid projects; and Section 14-2.0 for urban street projects. These criteria should be supplemented by the discussion on design speed in this section.

   The design speeds for 3R projects (freeways and non-freeways) and for spot improvements are determined by the criteria in Chapter 11.

2. Range. Design speeds typically range between 30 mph and 70 mph depending upon urban/rural location and functional classification. For design applications, the selected design speed is typically in a 10-mph increment up to 50 mph, although 5-mph increments are acceptable (i.e., 35 mph and 45 mph). Depending upon the project application, 5-mph increments are used for design speeds from 50 mph to 70 mph.

3. Posted/Regulatory Speed Limit. For all projects, the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility. This requirement recognizes the relationship between likely operating speeds and highway design. It also recognizes that posted speed limit creates a driver expectation of safe operating speed. Section 2-2.04 discusses the Department's policy on determining the posted speed limit.

4. Balance. The selected design speed should be a reasonable balance between topography, urban and rural character, and the functional class of the highway. A highway in level terrain may justify a higher design speed than one in rolling terrain, and a highway in a rural area may justify a higher design speed than one in an urban area.

5. Driver Expectancy. The element of driver expectancy should be considered when selecting the design speed. This is reflected in Comment #3 on posted/regulatory speed limit. Another application of driver expectancy is the design of horizontal curves at the end of long tangent sections. The overall design speed of a highway segment may be less than the top speeds on tangents within that segment. This suggests that, to accommodate driver expectancy, horizontal curves at the end of tangents should be flatter than the

maximum degree of curvature allowed by the design speed. This design practice will provide a transition for the driver back to the overall design speed for the highway segment.

6. Traffic Volumes. Traffic volumes may impact the selection of design speed. A highway carrying a large volume of traffic may justify a higher design speed than a less important facility in similar topography. However, a low design speed should not be automatically assumed for a low traffic volume road where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the importance of the highway but to the physical limitations and traffic using the facility.

Although the selected design speed establishes the criteria for highway alignment, the designer should always attempt to provide flatter horizontal curves and longer sight distances if practical.

2-2.02 Running Speed

Running speed is the average speed of a vehicle over a specified section of highway. It is equal to the distance traveled divided by the running time (the time the vehicle is in motion). The low-volume average running speed (ARS) is often used in design, which is the distance summation for all vehicles divided by the time summation. The relationship between design speed and ARS is provided in Table 2-2A.

The ARS has an important application in design, and it is reflected in several geometric design values including lengths of acceleration and deceleration lanes and minimum stopping sight distance values.

Table 2-2A
DESIGN SPEED VS. AVERAGE RUNNING SPEED

Design Speed (mph)	Average Running Speed (mph)
70	58
65	55
60	52
55	48
50	44
40	36
30	28

2-2.03 85th Percentile Speed

Many geometric design values are targeted to accommodate between 80 and 90 percent of all highway users. The most common application of the 85th-percentile value is its use as one of the factors for determining the posted, legal speed limit along a highway section (see Section 2-2.04). The Traffic Engineering Division is responsible for conducting field studies to determine the 85th percentile speed.

2-2.04 Posted Speed Limit

As discussed in Section 2-2.01, the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility. The Traffic Engineering Division is responsible for determining the posted speed on all State highways. This is based on an engineering and traffic investigation conducted after each project is completed. The following factors are considered in the evaluation:

1. road surface characteristics, shoulder condition, grade, alignment and sight distance;

2. the 85th-percentile speed and pace speed;

3. roadside development and cultural/roadside friction;

4. safe speed for curves or hazardous locations within the zone;

5. parking practices and pedestrian activity;

6. reported accident experience for a recent 12-month period;

7. the design speed used for the project design; and

8. the functional classification and type of area.

The selection of a posted speed limit should be based on all of these factors, although the 85th-percentile speed will serve as the maximum speed limit. The Traffic Engineering Division does not have the authority to conduct traffic investigations on non-State highways. This is the responsibility of the local government.

2-3.0 HIGHWAY CAPACITY

Highway capacity and geometric design are interrelated. Therefore, the designer should be familiar with the basic terminology in highway capacity and should be aware of the interrelationship between the two. The following sections discuss these factors.

2-3.01 Definitions

1. Capacity. The maximum number of vehicles which can reasonably be expected to traverse a point or uniform section of a roadway during a given time period under prevailing roadway, traffic and control conditions. The time period most often used for analysis is 15 minutes.

2. Level of Service. A qualitative concept which has been developed to characterize acceptable degrees of congestion. In the *Highway Capacity Manual*, the qualitative descriptions of each level of service (A to E) have been converted into quantitative measures for the capacity analysis for each highway element (freeway mainline, signalized intersection, etc.). The tables of geometric design criteria in the *Mississippi Design Manual* present guidelines for selecting the level of service for highway design. These apply to all highway elements (mainline, intersections, weaving areas, etc.)

3. Average Annual Daily Traffic (AADT). The total yearly volume in both directions of travel divided by the number of days in the year.

4. Average Daily Traffic (ADT). The calculation of average traffic volumes in both directions of travel in a time period greater than one day and less than one year and divided by the number of days in that time period. Although incorrect, ADT is often used interchangeably with AADT.

5. Hourly Volume. The total number of vehicles that pass over a given point or section of a lane or roadway during a hour.

6. Peak Rate of Flow. The highest equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval less than one hour, usually 15 minutes.

7. Peak-Hour Factor (PHF). A ratio of the total hourly volume to the maximum 15-minute rate of flow within the hour.

8. Design Hourly Volume (DHV). The 1-hr volume in both directions of travel in the design year selected for determining the highway design. Section 2-3.02 discusses the Department's policy for selecting the DHV for highway design. The 30th highest hourly volume is normally used for design.

9. Directional Design Hourly Volume (DDHV). The 1-hr volume in one direction of travel in the selected design year.

10. Design Service Volume or Flow Rate. The maximum hourly vehicular volume which can pass through a highway element at the selected level of service. The basic intent of a highway capacity analysis is to ensure that the DHV does not exceed the calculated design service volume of the highway element when considering the prevailing roadway, traffic and control conditions.

11. Density. The number of vehicles occupying a given length of lane, averaged over time. It is usually expressed as vehicles per mile (vpm).

12. Delay. A critical performance measure on interrupted flow facilities, especially at signalized intersections. For this element, average stopped-time delay is measured, which is expressed in seconds per vehicle.

13. Directional Distribution (D). The division, by percent, of the traffic in each direction of travel during the design hour.

14. Traffic Composition (T). A factor which reflects the percentage of heavy vehicles (trucks, buses and recreational vehicles) in the traffic stream during the DHV. The poorer operating capabilities and larger size of heavy vehicles must be reflected in the capacity analysis.

2-3.02 Selection of Design Year and DHV

A highway should be designed to accommodate the traffic volume that might occur within the life of the facility under reasonable maintenance. This involves projecting the traffic conditions for a selected future year. For new construction and reconstruction, traffic volume projections are usually made for the 15- to 25-year range, with 20 years from the expected construction completion date the typical selection. For 3R projects, the designer should normally provide a highway facility which will accommodate the DHV for 10 years beyond the construction completion date at the selected level of service.

The Planning Division maintains current records on traffic data for all roads and develops projections of expected future traffic volumes including the DHV, AADT, directional distribution and traffic composition. The design traffic data should be included in the contract plans.

2-3.03 Impacts on Geometric Design

The following briefly discusses the interrelationship between highway capacity and geometric design. These factors are discussed in detail in the *Highway Capacity Manual*:

1. Number of Lanes. This is the principal design element affected. The DHV combined with the selected level of service will determine the number of lanes on the highway facility. However, in some cases, a multilane facility may be constructed when not justified by a capacity analysis alone. This may be for safety, economic development or other reasons which indicate that the economic investment is justified. The decision on the number of lanes is normally made during the planning process before projects are assigned to individual squad leaders.

2. Traffic Composition. As the percentage of heavy vehicles increases, the highway capacity decreases. The standard procedure is to convert the volume of trucks and recreational vehicles to a passenger car equivalent.

3. Lane and Shoulder Width. As lane and shoulder widths decrease, the highway capacity decreases. Narrow lanes force drivers closer to vehicles in opposing lanes. Restricted shoulders force drivers to "shy" away from roadside obstructions. Adjustment factors are used to calculate the influence of narrow pavement widths.

4. Alignment. Horizontal and vertical alignment may affect highway capacity. The frequency and sharpness of curves and grades can be significant factors. For example, this will determine the frequency of adequate passing sight

distances. The capacity of a highway is reduced when there is limited opportunity for overtaking and passing slow-moving vehicles. Adjustment factors based on the "average travel speed" are used to calculate the influence of grades on passing opportunities.

5. Auxiliary Lanes. The presence or absence of auxiliary lanes may affect the capacity of the highway. These include truck-climbing lanes, turn lanes at intersections, weaving lanes at interchanges and continuous two-way left-turn lanes. Universal adjustment factors are not applicable, and individual analyses are necessary to determine their impact on capacity.

6. At-Grade Intersections. At-grade intersections, including driveways, may have a major impact on the capacity of the highway. In many cases, their influence on service volumes is so great that they govern the capacity of the entire highway segment. Therefore, intersections are evaluated individually.

7. Urban Area Development. Roadside development and adjacent land use along an urban arterial may reduce the capacity of the highway. The factors associated with development, including parking, driveways and pedestrians, will influence the traffic flow.

8. Freeway Interchanges. Weaving sections and ramp terminals at interchanges may impact the capacity of the freeway mainline. By definition these are the only allowable points of access onto a freeway. The design details of interchanges to provide sufficient capacity should be considered separately.

2-3.04 Capacity Analyses

The capacity analysis is the responsibility of the highway designer. Therefore, the designer should be familiar with the detailed methodologies presented in the *Highway Capacity Manual*. Where unusual or complex capacity situations exist, the designer may request assistance from the Traffic Engineering Division.

The highway mainline, intersection or interchange should be designed to accommodate the design hourly volume (DHV) at the selected level of service. This involves adjusting the various highway factors which affect capacity until a design is found that will accommodate the DHV. In capacity analyses, the designer should calculate the design service volume of the facility. Capacity assumes a level of service E; design service volume is the maximum volume of traffic that a projected highway of designed dimensions is able to serve without the degree of congestion falling below a preselected level. The designer should design for the level of service as specified in the appropriate Geometric Design Criteria table of this *Manual*.

2-4.0 CONTROL OF ACCESS

Access control is defined as the condition where the public authority fully or partially controls the right of abutting owners to have access to and from the public highway. The functional classification of a highway (see Section 2-1.0) is partially determined by the degree of access it allows. Access control may be exercised by statute, zoning, right-of-way purchases, driveway controls, turning and parking regulations or geometric design (e.g., grade separations and frontage roads). Section 6-10 discusses access control at interchanges. Sections 6-9.0 (rural driveways) and 14-2.0 (urban driveways) discuss procedures and criteria which may be used for access control. Additional information on control of access is presented in S.O.P. No. ADM-23-01-00-000 "Types of Highways."

The following provides definitions for the basic types of access control:

1. Type 1— Full Control (Freeway). Full control of access is achieved by providing access only at interchanges with selected public roads. No at-grade crossings or private driveway connections are allowed.

2. Type 2 — Partial Control. Partial control of access is an intermediate level between full control and regulatory restriction. Priority is given to through traffic, but a few at-grade intersections and private driveway connections may be allowed. The two types of partial control of access are:

a. Type 2A. Access to through traffic lanes permitted only at designated exits and entrances. Frontage roads may be provided for abutting property owners.

b. Type 2B. Access to through traffic lanes permitted only at designated exits and entrances.

3. Type 3 — Control by Regulation. All highways warrant some degree of access control. If access points are properly spaced and designed, the adverse effects on highway capacity and safety will be minimized. These points should be located where they can best suit the traffic and land-use characteristics of the highway under design. Their design should enable vehicles to enter and exit safely with a minimum of interference to through traffic. In Type 3, access to traffic lanes is permitted directly from the abutting property. Special permits are required for new access points after construction. The following Department policies will apply:

a. SOP No. MND-03-00-00-000 "Processing of Permit Applications."

b. SOP No. MND-03-03-01-000 "Driveway Connections."

c. SOP No. MND-03-04-01-000 "Construction and Maintenance of Driveway, County Road and Municipal Street Connections to State Highways."

d. SOP No. MND-04-03-01-000 "Driveway and Street Connections, Median Openings, Frontage Roads."

Designers will be provided the type of access control which applies on each project.

2-5.0 PROJECT SCOPE OF WORK

The scope of work for the proposed highway project is a major control in highway design. The Location/Design Committee is responsible for recommending the project scope of work. A general description of the Committee and its procedures is presented in S.O.P. No. ENV-03-01-00-000.

The project scope of work will reflect the basic intent of the highway project and will determine the overall level of improvement. This decision will determine which criteria in the *Design Manual* apply to the project. The following descriptions are intended to provide general definitions for the project scope of work.

2-5.01 New Construction

The Department's geometric design criteria for new construction are presented in Tables 2-7B through 2-7D (rural State highways and urban freeways), Tables 13-3A and 13-3B (State Aid roads), and Tables 14-2B through 14-2D (urban streets). New construction is defined as horizontal and vertical alignment on new location. In addition, any intersection which falls within the project limits of a new highway mainline or is relocated to a new point of intersection is considered new construction. Chapter 6 presents the Department's design criteria for the new construction of at-grade intersections.

2-5.02 Reconstruction

Due to the significant level of work for reconstruction, the design of the project should be determined by the criteria for new construction. Reconstruction of an existing highway mainline will typically include the addition of travel lanes and/or reconstruction of the existing horizontal and vertical alignment, but essentially within the existing highway corridor. These projects will usually require right-of-way acquisitions. The primary reason to perform reconstruction of an existing highway is often because the existing facility cannot accommodate its current or future traffic

demands or because the existing alignment is significantly deficient. In addition, any intersection which falls within the limits of a reconstruction project will be reconstructed as needed (Chapter 6). This could include the:

1. addition of through and auxiliary lanes for all approaches;
2. relocation and flattening of turning radii;
3. addition of turning roadways;
4. flattening the approach and intersection gradients;
5. realigning the angle of intersection;
6. rechannelizing the intersection; and/or
7. significant right-of-way acquisition.

2-5.03 3R Projects (Non-Freeways)

Chapter 11 presents the Department's criteria for 3R projects (resurfacing, restoration and rehabilitation) on non-freeway facilities. 3R projects are primarily intended to extend the service life of the existing facility and to enhance highway safety. In addition, 3R projects should make cost-effective improvements to the existing geometrics, where practical. 3R work on the mainline or at an intersection is work essentially within the existing alignment. Right-of-way acquisition is usually not involved, although small takings are sometimes justified. Typical improvements for 3R projects include:

1. pavement resurfacing,
2. lane and shoulder widening,
3. flattening an occasional horizontal or vertical curve,
4. widening the roadside clear zone,
5. flattening side slopes,
6. converting an existing median to a continuous two-way left-turn (CTWLT) lane,
7. adding a CTWLT lane,
8. converting an uncurbed urban street into a curbed street,
9. revising the location, spacing or design of existing driveways along the mainline,
10. adding or removing parking lanes,
11. adding sidewalks,
12. relocating utility poles,
13. upgrading guardrail to meet current criteria,
14. geometric and/or safety improvements to existing bridges within the project limits,
15. drainage improvements,
16. widening the approach width at an intersection,
17. adding or lengthening an auxiliary lane,
18. improving the corner sight distance,
19. flattening an existing turning radii,
20. minor realignment of an intersection angle,

21. widening an existing turning roadway, and/or

22. minor intersection rechannelization.

2-5.04 3R Projects (Freeways)

Chapter 11 presents the Department's criteria for 3R projects (resurfacing, restoration and rehabilitation) on existing freeways. 3R projects are primarily intended to extend the service life of the existing facility and enhance highway safety. In addition, these projects should make cost-effective improvements to the existing geometrics, where practical. Typical improvements include:

1. pavement resurfacing or reconstruction (including shoulders),

2. widening the roadside clear zone,

3. flattening side slopes,

4. improvements to interchange gore areas,

5. regrading median ditch plugs,

6. upgrading guardrail to meet current criteria,

7. geometric and/or safety improvements to existing bridges within the project limits,

8. drainage improvements,

9. addition of auxiliary lanes,

10. lengthening existing acceleration or deceleration lanes, and/or

11. realigning or widening an existing ramp.

Where extensive reconstruction is deemed necessary (realignment, significant right-of-way acquisition, adding lanes, etc.), new construction criteria for freeways should be applied.

2-5.05 Spot Improvements (Non-Freeways)

Spot improvements are intended to correct an identified deficiency at an isolated location. The deficiency may be related to structural, geometric, safety, drainage or traffic control problems. These projects are not intended to provide a general upgrading of the highway, as are projects categorized as new construction, reconstruction or 3R. Two types of spot improvements are:

1. safety improvements funded from the Federal-aid safety set-aside funds; and

2. bridge improvement projects funded by the Highway Bridge Replacement and Rehabilitation Program.

Chapter 11 discusses the Department's criteria for the geometric design of spot improvement projects.

2-6.0 OVERALL BASIS FOR DESIGN CRITERIA

2-6.01 Based on Functional Classification

The functional classification of the highway is the basic control for the Department's geometric design criteria. The classification system, discussed in Section 2-1.0, establishes the general type and character of the highway. For example, a freeway indicates relatively high design speeds, control of access, surfaced shoulders and wide clear zones. Collector roads usually have lower design speeds, access control by regulation and graded shoulders.

2-6.02 Based on Selected Design Speed

The following geometric design criteria are directly related to the design speed:

1. Horizontal Curvature and Superelevation. In the design of horizontal alignment, it is necessary to establish the proper relationship between design speed, the degree of curvature and superelevation. For a given design speed the maximum superelevation rate and side-friction factor determine the maximum degree of curvature. The maximum superelevation rate depends on several factors including climate, terrain and urban/rural area. The Department has adopted e_{max} = 10.0% for the design of rural highways and urban freeways. For urban streets and urban highways with a design speed of 50 mph or less, e_{max} = 6.0%. The maximum degree of curvature for various design speeds and e_{max} = 10.0% are presented in Table 2-7A. Chapter 3 presents the details for horizontal curvature and superelevation.

2. Vertical Curvature. The required sight distance, which is based on design speed, is the major control in the design of vertical curves. The line of sight should not be obstructed by the crest of a vertical curve. Sag vertical curves are designed to provide headlight sight distance equal to the stopping sight distance. Chapter 4 discusses the length of vertical curves based on the sight distance requirements and design speed.

3. Stopping Sight Distance. Sight distance is the length of highway ahead visible to the driver. The minimum distance available should be sufficient to enable a vehicle traveling at the average running speed to stop before reaching an object in its path. Stopping sight distance is the sum of two distances — the brake reaction distance and the braking distance. *A Policy on Geometric Design of Highways and Streets* discusses the concepts and procedures in detail.

   Sight distance may be restricted by vertical alignment, horizontal alignment, roadside obstructions or any combination of these elements. The stopping sight distance values are provided in Table 2-7A. The use of stopping sight distance in the design of horizontal and vertical curves is discussed in Chapters 3 and 4.

   The designer should always try to provide greater sight distance than the values in Table 2-7A. Stopping sight distance is measured from the driver's eye 3.5 ft above the pavement to a 6-in. height of object.

4. Decision Sight Distance. Drivers frequently are required to make decisions where information is difficult to perceive or where unexpected maneuvers are required. These are areas of concentrated demand where the roadway elements, traffic volume and traffic control devices all compete for the driver's attention. Examples of these locations where vehicular control errors might occur include interchanges, complex intersections and changes in cross sections (e.g., lane drops). In these cases, decision sight distance may be warranted to allow the driver more time to evaluate the roadway condition and to select a course of action.

   Table 2-6A provides values for decision sight distance. The distance is measured from a 3.5-ft height of eye to a 6-in. height of object.

Table 2-6A
DECISION SIGHT DISTANCE

Design Speed (mph)	Decision Sight Distance for Avoidance Maneuver (ft)				
	A	B	C	D	E
30	220	500	450	500	625
40	345	725	600	725	825
50	500	975	750	900	1025
60	680	1300	1000	1150	1275
70	900	1525	1100	1300	1450

Note: Avoidance Maneuvers are:

A: Stop on rural road
B: Stop on urban road
C: Speed/path/direction change on rural road
D: Speed/path/direction change on suburban road
E: Speed/path/direction change on urban road

5. Passing Sight Distance. Passing sight distance considerations are limited to two-lane, two-way highways on which vehicles frequently overtake slower moving vehicles and the passing must be accomplished on a lane used by opposing traffic. Passing sight distance values provided in Table 2-7A are based on the distance needed to safely complete a normal passing maneuver. The designer, where practical, should provide passing sight distance over a high proportion of the highway length. This may involve adjustments to the horizontal and vertical alignment, or additional lanes may be necessary in restricted areas. Passing sight distance is measured from a 3.5-ft height of eye to a 4.25-ft height of object.

   The passing sight distance values presented for design should not be confused with distances used for no-passing zone pavement markings. Values provided in the MUTCD are substantially less than AASHTO passing sight distances and are derived for traffic operating control needs which are based on assumptions different from those for geometric design. The Department's criteria for no-passing zones for pavement markings is presented in Section 9-7.0.

6. Grades. Maximum grades for each design speed are based on the functional classification of the highway and the terrain conditions. The maximum grades provided in Tables 2-7B through 2-7D should be used only where absolutely necessary.

2-6.03 Cross Section Elements

Cross section elements are usually determined by the functional class and capacity requirements. Figure 2-7A illustrates a typical highway cross section. The geometric design criteria for the following cross section elements are provided in Tables 2-7B through 2-7D:

1. Lane Width. The portion of the roadway for the movement of vehicles exclusive of shoulders is called the traveled way. For safety, efficiency and ease of operation, lane widths for the traveled way in the range of 10 ft to 12 ft are used. The use of 12-ft lanes is predominant on high-speed highways.

2. Auxiliary Lane Width. Auxiliary lanes are the portion of the roadway adjoining the traveled way intended for purposes supplementary to the through traffic movement. Auxiliary lanes include truck-climbing lanes and turning lanes at intersections. Desirably, these lanes should be as wide as the through lane.

3. Usable Shoulder Width. The shoulder is that portion of the cross section contiguous to the traveled way. It is available for stopped vehicles, emergencies and lateral support of the pavement structure. Shoulders also provide a recovery area for vehicles which inadvertently leave the through lanes. The usable shoulder width is measured from the edge of the traveled way to the intersection of the shoulder slope and the foreslope (or safety slope) planes.

4. Surfaced Shoulder Width. Surfaced shoulders provide a better all-weather riding surface and load support than afforded by native soils or stabilized materials. Therefore, they should be provided on all freeway facilities and sometimes on other multilane arterials.

5. Median Width. The median on a divided highway separates opposing traffic, reduces headlight glare and assists in the safe operation of crossing vehicles at intersections and crossovers. The median width is measured between the inside edges of the two traveled ways of the opposing roadways. On freeways, the median width is typically 64 ft. On rural multilane highways, the median width is typically 101 ft with, perhaps, an independent grade design of each roadway. In urban areas with right-of-way restrictions, it may not be practical to provide a median width of 64 ft on freeways. With a concrete median barrier (CMB), the minimum median width is 22 ft assuming a 10-ft inside shoulder. Desirably, a 12-ft inside shoulder will be used, and the median width will be 26 ft.

6. Right-of-Way Width. The right-of-way width is the sum of all cross section elements, including traveled way width, median width, side slopes and border widths. The right-of-way border width is defined as the distance between the right-of-way lines inward to the toe of fill slopes or top of cut slopes. See Figure 2-7A. Each situation must be evaluated individually considering the existing conditions and economic factors.

7. Bridge Width. The bridge width refers to the clear width measured between the gutter lines or bridge rails, whichever is less. Tables 2-7B through 2-7D provide the Department's bridge width criteria for new and reconstructed bridges. Chapter 11 discusses the Department's criteria for bridge widths within the limits of 3R projects and for bridge work which is limited to the bridge itself (e.g., bridge deck replacement).

8. Roadway Width at Bridge Ends. The Department's policy is to install guardrail protection at bridge ends on all projects. In some cases it will be necessary to widen the embankment of the bridge ends to facilitate guardrail installation. The approach roadway width at bridge ends should be at least 2 ft on each side (or 4 ft total) wider than the clear width of the bridge. The transition between normal roadway width and the roadway width at bridge ends should be according to the design of the guardrail installation. See the *Roadway Design Standard Drawings* for details.

9. Roadside Clear Zone. An adequate clearance between the edge of traveled way and roadside obstructions is an important safety design element. Vehicles leaving the roadway should have a reasonable distance to recover and return to the roadway without colliding with roadside objects. These include trees, retaining walls, drainage structures and fences. Breakaway light poles and sign posts are permitted within the lateral clearance zones. Clear zones are discussed in detail in Section 9-1.0.

10. Vertical Clearance. The vertical clearance is measured between overhead structures and the finished roadway surface. The designated clearance must be provided over the entire roadway width, including shoulders. For all State highways, the minimum vertical clearance should be 16.5 ft. This will ensure a minimum clearance of 16 ft after a 6-in. future resurfacing. The minimum vertical clearance is 17.5 ft for the highway passing under a pedestrian bridge or sign truss and, for structures over railroads, the minimum vertical clearance between the top of rail and bottom of the highway structure is 23 ft.

11. Travel Lane and Shoulder Cross Slope. The cross slope on tangent alignment is related to the type of surface. The pavement surface should be sloped sufficiently to ensure proper drainage and not affect vehicular operation. Cross slopes for travel lanes on tangent sections should be 2.0% for both concrete and bituminous pavements. All shoulder cross slopes should be 4.0% on tangent sections, except on curbed facilities where the shoulder width is less than 4 ft. In this case, the shoulder slope will match the travel lane slope.

12. Side Slopes. Roadway excavation and embankment slopes are part of the graded highway supporting or maintaining the roadway. The rate of slope depends on the terrain, the depth of excavation or embankment and the type of material encountered. Safety considerations, maintenance, type of highway facility and aesthetics further influence side slope rates.

    In cut sections, the definitions of cross section slopes are (see Figure 2-7A):

    a. Foreslope. The slope extending outward and downward from the shoulder to the ditch line.

    b. Backslope. The slope extending upward and outward from the ditch line to intersect the natural ground line.

In fill sections, the definitions of cross section slopes are (see Figure 2-7A):

a. Safety slope. The slope extending outward and downward from the shoulder point to a horizontal distance equal to the clear zone requirements (see Section 9-1.0).

b. Fill slope. The embankment slope extending outward and downward from the outer limit of the safety slope to intersect the natural ground line.

Rounding of the cross section should be provided at the ditch bottoms and tops of backslopes. All side slopes should be vegetated. Erosion control and ditch treatment are discussed in detail in Chapter 8.

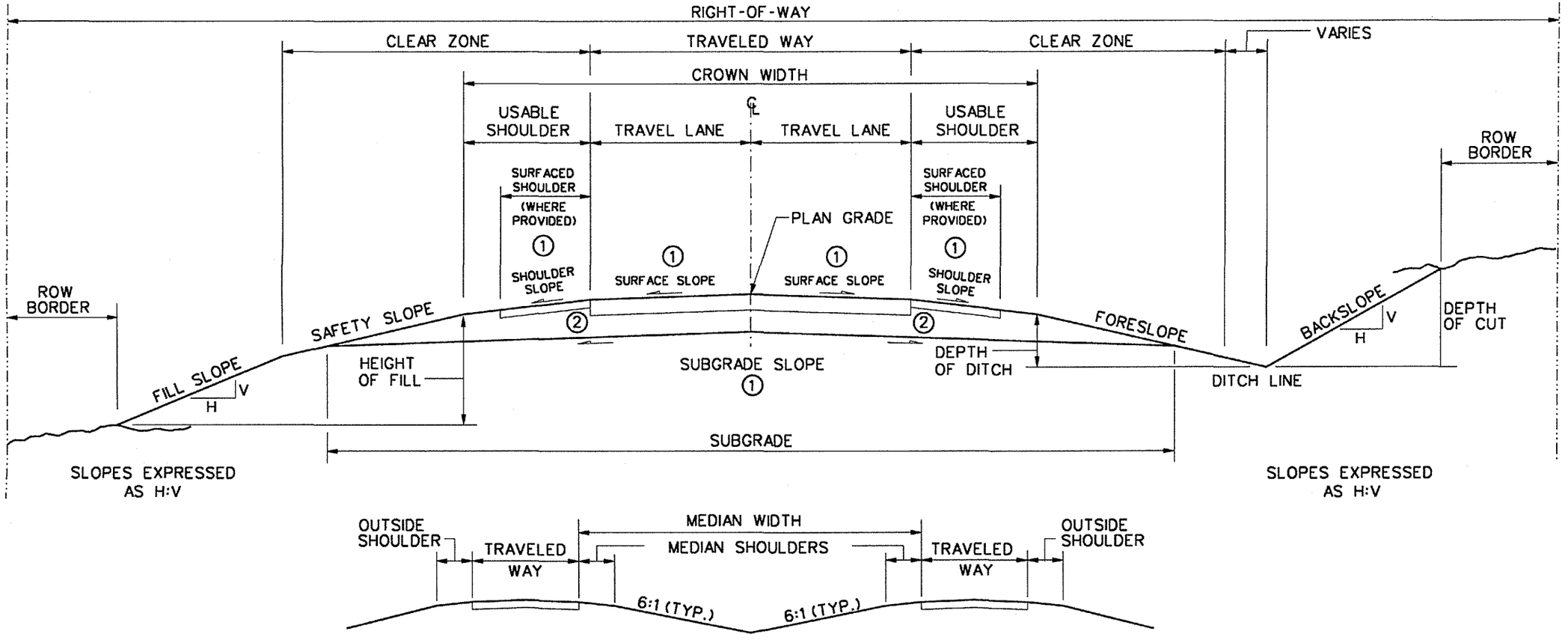
2-7.0 GEOMETRIC DESIGN CRITERIA

This section presents the Department's criteria for the design of new construction and reconstruction projects on rural State highways and urban freeways. Geometric design criteria based on design speed are provided in Table 2-7A. Figure 2-7A illustrates the typical cross section elements for both undivided and divided highways. Design criteria for cross section and alignment elements are presented in Table 2-7B (freeways), Table 2-7C (rural arterials) and Table 2-7D (rural collectors). The Department's geometric design criteria for State Aid projects is presented in Chapter 13. Chapter 14 presents the criteria for urban streets.

Table 2-7A
GEOMETRIC DESIGN CRITERIA BASED ON DESIGN SPEED
(New Construction/Reconstruction)

Design Element	Design Speed (mph)						
	30	40	50	55	60	65	70
Maximum Degree of Curvature (1)	24°45′	13°15′	8°15′	6°30′	5°15′	4°15′	3°30′
Minimum Stopping Sight Distance	185′	275′	380′	435′	505′	550′	615′
Passing Sight Distance	1100′	1500′	1800′	2000′	2100′	2300′	2500′

(1) Values based on e_{max}= 10.0%. See Table 3-4A for other combinations of design speed, degree of curvature and superelevation rates. The values in Table 2-7A apply to all rural highways (all design speeds) and urban highways (V > 50 mph). See Chapter 14 for maximum degree of curvature on urban facilities where V ≤ 50 mph.



Notes: ① Typical cross slopes:
Concrete and Bituminous = 2%
Shoulders = 4%
Subgrade = 2%-3%

② For graded shoulders, extend pavement base course 1 ft beyond the edge of the surfaced traveled way.

Note: Typical section for State-Aid roads (see Figure 13-3A).
Typical section for urban streets (see Figures 14-2H and I).

TYPICAL SECTION NOMENCLATURE
Figure 2-7A

Table 2-7B

GEOMETRIC DESIGN CRITERIA FOR FREEWAYS (New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Rural		Urban
Design Controls	Design Forecast Year			2-3.04	20 Years		20 Years
	✻Design Speed			2-2.01	70 mph		60 mph - 70 mph
	Control of Access			2-4.0	Full (Type 1)		Full (Type 1)
	Level of Service			2-3.04	B		Desirable: B Minimum: C
Cross Section Elements	✻Lane Width			2-6.03	12′		12′
	Outside Shoulder Width (1a)	✻Usable		2-6.03	12′		12′
		✻Surfaced			10′		10′
	Median Shoulder Width(1b)(1c)	✻Usable		2-6.03	8′		8′
		✻Surfaced			4′		4′
	✻Cross Slope	Travel Lane		2-6.03	2%		2%
		Shoulder			4%		4%
	Auxiliary Lanes	Lane Width		2-6.03	12′		12′
		Shoulder Width			Surfaced: 10′ Usable 12′		Surfaced: 10′ Usable 12′
	Median Width	Depressed		2-6.03	64′ Minimum		64′
		Concrete Median Barrier			N/A		4-Lane: 26′ Desirable; 22′ Minimum ≥ 6-Lane: 26′ Desirable; 22′ Minimum
	✻New and Reconstructed Bridges	Structural Capacity		2-6.03; 11-4.0	HS-20		HS-20
		Minimum Width			Traveled Way + 12′ (Outside Shoulder) + 6′ (Median Shoulder)		Traveled Way + 12′ (Outside Shoulder) + 6′ (Median Shoulder)
	✻Existing Bridges to Remain in Place	Structural Capacity		2-6.03; 11-4.0	HS-20		HS-20
		Minimum Width (2)			Traveled Way + 10′ (Outside Shoulder) + 4′ (Inside Shoulder)		Traveled Way + 10′ (Outside Shoulder) + 4′ (Inside Shoulder)
	Minimum Right-of-Way Width			2-6.03; 10-1.02	300′		(3)
	Desirable R/W Border Width (Beyond toe/top of fill/cut slope)				30′		15′
	Roadside Clear Zone	Guardrail		9-4.03; 9-2.0	Usable Shoulder Width		Usable Shoulder Width
		Obstruction (4)			30′		30′
	Slope Schedule (5)	Cut	Foreslope (within clear zone)	2-6.03	6:1		6:1
			Depth of Ditch		4′		4′
			Backslope		3:1		3:1
		Fill	Safety Slope (within clear zone)		6:1		6:1
			Fill Slope (outside clear zone)		3:1		3:1
Alignment Elements	DESIGN SPEED				60 mph	65 mph	70 mph
	✻Minimum Stopping Sight Distance			2-6.02	505′	550′	615′
	✻Maximum Degree of Curvature (e = 10%)			3-3.0	5°15′	4°15′	3°30′
	✻Superelevation Rate			3-4.01	See Table 3-4A (e_{max} = 10%)		
	✻Horizontal Sight Distance			3-5.0	(6)		
	✻Maximum Grades (7)	Level		4-3.01	3%	3%	3%
		Rolling			4%	4%	4%
	Minimum Grades			4-3.02	Desirable: 0.4% Minimum: 0.0%		
	✻Vertical Curve (K-values)			4-4.0	Crest: 192 Sag: 118	Crest: 228 Sag: 131	Crest: 285 Sag: 149
	✻Minimum Vertical Clearance (Freeway Under)	New/Reconstructed Bridges (8)		2-6.03	16.5′		
		Existing Bridges			16′		
		Sign Truss (8)			17.5′		
	Vertical Clearance (Freeway Over Railroad)			2-6.03	23′		

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR FREEWAYS
(New Construction/Reconstruction)

Footnotes to Table 2-7B

1. Shoulder Width. For 3 or more lanes in each direction, the following will apply:

   a. Outside Shoulder. Where the truck volume exceeds 250 DDHV, a 12-ft surfaced shoulder should be considered.

   b. Median Shoulder (Depressed Median). Where the two roadways are separated by a depressed median, provide a 12-ft usable shoulder and a 10-ft surfaced shoulder. Where the truck volume exceeds 250 DDHV, a 12-ft surfaced shoulder should be considered.

   c. Median Shoulder (Concrete Median Barrier). Where the two roadways are separated by a CMB, the desirable surfaced median shoulder width is 12-ft; the minimum surfaced median shoulder width is 10 ft. Where the truck traffic exceeds 250 DDHV, the 12-ft width merits stronger consideration.

2. Bridge Width. For existing bridges to remain in place which are considered major structures (i.e., lengths greater than 200 ft), the minimum clear width is the width of the traveled way plus 3.5 ft on each side.

3. Minimum Right-of-Way Width (Urban). The right-of-way limits in urban areas will be determined by build up, property values, etc.

4. Roadside Clear Zone. The recommended clear zones are for design speeds of 60 mph - 70 mph, for 6:1 fill slopes or foreslopes and for an ADT > 6000. See Section 9-2.0 for other roadside conditions. All values are measured from the edge of traveled way or auxiliary lane, if present.

5. Slopes. If high-volume change soil is present, see Section 12-2.04.

6. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

7. Maximum Grades. Grades 1 percent steeper may be used on 1-way down-grades. Grades 1 percent steeper may also be used in restricted urban areas where development precludes the use of flatter grades.

8. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

Table 2-7C
GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	2-Lane				Multilane
			Design ADT		Under 400	400 and Over			
			Design DHV			100 - 200	201 - 400	Over 400	
Design Controls	Design Forecast Year			2-3.04	20 Years				20 Years
	✻Design Speed			2-2.01	65 mph				65 mph
	Control of Access			2-4.0	Control by Regulation (Type 3)				Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)
	Level of Service			2-3.04	B				B
Cross Section Elements	✻Lane Width			2-6.03	12′				12′
	Outside Shoulder Width	✻Usable		2-6.03	6′	6′	8′	10′	10′
		Surfaced			(1)				(1)
	Median Shoulder Width	✻Usable		2-6.03	N/A				8′
		Surfaced			N/A				(1)
	✻Cross Slope	Travel Lane		2-6.03	2%				2%
		Shoulder			4%				4%
	Auxiliary Lanes	Lane Width		2-6.03	12′				12′
		Usable Shoulder Width			6′	6′	8′	10′	Same Usable/Surfaced Width Adjacent to Travel Lane
	Median Width			2-6.03	N/A				101′ Typical (2)
	✻New and Reconstructed Bridges	Structural Capacity		2-6.03; 11-4.0	HS-20				HS-20
		Minimum Width (3)			36′	36′	44′	44′	Traveled Way +10′ (outside shoulder) +6′ (median shoulder)
	✻Existing Bridges to Remain in Place	Structural Capacity		2-6.03; 11-4.0	HS-20				HS-20
		Minimum Width (3)			28′				Traveled Way + 4′ (Each Roadway)
	Minimum Right-of-Way Width			2-6.03; 10-1.02	100′	110′	120′	130′	240′
	Desirable R/W Border Width (Beyond toe/top of fill/cut slope)				10′	10′ - 15′	15′- 20′	20′	30′
	Roadside Clear Zone	Guardrail		9-4.03; 9-2.0	Usable Shoulder Width				Usable Shoulder Width
		Obstruction			See Note (4)				See Note (4)
	Slope Schedule (5)	Cut	Foreslope (within clear zone)	2-6.03	6:1				6:1
			Depth of Ditch		3′				4′
			Backslope		3:1				3:1
		Fill	Safety Slope (within clear zone)		6:1				6:1
			Fill Slope (outside clear zone)		3:1				3:1
Alignment Elements	DESIGN SPEED				65 mph				
	✻Minimum Stopping Sight Distance			2-6.02	550′				
	✻Maximum Degree of Curvature (e = 10%)			3-3.0	4°15′				
	✻Superelevation Rate			3-4.01	See Table 3-4A (e_{max} = 10%)				
	✻Horizontal Sight Distance			3-5.0	(6)				
	✻Maximum Grades (7)	Level		4-3.01	3%				
		Rolling			4%				
	Minimum Grades			4-3.02	Desirable: 0.4% Minimum: 0.0%				
	✻Vertical Curve (K-values)			4-4.0	Crest: 228 Sag: 131				
	✻Minimum Vertical Clearance (Arterial Under)	New/Reconstructed Bridges (8)		2-6.03	16.5′				
		Existing Bridges			16′				
		Sign Truss (8)			17.5′				
	Vertical Clearance (Arterial Over Railroad)			2-6.03	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (New Construction/Reconstruction)

Footnotes to Table 2-7C

1. Shoulder Surface Type (2-lane and Shoulder on Multilane). Generally, these shoulders will not be surfaced except as approved in special conditions.

2. Median Width. The 101-ft typical median width applies to new construction or to the dualization of an existing two-lane, two-way facility. Exceptions to this width may be considered for the following:

   a. areas with environmental restrictions,

   b. facilities with Type 1 access control,

   c. facilities where full-width bridges have already been constructed at a lesser width, or

   d. other areas with justification.

   For a reconstruction project on an existing divided facility, the existing median width may be retained.

3. Bridge Width. Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

4. Roadside Clear Zone. The recommended clear zones are based on design speed, side slopes and traffic volumes. See Section 9-2.0. All values are measured from the edge of traveled way or auxiliary lane, if present.

5. Slopes. If high-volume change soil is present, see Section 12-2.04.

6. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

7. Maximum Grades. Grades 1 percent steeper may be used on 1-way downgrades.

8. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

Table 2-7D

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(New Construction/Reconstruction)

	DESIGN ELEMENT		Design ADT	Manual Section	2-Lane: Under 400	2-Lane: 400 - 1500	2-Lane: 1500-2000	2-Lane: Over 2000	Multilane	
Design Controls	Design Forecast Year			2-3.04	20 Years				20 Years	
	⁎Design Speed (1)			2-2.01	55 mph			Level: 60 mph Rolling: 55 mph	65 mph	
	Control of Access			2-4.0	Control by Regulation (Type 3)				Control by Regulation (Type 3)	
	Level of Service			2-3.04	Desirable: B Minimum: C				B	
Cross Section Elements	⁎Lane Width			2-6.03	12′				12′	
	Outside Shoulder Width	⁎Usable		2-6.03	4′	5′	6′	8′	10′	
		Surfaced			N/A				(2)	
	Median Shoulder Width	⁎Usable		2-6.03	N/A				8′	
		Surfaced			N/A				(2)	
	⁎Cross Slope	Travel Lane		2-6.03	2%				2%	
		Shoulder			4%				4%	
	Auxiliary Lanes	Lane Width		2-6.03	11′				12′	
		Usable Shoulder Width			4′	5′	6′	8′	Same Usable/Surfaced Width Adjacent to Travel Lane	
	Median Width			2-6.03	N/A				101′Typical (3)	
	⁎New and Reconstructed Bridges	Structural Capacity		2-6.03; 11-4.0	HS-20				HS-20	
		Minimum Width (4)			28′	30′	32′	44′	Traveled Way +10′ (outside shoulder) +6′ (median shoulder)	
	⁎Existing Bridges to Remain in Place	Structural Capacity		2-6.03; 11-4.0	HS-20				HS-20	
		Minimum Width (4)			24′			28′	28′ (Each Roadway)	
	Minimum Right-of-Way Width			2-6.03; 10-1.02	80′	100	120′		240′	
	Desirable R/W Border Width (Beyond toe/top of fill/cut slope)				5′ - 10′	10′ - 15′	15′ - 20′		30′	
	Roadside Clear Zone	Guardrail		9-4.03; 9-2.0	Usable Shoulder Width				Usable Shoulder Width	
		Obstruction			See Note (5)				See Note (4)	
	Slope Schedule (6)	Cut: Foreslope (within clear zone)		2-6.03	V ≤ 55mph 4:1 V > 55 mph 6:1				6:1	
		Cut: Depth of Ditch			3′				4′	
		Cut: Backslope			3:1				3:1	
		Fill			Height of Fill < 5′ — 4:1 Height of Fill ≥ 5′ — 3:1		Height of Fill < 10′ — 4:1 Height of Fill ≥ 10′ — 3:1		6:1 (Within Clear Zone) 3:1 (Outside Clear Zone)	
Alignment Elements	DESIGN SPEED				55 mph		60 mph		65 mph	
	⁎Minimum Stopping Sight Distance			2-6.02	435′		505′		550′	
	⁎Maximum Degree of Curvature (e = 10%)			3-3.0	6°30′		5°15′		4°15′	
	⁎Superelevation Rate			3-4.01	See Table 3-4A (e_{max} = 10%)					
	⁎Horizontal Sight Distance			3-5.0	(7)					
	⁎Maximum Grades	Level		4-3.01	5.5%		5%		4.5%	
		Rolling			6.5%		6%		5.5%	
	Minimum Grades			4-3.02	Desirable: 0.4% Minimum: 0.0%					
	⁎Vertical Curve (K-values)			4-4.0	Crest: 143 Sag: 99		Crest: 192 Sag: 118		Crest: 228 Sag: 131	
	⁎Minimum Vertical Clearance (Collector Under)	New/Reconstructed Bridges (8)		2-6.03	16.5′					
		Existing Bridges			14.5′					
	Vertical Clearance (Collector Over Railroad)			2-6.03	23′					

⁎Controlling design criteria. *See Section 2-8.0.*

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (New Construction/Reconstruction)

Footnotes to Table 2-7D

1. Design Speed. A rural collector may pass through a relatively built-up area where the posted speed limit is below 55 mph. In these sections, a design speed as low as the posted speed limit may be used.

2. Shoulder Surface Type (Shoulder on Multilane). Generally, these shoulders will not be surfaced except as approved in special conditions.

3. Median Width. The 101-ft typical median width applies to new construction or to the dualization of an existing two-lane, two-way facility. Exceptions to this width may be considered for the following:

   a. areas with environmental restrictions,

   b. facilities with Type 1 access control,

   c. facilities where full-width bridges have already been constructed at a lesser width, or

   d. other areas with justification.

   For a reconstruction project on an existing divided facility, the existing median width may be retained.

4. Bridge Width. Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

5. Roadside Clear Zone. The recommended clear zones are based on design speed, side slopes and traffic volumes. See Section 9-2.0. All values are measured from the edge of traveled way or auxiliary lane, if present.

6. Slopes. If high-volume change soil is present, see Section 12-2.04.

7. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

8. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

2-8.0 EXCEPTIONS TO GEOMETRIC DESIGN CRITERIA

2-8.01 Controlling Design Criteria

Controlling design criteria are those highway design elements which are judged to be the most critical indicators of a highway's safety and its overall serviceability. Obviously, not every piece of design information in the Department's *Design Manual* meets this definition. Therefore, the Department and FHWA have identified those design elements which qualify as controlling design criteria. Any proposed design that does not provide at least the minimum design value must pass through the formal documentation process and be approved by the Chief Engineer and, where applicable, FHWA.

The design exception process only applies to the highway mainline; i.e., it does not apply to auxiliary lanes (e.g., climbing lanes, turn lanes) nor to interchange ramps. The following identifies those highway mainline design elements which require approval when the Department's values are not met:

1. design speed,
2. travel lane width,
3. shoulder width,
4. bridge width,
5. structural capacity,
6. maximum degree of curvature for horizontal curves,
7. K-values for minimum stopping sight distance at crest and sag vertical curves,
8. maximum grades,
9. middle ordinate values for minimum stopping sight distance around horizontal curves,
10. cross slopes for travel lanes and shoulders,
11. superelevation rates,
12. vertical clearances, and
13. horizontal clearances to obstructions on curbed facilities.

The tables for new construction/reconstruction projects are presented in Sections 2-7.0, 13-3.0 and 14-2.0. On all of these tables, an asterisk has been placed beside each controlling design element.

2.8.02 Procedures for Exceptions

This Section discusses the Department's policies and procedures for identifying and justifying exceptions to geometric design criteria. This applies to new construction, reconstruction and 3R projects on the State highway system and to all Federal-aid projects.

In general, designers are responsible for meeting or exceeding all design criteria in the *Manual*. This policy is intended to ensure that the Department will provide a highway system which meets the transportation needs of the State and provides a reasonable level of safety, comfort and convenience for the traveling public.

2-8.02.01 Department Procedures

The following establishes the procedures the highway designer must follow for proposed exceptions to any of the controlling design criteria:

1. The designer will investigate the impacts of meeting the minimum design criteria. Factors to evaluate include but are not limited to:

   a. amount and character of the current and design year traffic;
   b. number and type of traffic accidents, fatal accident rate and fatal accident rate on a comparable system;
   c. project's compatibility with adjacent sections of roadway (e.g., comparisons of lane widths, shoulder widths, slopes, curvature);
   d. probable timing for any scheduled future improvements within the proposed project limits;
   e. cost analysis (maximum service and safety benefits for dollar invested);
   f. right-of-way costs and impacts;
   g. environmental considerations; and
   h. assessments of the structural and functional adequacy of existing bridges.

2. Once the designer has determined that the minimum design criteria cannot reasonably be satisfied after evaluation of the factors in Comment #1, a report will be prepared that identifies the required design value, as provided within the tables of this *Manual*, and the proposed deviation from that value. The report will also fully document the basis for an exception request. Any safety enhancement measures to be undertaken as part of the proposed project should also be described within the report.

3. The report will be submitted to the Roadway Design Division Engineer for review and action.

4. The documentation for the design exception will become part of the project file (see Section 1-5.0).

2-8.02.02 Additional Procedures for Exceptions Requiring Federal Action

All proposed exceptions to the AASHTO values or approved State 3R values for the controlling criteria on all Interstate projects (regardless of project type, cost or class of funding (Federal, State, local or private)) and on all NHS non-Interstate projects if the estimated cost equals or exceeds $5,000,000 (regardless of project type or class of Federal-aid funds) must be submitted to the FHWA Division Office for review and approval. Various factors must be considered, fully evaluated, documented and submitted to the Roadway Design Division Engineer by the designer, as discussed in Section 2-8.02.01, for completion of the Department's internal process prior to the submission of an exception request and evaluation report to the FHWA Division Office.

The Department has full approval authority for design exceptions for all projects not listed in the preceding paragraph.

2-9.0 QUALIFYING WORDS

Many qualifying words are used in geometric design and in this *Manual*. For consistency and uniformity in the application of various design criteria, the following definitions apply:

1. Shall, require, will, must. A mandatory condition. Designers are obligated to adhere to the criteria and applications presented in this context or to perform the evaluation indicated. For the application of geometric design criteria, this *Manual* limits the use of these words.

2. Should, recommend. An advisory condition. Designers are strongly encouraged to follow the criteria and guidance presented in this context, unless there is reasonable justification not to do so.

3. May, could, can, suggest, consider. A permissive condition. Designers are allowed to apply individual judgment and discretion to the criteria when presented in this context. The decision will be based on a case-by-case assessment.

4. Desirable, preferred. An indication that the designer should make every reasonable effort to meet the criteria and that he/she should only use a "lesser" design after due consideration of the "better" design.

5. Ideal. Indicating a standard of perfection (e.g., traffic capacity under "ideal" conditions).

6. Minimum, maximum, upper, lower (limits). Representative of generally accepted limits within the design community but not necessarily suggesting that these limits are inviolable. However, where the criteria presented in this context will not be met, the designer will in many cases need approval.

7. Practical, feasible, cost-effective, reasonable. Advising the designer that the decision to apply the design criteria should be based on a subjective analysis of the anticipated benefits and costs associated with the impacts of the decision. No formal analysis (e.g., cost-effectiveness analysis) is intended, unless otherwise stated.

8. Possible. Indicating that which can be accomplished. Because of its rather restrictive implication, this word will not be used in this *Manual* for the application of design criteria.

9. Significant, major. Indicating that the consequences from a given action are obvious to most observers and, in many cases, can be readily measured.

10. Insignificant, minor. Indicating that the consequences from a given action are relatively small and not an important factor in the decision-making for road design.

11. Warranted, justified. Indicating that some well-accepted threshold or set of conditions has been met. As used in this *Manual*, "warranted" or "justified" may apply to either objective or subjective evaluations. Note that, once the warranting threshold has been met, this is an indication that the design treatment should be considered and evaluated not that the design treatment is automatically required.

12. Standard. Indicating a design value which cannot be violated without severe consequences. This suggestion is generally inconsistent with geometric design criteria. Therefore, "standard" will not be used in this *Manual* to apply to geometric design criteria.

13. Guideline. Indicating a design value which establishes an approximate threshold which should be met if considered practical.

14. Criteria. A term typically used to apply to design values, usually with no suggestion on the criticality of the design value. Because of its basically neutral implication, this *Manual* frequently uses "criteria" to refer to the design values presented.

15. Typical. Indicating a design practice which is most often used in application and which is likely to be the "best" treatment at a given site.

16. Target. If practical, criteria the designer should be striving to meet. However, not meeting these criteria will typically not require a justification.

17. Acceptable. Design criteria which do not meet desirable values, but yet is considered to be reasonable and safe for design purposes.

18. Policy. Indicating MDOT practice which the Department generally expects the designer to follow, unless otherwise justified.

2-10.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 1990, 1994.

2. *Highway Capacity Manual*, Special Report No. 209, Transportation Research Board, 1985.

3. *Federal Laws and Material Relating to the Federal Highway Administration*, Federal Highway Administration, November, 1976.

CHAPTER 3

HORIZONTAL ALIGNMENT

MISSISSIPPI DESIGN MANUAL

Chapter 3

HORIZONTAL ALIGNMENT

Section		Page
3-1.0	LOCATION SURVEYS	3-1
3-2.0	GENERAL CRITERIA	3-1
3-3.0	HORIZONTAL CURVES	3-2
3-3.01	Curvature Types and Selection	3-2
3-3.01.01	Simple Curves	3-2
3-3.01.02	Spiral Curves	3-4
3-3.01.03	Compound Curves	3-4
3-3.02	Traveled Way Widening	3-4
3-3.03	Equations in Stationing	3-7
3-3.04	Divided Highways	3-7
3-3.04.01	Parallel Curves	3-9
3-3.04.02	Non-Parallel Curves	3-9
3-4.0	SUPERELEVATION	3-9
3-4.01	Superelevation Rate	3-9
3-4.02	Superelevation Transition Length	3-11
3-4.02.01	Rate of Transition	3-11
3-4.02.02	Width of Rotation	3-11
3-4.02.03	Segments	3-11
3-4.02.04	Application of Transition Length	3-11
3-4.03	Superelevation Axis of Rotation	3-12
3-4.03.01	Case I: Superelevation Rotated About the Centerline	3-12
3-4.03.02	Case II: Superelevation Rotated About the Median Edge	3-15
3-4.04	Shoulder Superelevation	3-15
3-5.0	HORIZONTAL SIGHT DISTANCE	3-15
3-5.01	Middle Ordinate for Stopping Sight Distance	3-15
3-5.01.01	Length of Curve > Stopping Sight Distance	3-15
3-5.01.02	Length of Curve < Stopping Sight Distance	3-17
3-5.01.03	Longitudinal Barriers	3-18
3-5.02	Decision Sight Distance	3-18
3-5.03	Passing Sight Distance	3-18
3-6.0	REFERENCES	3-19

Chapter 3

HORIZONTAL ALIGNMENT

This Chapter discusses the details for the proper design of horizontal alignment. Much of the criteria is equally applicable to rural and urban roads. However, the horizontal alignment for low-speed urban streets (design speed of 45 mph and below) involves special considerations. These are discussed in detail in Section 14-2.0.

3-1.0 LOCATION SURVEYS

Most location surveys for State highways in Mississippi are the responsibility of the Districts and are performed by District personnel. Completed surveys and design recommendations are submitted to the Roadway Design Division.

The designer will normally adhere to the alignment established by the Districts. If unusual circumstances are encountered during the design process, office projections may be computed for variations from the surveyed line, or additional field surveys may be requested if the departure from the survey line is significant.

The ultimate quality of the horizontal alignment is thus influenced by decisions of both District and Roadway Design personnel. Horizontal alignment decisions should be governed by the criteria set forth in this Chapter.

3-2.0 GENERAL CRITERIA

Many design criteria for horizontal alignment (e.g., design speed, maximum degree of curvature, superelevation rates) are specific and can be measured. Other factors cannot be defined and require the designer's judgment and consideration of existing conditions. General criteria for horizontal alignment include:

1. Alignment should be as directional as possible consistent with physical and economic constraints. Avoid a winding alignment composed of short curves.

2. Alignment should be consistent. Avoid sharp curves at the ends of long tangents and sudden changes from areas of flat curvature to sharp curvature. Where sharp curvature must be introduced, it should be approached by successively sharper curves from the generally flat curvature.

3. Avoid sharp curves for small deflection angles. Curves should be sufficiently long enough to preclude the appearance of a kink. Curves should be at least 500 ft long for a central angle of 5 degrees or less. On high-speed controlled access facilities, a desirable length of curve should be about 30 times the design speed, or $L = 30V$, where V is in mph.

4. Avoid sharp curves on long, high fills. Under these conditions it is difficult for drivers to perceive the extent of horizontal curvature.

5. Avoid abrupt reversals in alignment (S curves). There should be a sufficient tangent distance between the curves to ensure proper superelevation transition for both curves and sufficient distance for required signing.

6. Avoid "broken-back curves" (short tangent between two curves in the same direction). This arrangement is not pleasing in appearance. The preferred approach is to use spiral transition curves which provide the needed smoothness and continuity.

7. Avoid sharp horizontal curvature near crest or sag vertical curves. These combinations of curvature can cause sight distance problems. Horizontal alignment should be carefully coordinated with profile design.

8. Avoid horizontal curves on bridges. These may cause design, construction and operational problems. Where a curve is necessary on a bridge, a simple curve should be used on the bridge, and any curve or superelevation transitions should be placed on the approaching roadway.

3-3.0 HORIZONTAL CURVES

3-3.01 Curve Types and Selection

Deflectional changes in alignment may be accomplished by using a simple curve or by using a spiral curve leading into and out of a simple curve.

3-3.01.01 Simple Curves

The following applies to simple curves:

1. Layout. A simple curve is that portion of the arc of a circle which achieves the desired deflection without using an entering or exiting transition. The degree of curvature is used to define a horizontal curve. The degree of curvature (D) (arc definition) is defined as the central angle which subtends an arc length of 100 ft. Figure 3-3A illustrates a typical simple curve layout. The maximum degree of curvature for various design speeds is discussed in Section 3-4.01.

2. Computations. Usually for simple curves, the PI station, the deflection angle (Δ) and the degree of curvature (D) are established. The remaining curve data can be computed using the formulas in Figure 3-3A.

3. Stationing. Curve stationing should be determined as follows:

   a. PC Station = PI Station - T
   b. PT Station = PC Station + L

   Figure 3-3A defines the terms.

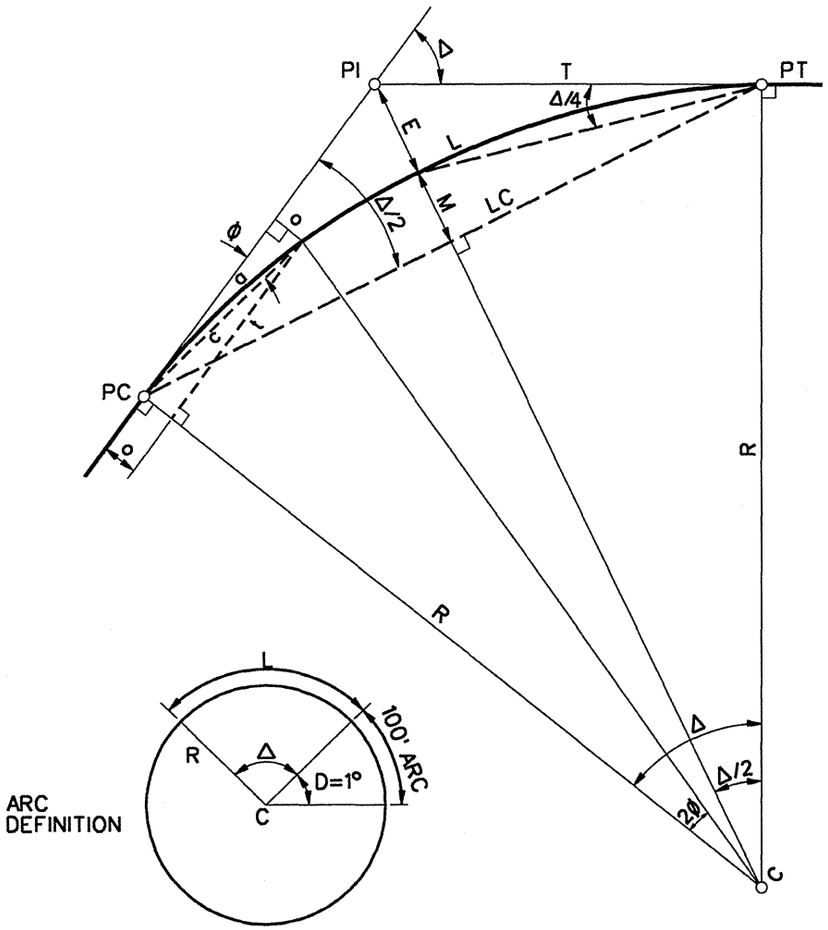
4. Accuracy (For Closure Purposes). Curve data should be calculated and recorded to the following level of accuracy:

   a. the degree of curvature (D) to the nearest minute for new horizontal curves,

   b. the deflection angle (Δ) to the nearest one thousandth of a second (i.e., three decimal places), and

   c. all linear distances to the nearest one thousandth of a foot (i.e., three decimal places).

   Alignments which incorporate a previously defined horizontal curve will use the existing degree of curvature.



PC	=	Point of Curve (Beginning of Curve)
PT	=	Point of Tangency (End of Curve)
PI	=	Point of Intersection of Tangents
PRC	=	Point of Reverse Curve
PCC	=	Point of Compound Curve

Δ	=	Deflection angle, degrees
T	=	Tangent distance, ft. T = distance from PC to PI or distance from PI to PT
L	=	Length of curve, ft. L = distance from PC to PT along curve
R	=	Radius of curve, ft
D	=	Degree of curvature (100′ arc definition)
E	=	External distance (PI to mid-point of curve), ft
C	=	Intersection of radii at center of circular arc
LC	=	Length of long chord (PC to PT), ft
M	=	Middle ordinate, ft (mid-point of arc to mid-point of long chord)
a	=	Length of arc to any point on a curve, ft
c	=	Length of chord from PC to any point on curve, ft
φ	=	Deflection angle from tangent to any point on curve, degrees
t	=	Distance along tangent from PC to any point on curve, ft
o	=	Tangent offset to any point on curve, ft

$$R = 100\,\frac{(360/2\pi)}{D} = 5729.577951/D$$

$$T = R(\tan\,(\Delta/2)) = R\,\frac{\sin\,(\Delta/2)}{\cos\,(\Delta/2)}$$

$$L = \frac{\Delta}{360}\,2\,\pi\,R = 100\,\Delta/D$$

$$E = \frac{R}{\cos\,(\Delta/2)} - R = T\tan\,(\Delta/4)$$

$$LC = 2R\,(\sin\,(\Delta/2) = 2T\,(\cos\,\Delta/2)$$

$$M = R(1-\cos\,(\Delta/2)) = E\cos\,(\Delta/2)$$

$$a = \frac{(200\varphi)(2\pi R)}{100(360)} = \frac{(\varphi)(\pi R)}{90}$$

$$c = 2R\left(\sin\,\frac{(100)(360a)}{(200)(2\pi R)}\right) = 2R\,(\sin\,\frac{90a}{\pi R})$$

$$\varphi = \frac{90a}{(\varphi)(\pi R)}$$

$$\cos\,\varphi = (R - o)/2R$$

$$t = R\sin 2\varphi = (c)\cos\,\varphi$$

$$o = (c)\sin\,\varphi$$

$$o = R - \sqrt{R^2 - t^2}$$

$$o = R - (R\cos\,2\varphi)$$

$$o = R(1 - \cos\,2\varphi)$$

$$\pi \approx 3.141592654$$

TYPICAL SIMPLE CURVE LAYOUT
Figure 3-3A

3-3.01.02 Spiral Curves

The following applies to spiral curves:

1. Layout. Spiral curves provide a gradual vehicular transition into and out of circular curves without a sharp break from the tangent sections. The spiral also provides a convenient arrangement for the superelevation runoff length. Typical design practice is to use this length for the length of the spiral curve (L_S). Superelevation runoff lengths are discussed in Section 3-4.02. Figure 3-3B illustrates a typical spiral curve layout.

2. Warrants. Table 3-4A indicates the recommended threshold for the use of a spiral curve. On rural State highways, the designer should use a spiral curve where the degree of curvature for a given design speed yields a required superelevation rate greater than 5.0%. This includes all curves below the heavy lines in Table 3-4A.

3. Computation. Usually for spiral curves, the Master PI station, the deflection angle (Δ) and the degree of curvature (D_c) are established. The length of the spiral curve (L_s) will be set equal to the length of the superelevation runoff (Table 3-4A). Based on the values of Δ, L_S and D_c, θ_s can be calculated as indicated in Figure 3-3B, and the p and k values can be read from Table II in *Transition Curves for Highways* by Joseph Barnett. The tangent length (T_s), the external distance (E_s) and the remaining spiral curve data can be computed as described in Figure 3-3B.

4. Stationing. Curve stationing should be determined as follows:

   a. Master PI Station - T_s = TS Station
   b. TS Station + L_s = SC Station
   c. SC Station + L_c = CS Station
   d. CS Station + L_s = ST Station

5. Accuracy. The accuracy of curve data for simple curves also applies to spiral curves. See Section 3-3.01.01

3-3.01.03 Compound Curves

Compound curves are appropriate for interchange ramps, turning roadways and intersection curb returns. Occasionally, they may be used on highway mainlines but only where some physical control prevents the use of a spiral curve or simple curve. Where a compound curve is used on a highway mainline, the degree of curvature of the flatter circular arc (D_1) should not be more than 50 percent greater than the degree of curvature of the sharper circular arc (D_2); i.e., $D_1 \geq 0.67\ D_2$.

3-3.02 Traveled Way Widening

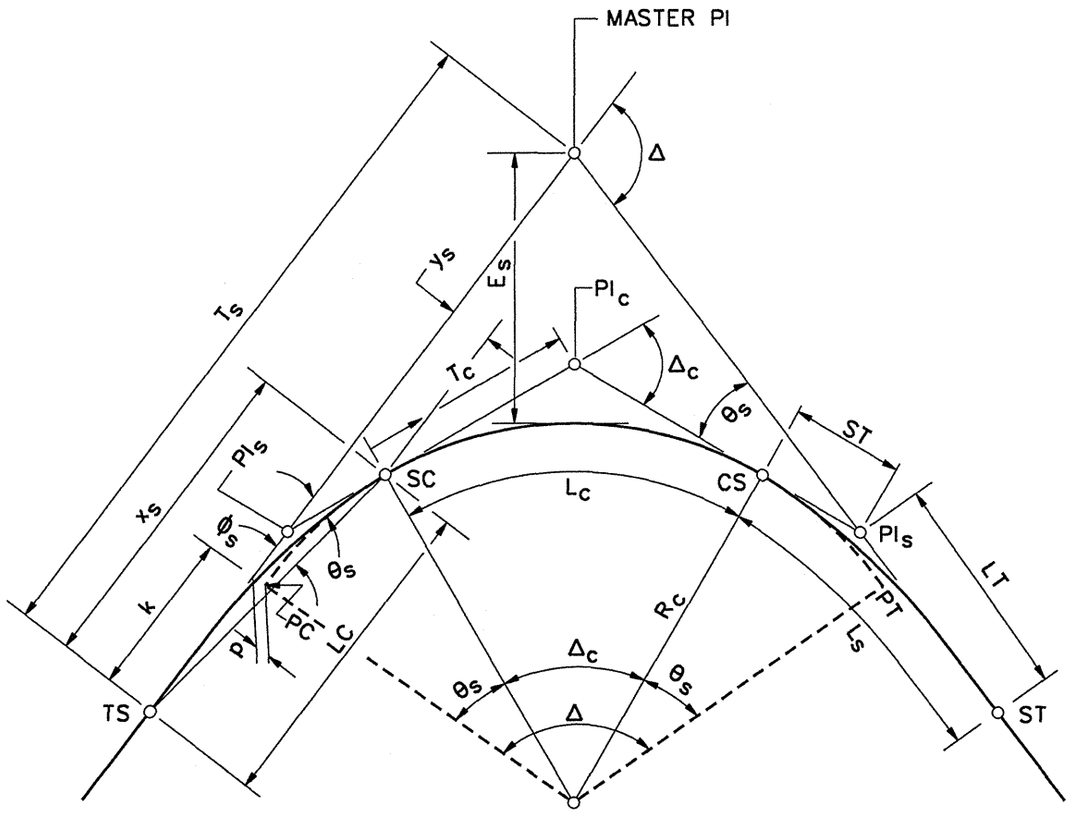
Traveled way widening may be necessary on the inside edge of horizontal curves for the following reasons:

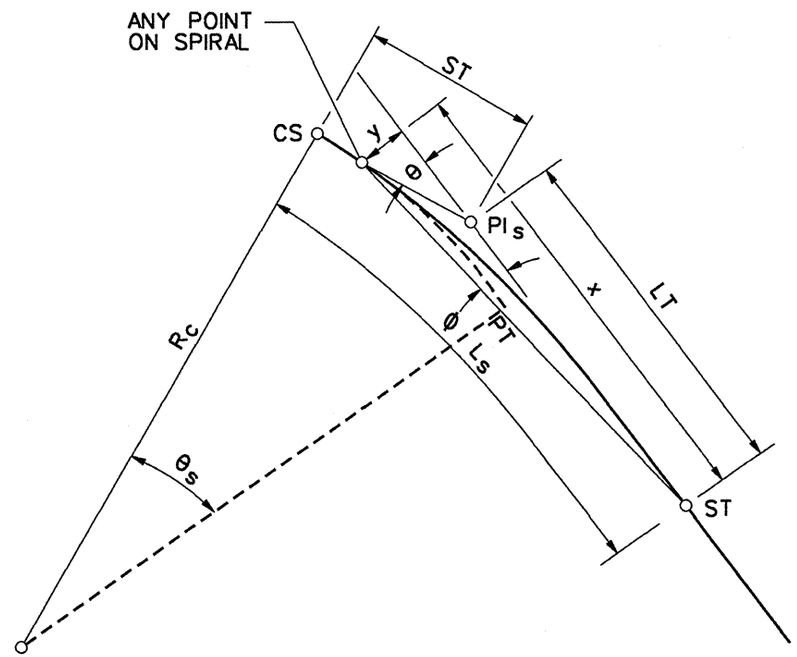
1. Vehicles (especially trucks) occupy a greater effective width because rear wheels track inside of front wheels when rounding a curve.

2. Where lane widths are less than 12 ft, drivers may experience difficulty in steering within the traffic lane on curves.

Table 3-3A presents design values for traveled way widening on highway curves. Traveled way widening will be applied on the inside edge of pavement, and transition distances will be attained over the superelevation runoff length. See Section 3-4.02.

On existing facilities with narrow lanes and sharp curves, there may be a need for traveled way widening on curves as part of a 3R project design. 3R projects are discussed in detail in Chapter 11.





Layout for Any Point on Spiral

Notes:

1. This figure is consistent with the nomenclature in *Transition Curves for Highways* by Public Roads Administration (Joseph Barnett).

2. See next page for definition of terms.

TYPICAL SPIRAL CURVE LAYOUT

Figure 3-3B

SPIRAL TRANSITION CURVE NOMENCLATURE

Master PI = Point of intersection of the main tangents.

PC = Point at which the circular curve extended becomes parallel to the line from TS to the Master PI.

PT = Point at which the circular curve extended becomes parallel to the line from ST to the Master PI.

PI_c = Point of intersection of circular curve tangents.

PI_s = Point of intersection of the main tangent and tangent of circular curve.

TS = Tangent to spiral; common point of spiral and near transition.

SC = Spiral to curve; common point of spiral and circular curve of near transition.

CS = Curve to spiral; common point of circular curve and spiral of far transition.

ST = Spiral to tangent; common point of spiral and tangent of far transition.

D_C = Degree of circular curve (SC to CS)

R_C = Radius of the circular curve (SC to CS), ft

L_S = Length of spiral, ft

L_C = Length of circular curve, ft

T_S = Tangent distance Master PI to TS or ST, ft

T_C = Tangent distance from SC or CS to Pi_c, ft

E_S = External distance Master PI to midpoint of circular curve, ft

LT = Long tangent of spiral only, ft

ST = Short tangent of spiral only, ft

LC = Long chord of spiral, ft

p = Offset distance from the main tangent to the PC or PT of the circular curve produced, ft

k = Distance from TS to point on main tangent opposite the PC of the circular curve produced, ft

Δ = Total deflection angle between main tangents of the entire curve, degrees

Δ_C = Deflection angle between tangents at the SC and the CS or the central angle of the circular curve, degrees

θ_S = Central angle between the tangent of the complete curve and the tangent at the SC; i.e., the "spiral angle," degrees

ϕ_S = Spiral deflection angle from tangents at TS to SC or from ST to SC, degrees

$x_S y_S$ = Coordinates of SC from the TS or of CS from ST.

L = Length of spiral arc from the TS or ST to any point on the spiral, ft

x,y = Coordinates to any point on the spiral from TS or ST.

ϕ = Spiral deflection angle from TS or ST to any point on spiral, degrees

θ = The central angle of spiral arc L to any point on the spiral, degrees. θ equals θ_s when L equals L_s. Note that the θ referred to in Table II of *Transition Curves for Highways* is actually θ_s. *Note: For Equations 17 and 18, θ is in radians.*

$$R = 100\,\frac{360/2\pi}{D} = 5729.577951/D$$

CURVE FUNCTIONS

1. $\theta_s = (L_s/R_c)\,(90/\pi) = L_S\,D_C/200$
2. $\Delta_c = \Delta - 2\theta_s$
3. $L_c = \dfrac{\Delta_c}{360}\,2\pi R_c$
4. $T_s = (R_c + p)\tan(\Delta/2) + k$
5. $E_s = (R_c + p)\,(1/\cos(\Delta/2) - 1) + p = \left[\dfrac{(R_c + p)}{\cos(\Delta/2)} - (R_c + p)\right] + p$
6. p and k are obtained from *Transition Curves for Highways* by Barnett.

SPIRAL FUNCTIONS

Corrections for C in Formula: $\varphi = \dfrac{\theta}{3} - C$

θ_s in Degrees	15	20	25	30	35	40	45	50
C in Minutes	0.2	0.4	0.8	1.4	2.2	3.4	4.8	6.6

7. φ (approx.) $= \dfrac{\theta}{3}$, *if* $\theta_s < 15°00'$
8. φ (approx.) $= \dfrac{\theta}{3} - C$, *if* $\theta_s \geq 15°00'$
9. $\varphi = \dfrac{\theta_s}{3}\left[\dfrac{L}{L_s}\right]^2$
10. Exact value of φ by coordinates: $\tan\varphi = \dfrac{y}{x}$
11. $ST = \dfrac{y_s}{\sin\theta_s}$
12. $LT = x_s - \left(\dfrac{y_s}{\tan\theta_s}\right)$
13. $LC = \dfrac{x_s}{\cos\varphi_s}$
14. $x_s = LC\cos\varphi_s$
15. $y_s = LC\sin\varphi_s$
16. $\theta = \dfrac{L^2}{L_S^2}\,\theta_s$
17.* $x = L\left(1 - \dfrac{\theta^2}{10} + \dfrac{\theta^4}{216} - \dfrac{\theta^6}{9360} + \dfrac{\theta^8}{685\,440}\right)$
18.* $y = L\left(\dfrac{\theta}{3} - \dfrac{\theta^3}{42} + \dfrac{\theta^5}{1320} - \dfrac{\theta^7}{75\,600} + \dfrac{\theta^9}{6\,894\,720}\right)$

**θ is in radians*

Note: These equations are based on *Transition Curves for Highways* by Barnett.

TYPICAL SPIRAL CURVE LAYOUT
Figure 3-3B
(continued)

Table 3-3A

DESIGN VALUES FOR TRAVELED WAY WIDENING ON HIGHWAY CURVES (Feet)

Degree of Curvature	22-ft Traveled Way					20-ft Traveled Way			
	Design Speed (mph)					Design Speed (mph)			
	30	40	50	60	70	30	40	50	60
1	0.5	0.5	0.5	1.0	1.0	1.5	1.5	1.5	2.0
2	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5
3	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5
4	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0
5	1.5	1.5	2.0	2.0		2.5	2.5	2.5	3.0
6	1.5	2.0	2.0	2.5		2.5	3.0	3.0	3.5
7	1.5	2.0	2.5			2.5	3.0	3.5	
8	2.0	2.0	2.5			3.0	3.0	3.5	
9	2.0	2.5	3.0			3.0	3.5	4.0	
10-11	2.0	2.5				3.0	3.5		
12-14.5	2.5	3.0				3.5	4.0		
15-18	3.0					4.0			
19-21	3.5					4.5			
22-25	4.0					5.0			
26-26.5	4.5					5.5			

Notes:

1. *Extra width to be added on the inside of curve. The specified extra width should be added at a uniform rate throughout superelevation runoff. Centerline stripe should equally divide widened traveled way.*

2. *On 4-lane roadways, multiply table values by 2.*

3. *Where there are significant semitrailers, increase the roadway widths in the table by the values listed in Table 3-3B.*

Table 3-3B

INCREASED WIDTHS FOR SEMITRAILERS

WB-50/WB-60		WB-62	
Degree of Curvature	Increased Width (ft)	Degree of Curvature	Increased Width (ft)
17° > D ≥ 10°	0.5	9° > D ≥ 4°	0.5
D ≥17°	1.0	12° > D ≥ 9°	1.0
		21° > D ≥ 12°	1.5
		28° > D ≥ 21°	2.0
		D ≥ 28°	2.5

Note: These values should be added to those obtained in Table 3-3A.

3-3.03 Equations in Stationing

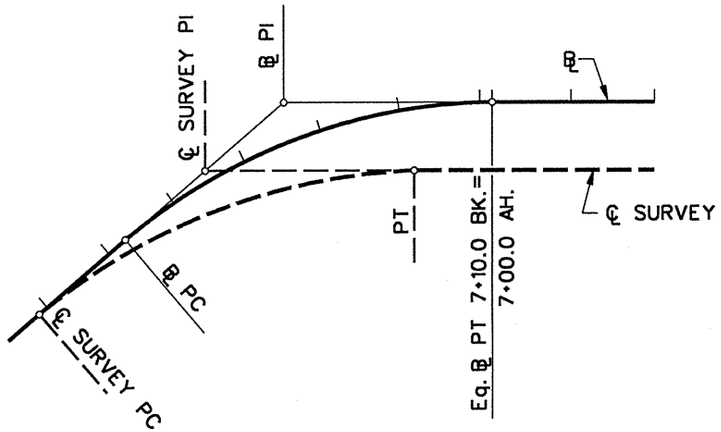
Departures from the line surveyed in the field usually involve changes in the length of alignment. To avoid revising the stationing throughout the project, equations in stationing are introduced. The equation identifies two station numbers — one that is correct when measuring on the line back of the equation and one that is correct when measuring on the line ahead of the equation.

For parallel construction projects with base lines that are not common, the tangent sections of the base lines should have stationing that is equal and opposite to each other. To accomplish this, an equation should be added at the PC or PT of horizontal curves, but not at both the PC and PT.

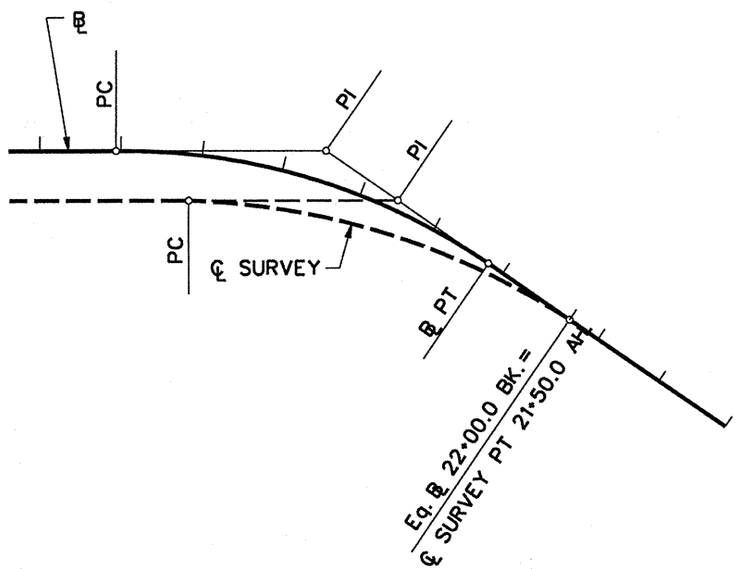
Typical equations in stationing are illustrated in Figure 3-3C.

3-3.04 Divided Highways

Where the median width of a divided highway is a constant distance of 64 ft or less, the stationing and all other alignment computations are based on a single survey base



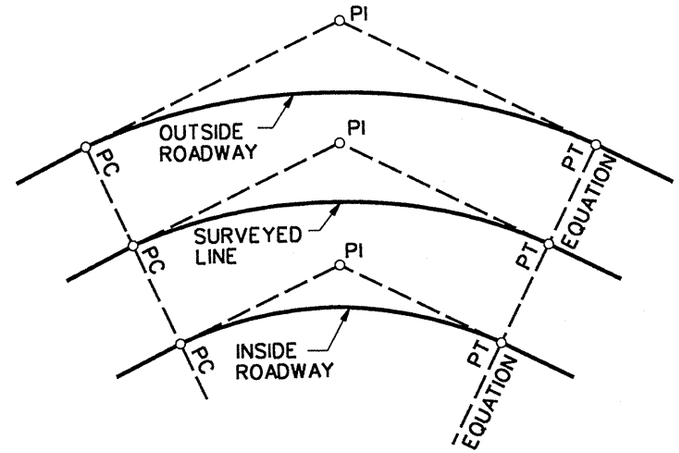
PROJECTED LINE DEPARTS FROM SURVEYED LINE



PROJECTED LINE RETURNS TO SURVEYED LINE

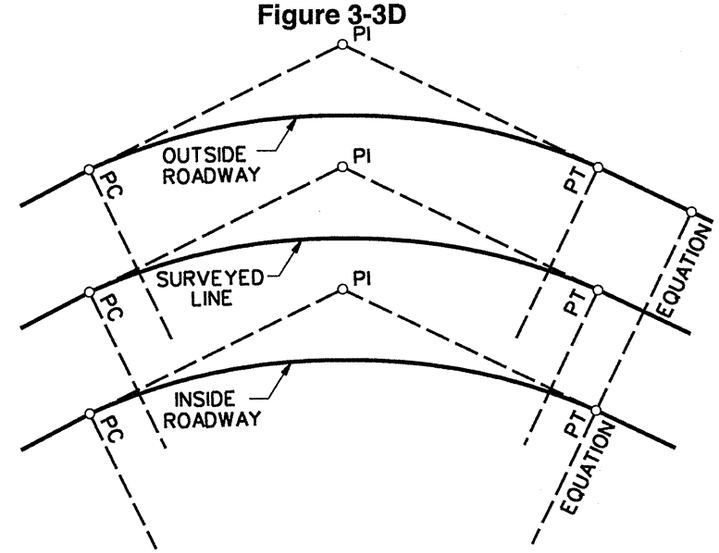
EQUATIONS IN STATIONING

Figure 3-3C



PARALLEL CURVES

Figure 3-3D



NON-PARALLEL CURVES

Figure 3-3E

line, normally the center of the median. A common profile grade and one set of curve data established from the base line will serve both roadways.

Where the median width on a divided highway exceeds 64 ft, each roadway should have separate horizontal and vertical controls independent of a single survey base line. Each roadway will have its own separate profile grade and horizontal curve data.

3-3.04.01 Parallel Curves

The most common practice for horizontal curvature on divided highways is the use of parallel curves (common center on radii) as illustrated in Figure 3-3D. The deflection angle (Δ) and PC stationing will be identical for the survey line, inside roadway and outside roadway. The remaining curve data including D, L and T will vary for each line. Equations in stationing will re-establish common stationing from the PT on both roadways as discussed in Section 3-3.03.

3-3.04.02 Non-Parallel Curves

Desirably, use parallel curves for horizontal curvatures on divided highways. Non-parallel curves can be used where field conditions warrant. These are illustrated in Figure 3-3E. A reasonable approximation to parallel conditions is attained if the degree of curvature (D) is the same for the inside and outside roadways and for the survey line. The basic curve data including Δ, L and T will be identical for all three lines. Stationing of the PC, PI and PT will differ for each line. An equation for stationing should be placed at the PT of the inside roadways to reestablish common stationing from that point ahead and opposite the PT of the inside roadway to reestablish common stationing for the outside roadway.

3-4.0 SUPERELEVATION

Table 3-4A provides the designer with the:

1. superelevation rate (discussed in Section 3-4.01);
2. superelevation runoff length (discussed in Section 3-4.02);
3. impacts of the axis of rotation (discussed in Section 3-4.03); and
4. the maximum degree of curvature for each design speed and e = 10.0%.

Table 3-4A applies to rural highways (all design speeds) and urban facilities (V ≥ 55 mph). Superelevation for other urban highways and streets is discussed in detail in Section 14-2.0.

3-4.01 Superelevation Rate

The Department has adopted e_{max} = 10% for the design of rural highways (all design speeds) and urban facilities (V ≥ 55 mph). For other urban highways and streets, e_{max} = 6%. See Section 14-2.0.

Table 3-4A provides superelevation rates for various combinations of curve for each design speed; the designer should provide flatter curvature where the maximum degree of curvature will not provide the necessary stopping sight distance for horizontal curves. See Section 3-5.0.

The indicated superelevation rates are applicable regardless of the number of lanes.

A horizontal curve with a small degree of curvature does not require superelevation. For a given design speed, the normal crown section (NC) used on tangents (2.0% slope typical) can be maintained throughout a very flat curve. On successively sharper curves for the same design speed, a point is reached where a slope equal to the cross slope across the total traveled way width is necessary (RC). As the curvature increases, it becomes necessary to provide a superelevation grater than 2%. At this point, the designer will use the superelevation rates in Table 3-4A for the horizontal curve. Table 3-4B provides the maximum degree of curvature for a normal crown slope and reverse crown slope at various design speeds.

Table 3-4A

SUPERELEVATION RATE AND LENGTH OF RUNOFF
(Rural Highways (All Design Speeds) and Urban Facilities (V ≥55 km/h))

D*	V = 30 mph			V = 40 mph			V = 50 mph			V = 55 mph			V = 60 mph			V = 65 mph			V = 70 mph		
		L (ft)			L (ft)			L (ft)			L (ft)			L (ft)			L (ft)			L (ft)	
	e	A	B	e	A	B	e	A	B	e	A	B	e	A	B	e	A	B	e	A	B
0°15′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
0°30′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200
0°45′	NC	0	0	NC	0	0	RC	150	150	RC	160	160	.023	175	175	.025	190	190	.028	200	200
1°00′	NC	0	0	NC	0	0	.021	150	150	.025	160	160	.030	175	175	.033	190	190	.037	200	200
1°30′	NC	0	0	.021	125	125	.031	150	150	.037	160	160	.043	175	190	.048	190	220	.054	200	260
2°00′	RC	100	100	.028	125	125	.040	150	150	.048	160	180	.055	175	230	.062	190	290	.070	220	330
2°30′	.021	100	100	.034	125	125	.049	150	180	.056	160	210	.067	190	280	.075	220	330	.085	260	390
3°00′	.025	100	100	.040	125	125	.057	150	210	.067	170	250	.077	210	320	.087	250	380	.096	280	420
3°30′	.029	100	100	.046	125	140	.065	160	240	.075	190	290	.086	230	350	.095	270	400	.100	300	450
4°00′	.033	100	100	.051	125	160	.072	180	260	.083	210	320	.093	250	380	.099	280	420	D_{max} = 3°30′		
5°00′	.040	100	110	.061	130	190	.083	200	300	.094	240	360	.098	270	400	D_{max} = 4°15′					
6°00′	.046	100	120	.070	150	220	.092	220	330	.099	250	380	D_{max} = 5°15′								
7°00′	.053	100	140	.078	160	240	.098	240	350	D_{max} = 6°30′											
8°00′	.058	110	160	.084	180	260	.100	240	360												
9°00′	.063	120	170	.089	190	280	D_{max} = 8°15′														
10°00′	.068	120	180	.094	200	290															
11°00′	.072	130	200	.097	200	310															
12°00′	.076	140	210	.099	210	310															
13°00′	.080	140	220	.100	210	320															
14°00′	.083	150	220	D_{max} = 13°15′																	
16°00′	.089	160	240																		
18°00′	.093	170	250																		
20°00′	.097	170	260																		
22°00′	.099	180	270																		
24°00′	.100	180	270																		
	D_{max} = 24°45′																				

e_{max} = 10%

Key:
- D = Degree of curvature
- V = Design speed, mph
- e = Superelevation rate, %
- L = Minimum length of superelevation runoff (from adverse crown removed to full super), ft
- A = "L" for 1-lane width of rotation
- B = "L" for 2-lane width of rotation
- NC = Normal Crown (2% cross slope)
- RC = Reverse Crown; superelevate at normal crown slope (2%)

* *For degree of curvature intermediate between table values, use a straight-line interpolation to determine the superelevation rate.*

Notes:

1. Superelevation runoff length (L) is: $L = W \times \frac{e}{S}$, subject to minimum design values,

   where: W = width of pavement rotation (12-ft lanes assumed)
   e = superelevation rate, decimal
   S = maximum longitudinal slope compared to normal highway profile

2. Maximum longitudinal slope:

Design Speed	30 mph	40 mph	50 mph	55 mph	60 mph	65 mph	70 mph
S	0.0067	0.0058	0.0050	0.0047	0.0045	0.0041	0.0040

3. The equation for superelevation runoff length is used directly for rotation of a 1-lane width (i.e., two 12-ft lanes rotated about the centerline), subject to minimum design values. These lengths are listed in Column A. Column B lists the lengths for rotation of a 2-lane width (i.e., a 2-lane pavement rotated about the edge of traveled way, or a 4-lane undivided pavement rotated about the centerline). The Column B lengths are computed by:

   $$L = \frac{1.5 \times W \times e}{S}$$

   For greater widths of rotation:

   L(CTWLTL) = 1.2 x L (Column B) *Note: Assuming that the axis of rotation is about centerline of 5-lane section.*
   L (3 lanes) = 1.33 x L (Column B)
   L (4 lanes) = 1.67 x L (Column B)

4. Total superelevation transition (from "NC" to "e") = superelevation runoff length (L) + tangent runout length. Tangent runouts will be 100 ft.

5. See Table 3-4B for a more precise range for using "NC" and "RC" to superelevate the roadway.

6. Spiral transition should be used below heavy line.

Table 3-4B

DEGREE OF CURVATURE RANGE FOR NORMAL CROWN AND REVERSE CROWN SLOPE (2.0% Typical)

Design Speed (mph)	Degree of Curvature		
	Normal Crown	Reverse Crown	See Table 3-4A
30	D ≤ 1°39′	1°39′ < D ≤ 2°15′	D > 2°15′
40	D ≤ 1°01′	1°01′ < D ≤ 1°22′	D > 1°22′
50	D ≤ 0°42′	0°42′ < D ≤ 0°57′	D > 0°57′
55	D ≤ 0°36′	0°36′ < D ≤ 0°48′	D >0°48′
60	D ≤ 0°30′	0°27′ < D ≤ 0°41′	D >0°41′
65	D ≤ 0°27′	0°27′ < D ≤ 0°36′	D >0°36′
70	D ≤ 0°24′	0°24′ < D ≤ 0°32′	D >0°32′

Note: e_{max} = 10%

3-4.02 Superelevation Transition Length

The superelevation transition length is the distance required to transition the roadway from a normal crown slope to the full superelevation needed. Superelevation should be introduced and removed uniformly over a length adequate for a given design speed. The total transition length is a function of the amount of superelevation, the rate of transition and the width of rotation.

3-4.02.01 Rate of Transition

The rate of transition is the relative longitudinal slope (S) between the outside edge of the traveled way and the point about which the pavement is rotated. Note 2 in Table 3-4A provides the maximum S for the rotation of a 1-lane width at various design speeds.

3-4.02.02 Width of Rotation

Note 3 in Table 3-4A discusses width of rotation. A lane width of 12 ft has been assumed.

3-4.02.03 Segments

The total superelevation transition length consists of two segments — the length of tangent runout and the length of superelevation runoff.

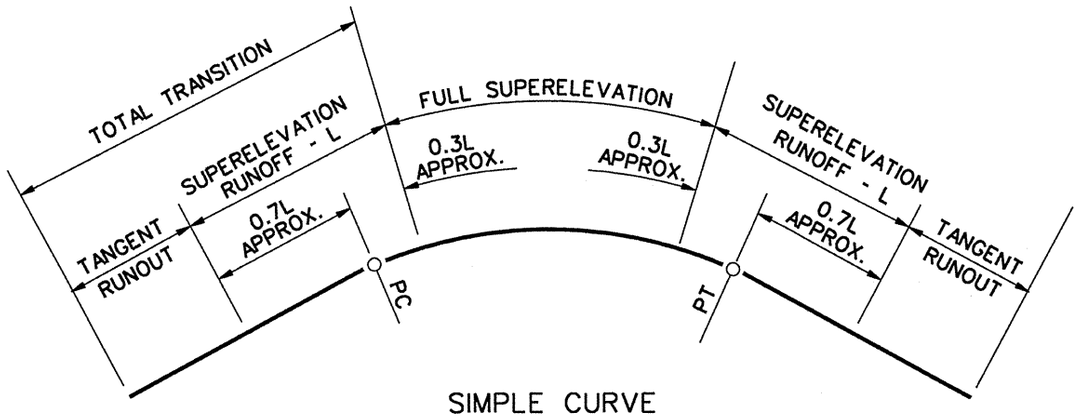
1. Tangent Runout. Tangent runout is a gradual change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed and that portion of the cross section is level. A distance of 100 ft is provided for the length of tangent runout for all design speeds.

2. Superelevation Runoff. Superelevation runoff is a gradual change from the end of the tangent runout to a cross section that is fully superelevated. Table 3-4A provides the minimum length of superelevation runoff for a given design speed, degree of curvature and rotation width. The calculated lengths are subject to minimum design values.

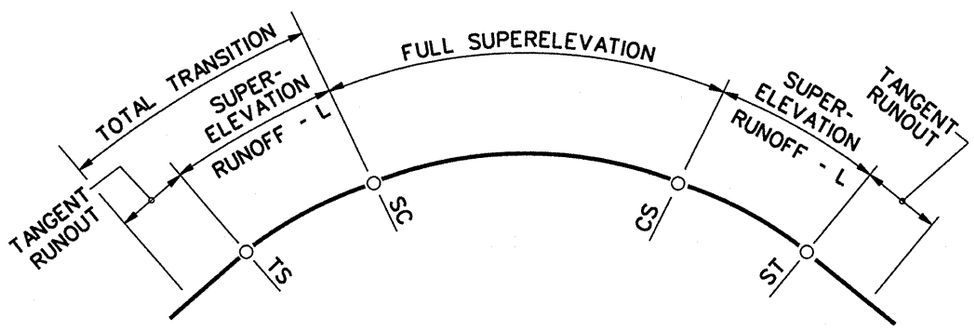
3-4.02.04 Application of Transition Length

Typical applications of the superelevation transition length are shown in Figure 3-4A for the simple curve and spiral curve.

On simple curves approximately 70 percent of the superelevation runoff length should occur along the tangent section (before the PC and after the PT), and about 30 percent of the superelevation runoff length should occur within the circular curve. This is a compromise between placing all of the transition on the tangent section (where superelevation is not needed) and placing all of the transition within the curve (where full superelevation is needed throughout the entire length.) These values may be



SIMPLE CURVE



SPIRAL TRANSITION CURVE

APPLICATION OF THE SUPERELEVATION TRANSITION LENGTH

Figure 3-4A

adjusted for the convenience of making the runoff coincide with 25-ft increments of stationing.

On spiral curves, the length of superelevation runoff is coincident with the length of spiral (TS to SC or CS to ST), and full superelevation is provided between SC and CS.

At the outer limits of the superelevation runoff sections, a tangent runout distance of 100 ft is provided.

3-4.03 Superelevation Axis of Rotation

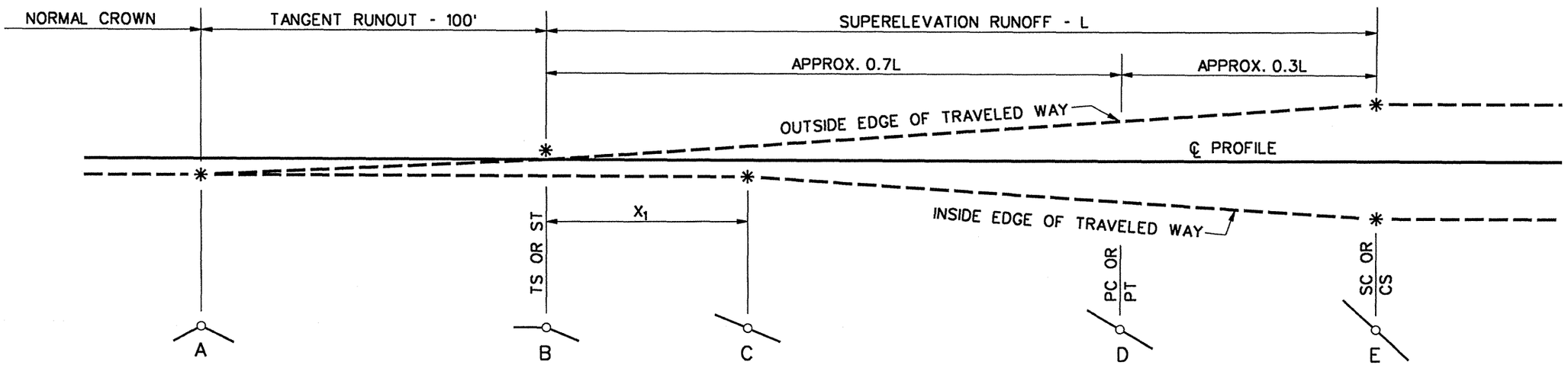
The superelevation axis of rotation is the line about which the pavement is revolved to superelevate the roadway. This line will maintain the normal highway profile throughout the curve and its transitions. The designer should follow the criteria in the following sections to determine the proper axis of rotation.

Angular breaks occur in the vertical profile at points where the transition begins and ends. These should be smoothed by inserting a short vertical curve at the breaks. The length of curve in feet should approximately equal the design speed in miles per hour.

Additional details for superelevation transitions are provided in the *Roadway Design Standard Drawings*.

3-4.03.01 Case I: Superelevation Rotated About the Centerline

The roadway is normally rotated about the centerline profile for 2-lane roadways and multilane undivided roadways. Rotation about the centerline profile is illustrated in Figure 3-4B. On a 2-lane roadway with an auxiliary lane, such as a turning lane, the axis of rotation will be the centerline of the two through lanes.



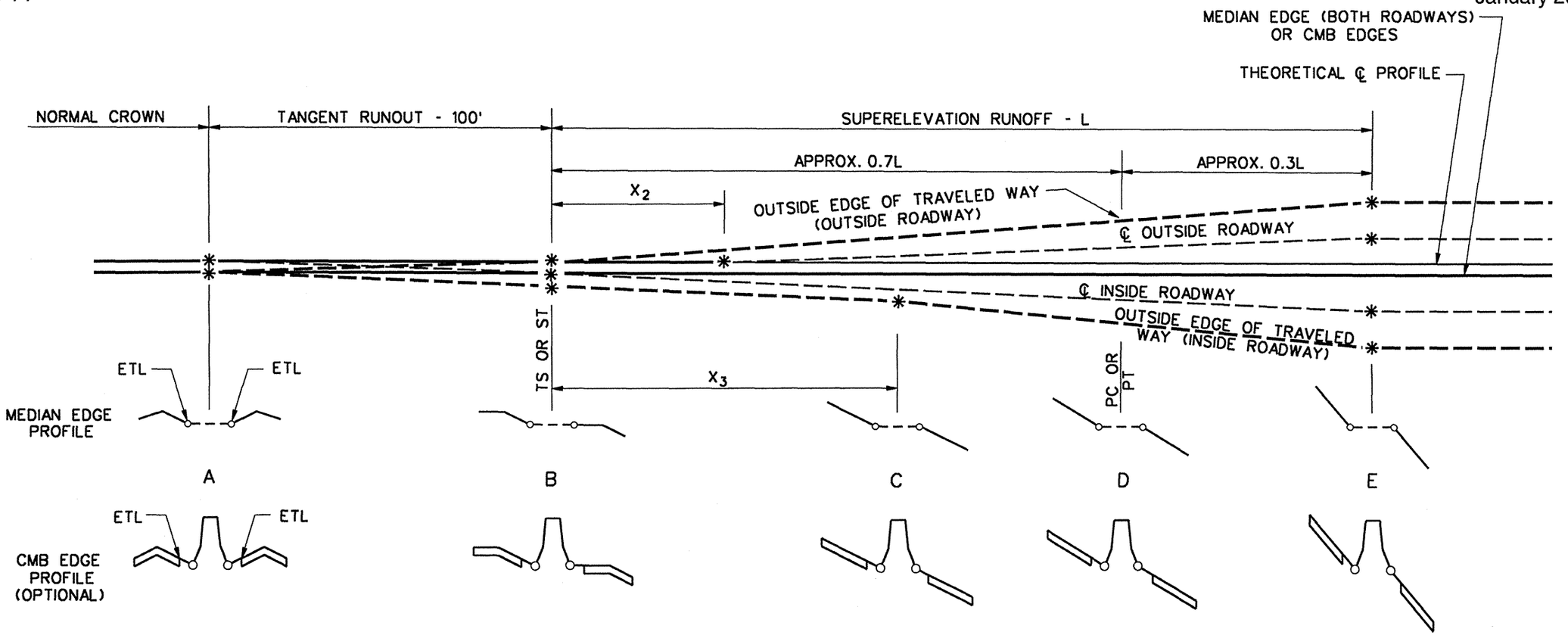
KEY:

A = Normal Crown Slope
B = Adverse Crown Removed
C = Entire Roadway Cross Slope at Normal Crown Rate (2.0% typical) (RC = Reverse Crown)
D = Approximately 0.7 Full Superelevation
E = Full Superelevation
✻ = Short Vertical Curves at Angular Breaks
Length of Curve (ft) ≈ Design Speed (mph)

$$X_1 = L\left[\frac{(NC\ Slope)}{(Full\ Super\ Rate)}\right]$$

SUPERELEVATION DEVELOPMENT — CASE I
(Rotation About Centerline Profile)

Figure 3-4B



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A = Normal Crown Slope
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D = Approximately 0.7 Full Superelevation
E = Full Superelevation
✻ = Short Vertical Curves at Angular Breaks
Length of Curve (ft) ≈ Design Speed (mph)

$$X_2 = L\left[\frac{(NC\ Slope)}{2(Full\ Super\ Rate) - (NC\ Slope)}\right]$$

$$X_3 = L\left[\frac{(NC\ Slope)}{(Full\ Super\ Rate)}\right]$$

SUPERELEVATION DEVELOPMENT — CASE II
(Rotation About Median Edge Profile)

Figure 3-4C

However, on 2-lane highways, it may occasionally be desirable to rotate the roadway about the inside edge of the traveled way. This may be preferable where the lower edge profile is a major control, as for drainage. In these cases, the designer will use the values in Column B of Table 3-4A for the superelevation runoff lengths.

Where the median width on a divided highway exceeds 64 ft, each roadway is rotated separately about its centerline profile. On a 6-lane divided highway (3 lanes in each direction) with a median width greater than 64 ft, each roadway is typically rotated about one of the edges of its inside lane.

3-4.03.02 Case II: Superelevation Rotated About the Median Edge

On divided highways separated by a median width of 64 ft or less, a significant elevation differential may occur between the median edge shoulders of two roadways rotated independently. To minimize the elevation differential, the highway is typically rotated about its two median edge profiles, and the median is held in a horizontal plane as illustrated in Figure 3-4C.

Where a divided highway is separated by a concrete median barrier, the designer may, as an option, consider rotating about the two barrier edge profiles. The concrete barrier is held in a horizontal plane, and the shoulder adjacent to the barrier will be superelevated at the same rate as the traveled way. This option is also presented in Figure 3-4C.

3-4.04 Shoulder Superelevation

The maximum algebraic difference between the traveled way slope and shoulder slope ("rollover") is 7.0%. For example, where the superelevation rate for the outside travel lane equals 6.0%, the adjacent shoulder will slope away from the travel lane at a rate of 1.0%. Where the superelevation rate for the outside travel lane equals 10.0%, the adjacent shoulder will slope toward the travel lane at a rate of 3.0%. Details of shoulder superelevation are illustrated for various points along the superelevation transition length in the *Roadway Design Standard Drawings*.

3-5.0 HORIZONTAL SIGHT DISTANCE

Sight obstructions on the inside of a horizontal curve are obstacles of considerable length which interfere with the line of sight on a continuous basis. These include walls, median barriers, bridge rails, cut slopes, wooded areas, buildings and high farm crops. In general, point obstacles such as traffic signs and utility poles are not considered sight obstructions. The designer must examine each curve individually to determine whether it is necessary to remove an obstruction or adjust the horizontal alignment to obtain the required sight distance.

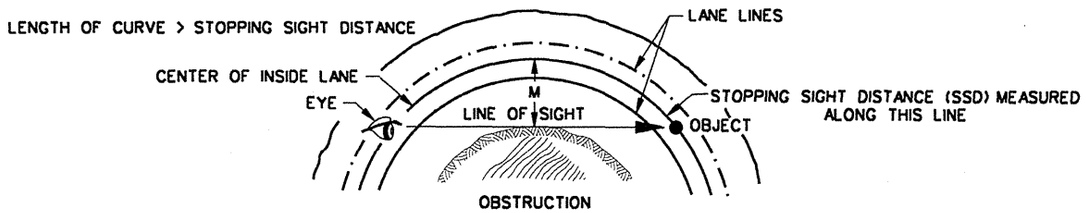
3-5.01 Middle Ordinate for Stopping Sight Distance

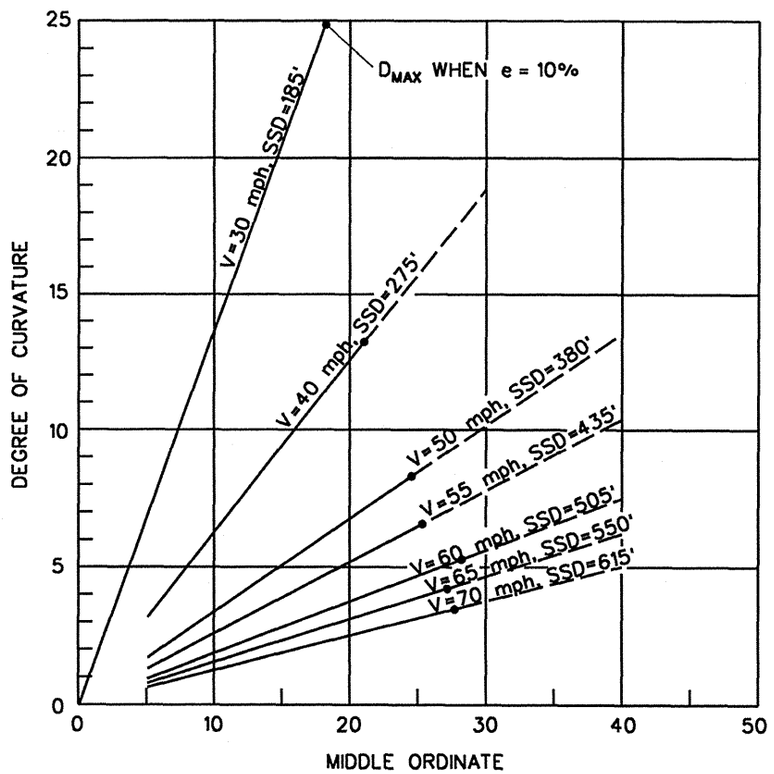
The designer must provide a "middle ordinate" (M) between the center of the inside travel lane and the sight obstruction. Figure 3-5A provides the criteria to determine the middle ordinate at various design speeds for minimum stopping sight distance values. See Table 2-7A for sight distance values. For application, the height of eye is 3.5 ft and the height of object is 6 in. The line-of-sight intercept with the obstruction is at the midpoint of the sight line and 2 ft above the center of the inside travel lane.

The M values from Figure 3-5A only apply to that portion of the curve between the PC and the PT. In addition, the figure does not apply to curves whose lengths are less than SSD. The following two sections describe how to address these conditions.

3-5.01.01 Length of Curve > Stopping Sight Distance

Where the length of curve (L) is greater than the stopping sight distance (SSD) used for design, the needed clearance on the inside of the horizontal curve is calculated as follows:





MIDDLE ORDINATE AT HORIZONTAL CURVES (Minimum SSD)

Figure 3-5A

$$M = \frac{5730}{D}\left(1 - \cos\frac{SSD(D)}{200}\right) \qquad \text{(Equation 3-5.1)}$$

Where:

M = Middle ordinate, or distance from the center of the inside travel lane to the obstruction, ft

D = Degree of curvature, ft

SSD = Stopping sight distance, ft

$\frac{(SSD)(D)}{200}$ = Angle, degrees

The M values from Equation 3-5.1 and Figure 3-5A apply between the PC and PT. In addition, some transition is needed in advance of and beyond the horizontal curve. The designer should use the following steps:

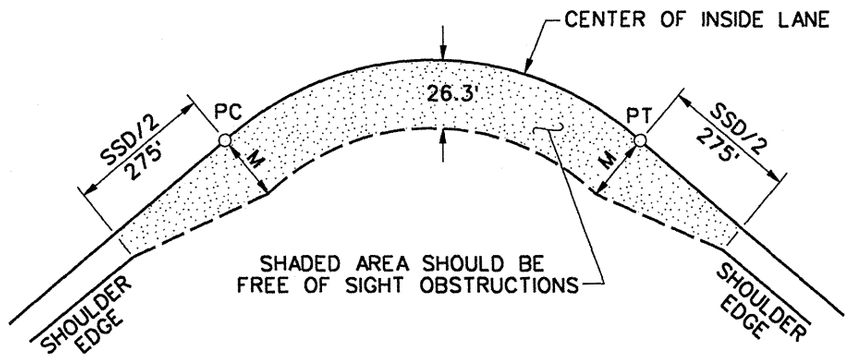
Step 1: Locate the point which is on the outside edge of shoulder and a distance of SSD/2 before the PC.

Step 2: Locate the point which is a distance M measured laterally from the center of the inside travel lane at the PC.

Step 3: Connect the two points located in Steps 1 and 2. The area between this line and the roadway should be clear of all continuous obstructions.

Step 4: A symmetrical application of Steps 1 through 3 should be used beyond the PT.

The Example on Figure 3-5B illustrates the determination of clearance requirements before and after a horizontal curve.



Example 3-5.1

Given: Design Speed = 65 mph
D = 4°

Problem: Determine the horizontal clearance requirements for the horizontal curve to meet the SSD.

Solution: Table 2-7A yields a SSD = 550 ft. Using Equation 3-5.1 for horizontal clearance (L > SSD):

$$M = \frac{5730}{D}\left(1 - \cos\frac{SSD(D)}{200}\right)$$

$$M = \frac{5730}{4}\left(1 - \cos\frac{(550)(4)}{200}\right) = 26.3 \text{ ft}$$

The above figure also illustrates the horizontal clearance requirements before and after the horizontal curve.

SIGHT CLEARANCE REQUIREMENTS FOR HORIZONTAL CURVES (L > SSD)

Figure 3-5B

3-5.01.02 Length of Curve < Stopping Sight Distance

Where the length of curve is less than the stopping sight distance used in design, the M value from Equation 3-5.1 will never be reached. As an approximation, the horizontal clearance for these curves should be determined as follows:

Step 1: For the given D and S, calculate M assuming L > SSD.

Step 2: The maximum M′ value will be needed at a point of L/2 beyond the PC. M′ is calculated from the following proportion:

$$\frac{M'}{M} = \frac{1.2\ L}{SSD}$$

$$M' = \frac{1.2\ (L)(M)}{SSD} \qquad \text{(Equation 3-5.2)}$$

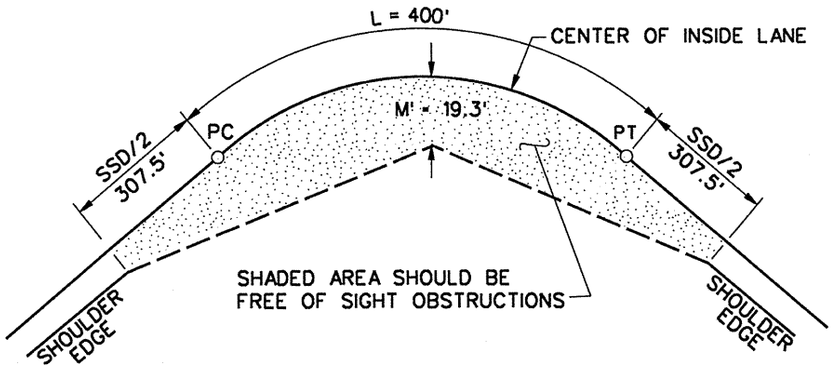
W
here: $M' \leq M$

Step 3: Locate the point which is on the outside edge of shoulder and a distance of SSD/2 before the PC.

Step 4: Connect the two points located in Steps 2 and 3. The area between this line and the roadway should be clear of all continuous obstructions.

Step 5: A symmetrical application of Steps 2 through 4 should be used on the exiting portion of curve.

The Example on Figure 3-5C illustrates the determination of the clearance requirements where L < SSD.



Example 3-5.2

Given: Design Speed = 70 mph
D = 3°
L = 400 ft

Problem: Determine the horizontal clearance requirements for the horizontal curve.

Solution: Figure 2-7A yields a SSD of 615 ft for 70 mph. Therefore, L < SSD (400 ft < 615 ft), and the horizontal clearance is calculated as follows:

$$M\ (L > SSD) = \frac{5730}{3}\left(1 - \cos\frac{(615)(3)}{200}\right) = 24.7 \text{ ft}$$

$$M'\ (L < SSD) = \frac{1.2\ (400)(24.7)}{615}$$

$$M' = 19.3 \text{ ft}$$

Therefore, a maximum clearance of 19.3 ft should be provided at a distance of L/2 = 200 ft beyond the PC

SIGHT CLEARANCE REQUIREMENTS FOR HORIZONTAL CURVES
(L < SSD)
Figure 3-5C

3-5.01.03 Longitudinal Barriers

Longitudinal barriers can cause sight distance problems. These include guardrail, median barriers and bridge rails. The problem results because barriers are placed relatively close to the traveled way (often, 10 ft or less) and because their height is greater than 2 ft. However, barrier location is determined by factors other than sight distance considerations, and there are usually no practical alternatives to their location. Therefore, where barriers interfere with the line of sight, it will not be necessary to seek a design exception.

3-5.02 Decision Sight Distance

Table 2-6A provides values for decision sight distance. If the designer chooses to use decision sight distance at a horizontal curve, the applicable values should be used in Equation 3-5.1 to determine the "clearance requirements" on a horizontal curve.

3-5.03 Passing Sight Distance

On high-volume, 2-lane roadways it may occasionally be warranted to provide passing sight distance to maintain capacity. Table 2-7A provides values for passing sight distance.

Generally, it is impractical to provide passing sight distance on horizontal curves. These values yield very large middle ordinates for the required clearance. In addition, many drivers will not pass on horizontal curves regardless of the available sight distance. Where passing sight distance is used, the designer should use Equation 3-5.1 to calculate the clearance requirements.

3-6.0 REFERENCES

1. AASHTO *A Policy on Geometric Design of Highways and Streets*, 1990, 1994.

2. *Transition Curves for Highways*, Barnett.

Chapter 4

Vertical Alignment

MISSISSIPPI DESIGN MANUAL

Chapter 4

VERTICAL ALIGNMENT

Section		Page
4-1.0	GENERAL CRITERIA	4-1
4-2.0	DEFINITIONS	4-2
4-3.0	GRADES	4-2
4-3.01	Maximum Grades	4-2
4-3.02	Minimum Grades	4-2
4-3.03	Critical Length of Grade	4-3
4-3.04	Climbing Lane	4-3
4-4.0	VERTICAL CURVES	4-6
4-4.01	Characteristics of Vertical Curves	4-6
4-4.02	Crest Vertical Curves	4-7
4-4.02.01	Stopping Sight Distance	4-7
4-4.02.02	Decision Sight Distance	4-7
4-4.02.03	Passing Sight Distance	4-7
4-4.03	Sag Vertical Curves	4-10
4-4.03.01	Headlight Sight Distance	4-10
4-4.03.02	Comfort Criteria	4-10
4-5.0	ESTABLISHING PROFILE GRADE	4-10
4-5.01	Grade Computations	4-10
4-5.02	Vertical Curve Computations	4-10
4-5.03	Recording Profile Grades	4-12
4-6.0	SPECIAL CONSIDERATIONS	4-12
4-6.01	Field Recommendations	4-12
4-6.02	Ties with Adjoining Projects	4-12
4-6.03	Balancing Earthwork	4-12
4-6.04	Drainage Structures	4-13
4-6.05	Major Structures	4-13
4-6.06	Railroad Grade Crossings	4-13
4-6.07	Adjacent Multilane Roadways	4-13

Section		Page
4-7.0	REFERENCES	4-14

Chapter 4

VERTICAL ALIGNMENT

Many of the important decisions regarding horizontal alignment are made during the planning process and the location survey. However, the vertical alignment is primarily established by the judgments and decisions of the designer which, in turn, affect the costs, aesthetics and safety characteristics of the facility.

4-1.0 GENERAL CRITERIA

Several general criteria should be considered in the design of vertical alignment:

1. The designer should provide smooth and gradual changes in a grade line that are consistent with both the highway type and the character of terrain. Grade lines with numerous breaks and short tangent lengths should be avoided.

2. "Roller coaster" or "hidden dip" profiles should be avoided. They are hazardous and aesthetically undesirable.

3. A "broken back" grade (i.e., two crest or two sag vertical curves separated by a short tangent) should be avoided. One long vertical curve is more desirable.

4. On long grades, it is preferable to place the steepest grade at the bottom and flatten the grade near the top.

5. Moderate to steep grades should be reduced through intersections. This benefits turning vehicles and minimizes the hazard potential.

6. Vertical sight distance restrictions will reduce passing opportunities. On high-volume, 2-lane roads, the designer should consider maximizing the passing opportunities to ensure adequate capacity. In extreme cases, it may be necessary to introduce passing lanes.

7. The terrain designation has a significant impact on several of the geometric design elements (e.g., maximum allowable grades and design speed). The following general definitions apply:

   a. Level. Highway sight distances are either long or could be made long without major construction expense.

   b. Rolling. The natural slopes consistently rise above and fall below the roadway grade. Occasionally, steep slopes present some restriction to the desirable highway alignment.

   c. Mountainous. Longitudinal and transverse changes in elevation are abrupt. Benching and side hill excavation are frequently required to provide a desirable highway alignment.

   For design applications, Mississippi's terrain will be considered either level or rolling.

8. Care should be taken to avoid placement of intersecting roadways at or near the VPC or VPT of crest vertical curves.

9. Placement of sag vertical curves in cut sections should be avoided unless adequate drainage can be provided.

4-2.0 DEFINITIONS

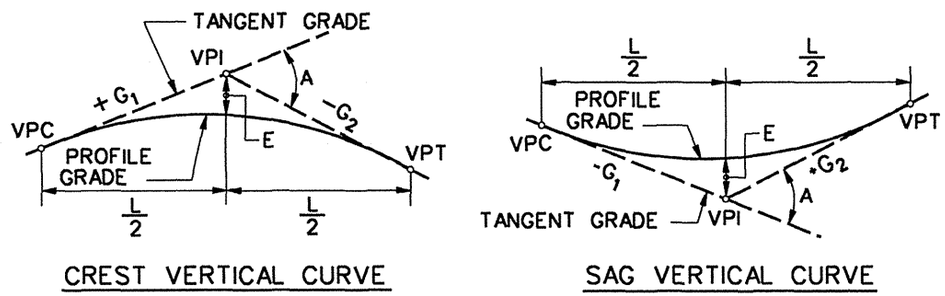
Figure 4-2A illustrates the various elements of vertical alignment. The following defines each of the elements shown in Figure 4-2A:

1. Vertical Point of Curvature (VPC). The point at which the tangent grade ends and the vertical curve begins.

2. Vertical Point of Tangency (VPT). The point at which the vertical curve ends and the tangent grade begins.

3. Vertical Point of Intersection (VPI). The point where the extension of two tangent grades intersect.

4. Grade Slopes (G_1 or G_2). The rate of slope between two adjacent VPI's is expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in feet for each 100 ft of horizontal distance.



SCHEMATICS OF VERTICAL CURVE

Figure 4-2A

5. Sign Convention. Upgrades in the direction of stationing are identified as plus (+). Downgrades are identified as minus (-).

6. External Distance (E). The vertical distance between the VPI and the roadway surface along the vertical curve.

7. Algebraic Difference (A). The value of A is determined by the deflection, in percent, between the two tangent grades.

8. Length of Vertical Curve (L). The horizontal distance, in feet, from the VPC to the VPT.

4-3.0 GRADES

The vertical alignment of a highway is referenced to a profile grade line that consists of a series of tangents which are connected by vertical curves.

4-3.01 Maximum Grades

Chapters 2, 11, 13 and 14 present the Department's criteria for maximum grades based on functional classification, urban/rural location, project scope of work, type of terrain and design speed. The maximum grades should be used only where absolutely necessary. Normally, grades much flatter than the maximum should be used.

4-3.02 Minimum Grades

On highways and streets with curb and gutter, the minimum longitudinal gradient should be 0.4%. This is necessary to facilitate longitudinal drainage.

On highways without curb and gutter, level gradients are acceptable on pavements which are adequately crowned to drain laterally. However, it is desirable to provide a 0.4% minimum longitudinal grade because the original lateral crown slope may degrade as a result of swell, consolidation, maintenance operations or resurfacing.

For adequate drainage, roadside ditches and median swales require a steeper gradient than that required for roadway surfaces. This is discussed further in Chapter 7 "Drainage."

4-3.03 Critical Length of Grade

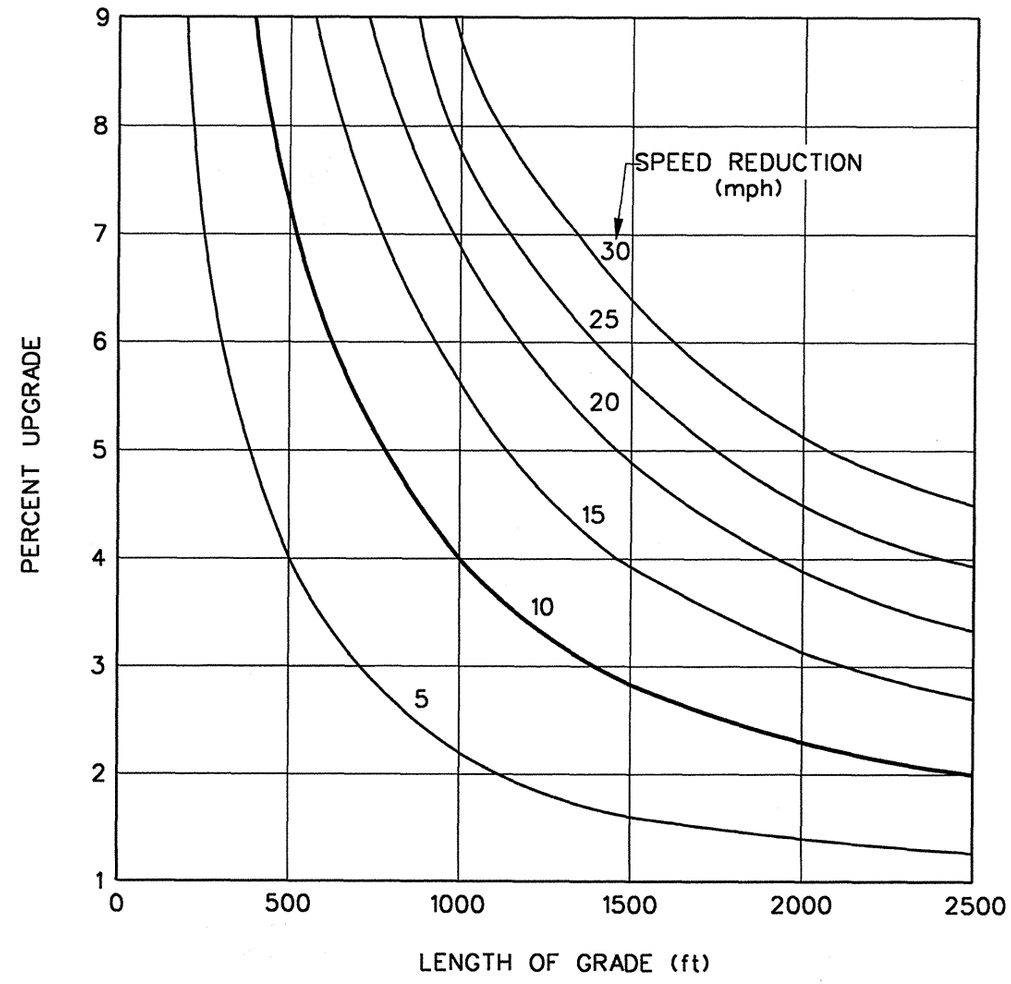
The upgrade gradient in combination with its full length will determine the resulting truck speed reduction. Figure 4-3A provides the critical length of grade for a given percent grade and an acceptable truck speed reduction. This figure should be used for 2-lane roadways, and it is applicable to any design speed. For design, the 10-mph speed reduction curve should be used. If this value is exceeded, the grade should be flattened, or the designer should evaluate the need for a climbing lane (Section 4-3.04).

Examples 1 and 2 illustrate how to use Figure 4-3A to determine the critical length of grade.

4-3.04 Climbing Lane

Climbing lanes offer a comparatively inexpensive means of increasing capacity on highly traveled upgrades where congestion is caused by slow trucks. A 2-lane highway with climbing lanes may be a more cost-effective solution than providing a multilane highway section.

The warrants for a climbing lane should be used as follows:



CRITICAL LENGTH OF GRADE

Figure 4-3A

Example 1

Given: $G_1 = 0\%$
$G_2 = +4\%$
$L_2 = 1200$ ft (VPI to VPI)

Problem: Determine if the critical length of grade is exceeded.

Solution: Figure 4-3A yields a critical length of grade of 1000 ft for a 10-mph speed reduction. The length of grade (L_2) exceeds this value, and the designer should flatten the grade, if practical, or evaluate the need for a climbing lane.

Example 2

Given: $G_1 = 0\%$
$G_2 = +5\%$
$L_2 = 400$ ft (VPI to VPI)
$G_3 = +2\%$
$L_3 = 600$ ft (VPI to VPI)

Problem: Determine if the critical length of grade is exceeded for the combination of grades G_2 and G_3.

Solution: Using Figure 4-3A, G_2 yields a truck speed reduction of approximately 5 mph. G_3 yields approximately 3 mph. The total reduction in speed of 8 mph is less than the allowable 10 mph. Therefore, the critical length of grade is not exceeded.

1. Critical Length of Grade. Figure 4-3A provides the criteria for critical length of grade. If this value is exceeded, the designer should determine if a momentum grade can be assumed.

2. Momentum Grade. Where an upgrade is preceded by a momentum grade (i.e., a downgrade), trucks often increase speed to make the climb. Speed increases of 5 mph on moderate downgrades (3%-5%) and 10 mph on steeper grades (6%-8%) of sufficient length are reasonable adjustments. These values can be applied to Figure 4-3A to allow the use of a higher speed reduction curve. If the critical length of grade is still exceeded, the designer should conduct a capacity analysis.

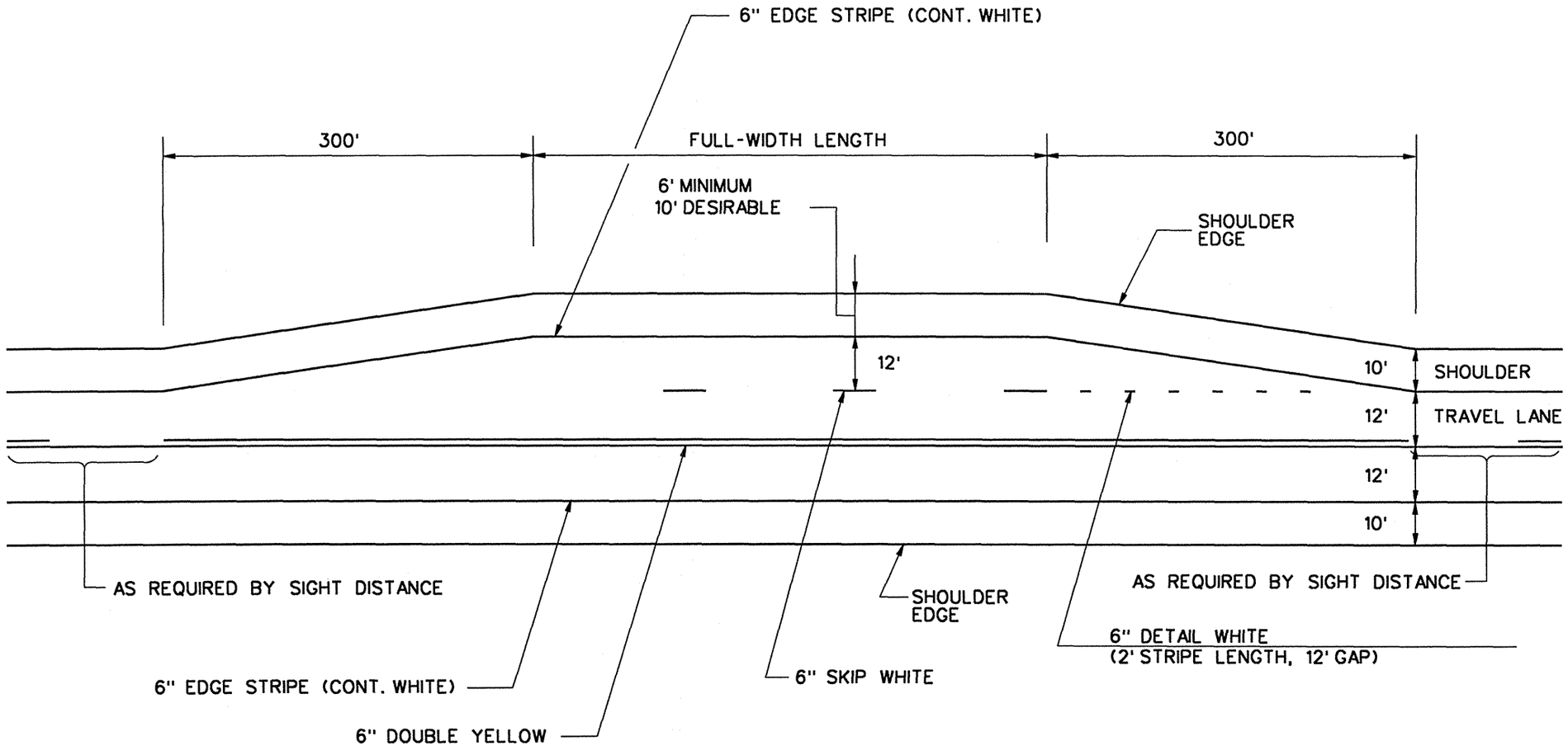
3. Capacity. The *Highway Capacity Manual* (HCM) presents the detailed methodologies to conduct a capacity analysis on a grade.

When conducting a capacity analysis, the designer must first select a desired level-of-service (LOS). The functional classification tables in Chapters 2, 11, 13 and 14 present LOS criteria for each highway class and urban/rural location. If the capacity analysis determines that the grade will operate below the desired LOS for the DHV, then the designer should consider providing a climbing lane if the construction costs and associated impacts are reasonable. If the anticipated LOS is less than "D," then a climbing lane is almost always warranted.

Figure 4-3B illustrates the design and pavement markings of a typical climbing lane. If a climbing lane is warranted, the following criteria should be used in design:

1. Lane widths should be 12 ft wide.

2. The shoulder width should be the same as the width of a normal section, but not less than 6 ft.



TYPICAL CLIMBING LANE

Figure 4-3B

3. The cross slope of the climbing lane will be the same as the adjacent travel lane.

4. The full width of the climbing lane should be attained where the truck speed reduction equals 10 mph or less.

5. The full width of the climbing lane should be extended to at least the point where the truck speed is within 10 mph of the highway design speed.

6. Both the entering and exiting taper of the climbing lane should be 300 ft.

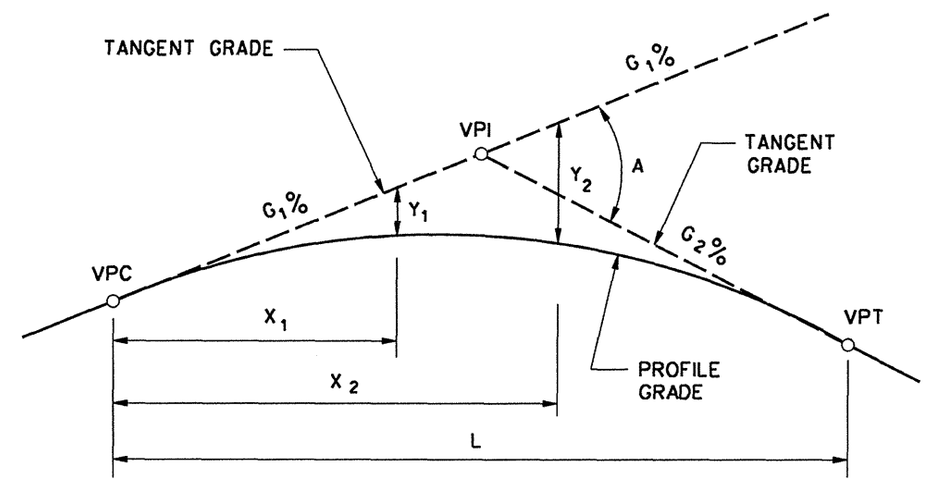
4-4.0 VERTICAL CURVES

4-4.01 Characteristics of Vertical Curves

Vertical curves have the properties of a simple parabolic curve. The vertical offset from the tangent at any point varies as the square of the horizontal distance from the end of the curve. Figure 4-4A illustrates a symmetrical vertical curve. Curves which are offset below the tangents are called crest vertical curves; those which are offset above the tangents are sag vertical curves.

The rate of change of grade per foot (R) along a vertical curve is constant. It is measured by dividing the algebraic difference between the grades (A) in percent by the length of the curve (L) in feet (i.e., A/L). This value gives the percent change in grade per horizontal foot of curve.

The reciprocal of this value (i.e., L/A) represents the horizontal distance required to produce a 1% change in the gradient along the curve. The expression L/A is designated K and is useful for determining minimum lengths of vertical curves. Based on the geometrics of each sight distance condition, formulas are used to compute values of K for each design speed. The minimum length of vertical curve is:



The computation of an offset at any given point on the vertical curve is determined by:

$$Y = RX^2 / 200$$

where:

Y	=	the vertical offset to the tangent grade from any given point on the vertical curve (feet)
X	=	horizontal distance from VPC to any given point on the vertical curve (feet)
R	=	$A / L = [(G_2 - G_1) / L]$
	=	the rate of change of grade along the vertical curve (% per foot)
G_2	=	the tangent grade beyond the VPI (%)
G_1	=	the tangent grade before the VPI (%)
A	=	$(G_2 - G_1)$ = the algebraic difference between tangent grades (%)
L	=	the total length of vertical curve (feet)

The computation of the horizontal distance to the high point (crest) or low point (sag) of a vertical curve is determined by:

$$X_{high/low} = [LG_1 / (G_1 - G_2)]$$

SYMMETRICAL VERTICAL CURVE COMPUTATION

Figure 4-4A

$$L = KA$$

where:

L = the minimum length of vertical curve (feet)
A = the algebraic difference between tangent grades (%)
K = a constant value for the design speed

Values of K are presented in the following sections for crest and sag vertical curves.

4-4.02 Crest Vertical Curves

The minimum length of a crest vertical curve, regardless of sight distance is $L_{min} = 4V$, where V equals the design speed in miles per hour.

4-4.02.01 Stopping Sight Distance

A driver should be able to see an object on the road in time to stop. The line of sight should not be obstructed by the crest of a vertical curve. A 3.5-ft height-of-eye and a 6-in. height-of-object are used in the equation for crest vertical curves. Table 4-4A and Figure 4-4B provide the criteria to determine the length of a crest vertical curve at various design speeds and minimum stopping sight distance values.

4-4.02.02 Decision Sight Distance

Table 2-6A presents the values for decision sight distances. These distances provide the driver with more time to detect a hazard and decide what course of action to take. At some highway locations, a crest vertical curve may obstruct the line-of-sight to an obstacle which may be unexpected or difficult to perceive. For example, an exit gore from a freeway may be located just beyond a crest vertical curve.

Once the determination has been made to use decision sight distance as a design control, the applicable values should be used in the equation for crest vertical curves to yield the necessary curve length at the site.

4-4.02.03 Passing Sight Distance

On 2-lane highways, the use of passing sight distance as a design control for crest vertical curves may be warranted. Table 4-4B and Figure 4-4C provide the criteria to determine the length of a crest vertical curve at various design speeds and minimum passing sight distance values. The height of object is 4.25 ft.

Generally, it is impractical to design crest vertical curves to provide for passing sight distance because of the high cost and the difficulty of fitting the required long vertical curves to the terrain. In addition, drivers may be reluctant to pass on vertical curves even where sufficient sight distance is available. Ordinarily, passing sight distance will be provided only at those locations where combinations of horizontal alignment and vertical profile do not require the use of crest vertical curves.

The Department's criteria for striping no-passing zones is presented in Section 9-9.01. These criteria differ from the passing sight distance criteria and are based on different operating assumptions.

Table 4-4A

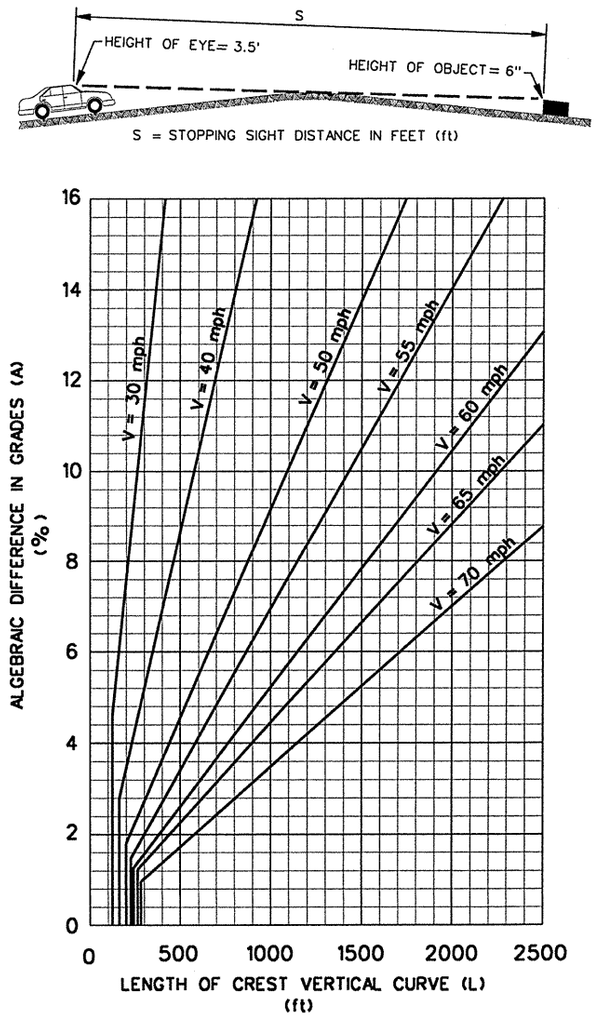
K-VALUES FOR CREST VERTICAL CURVES
(Minimum SSD Values)

Design Speed (mph)	Minimum SSD Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/1329$
30	185	26
40	275	57
50	380	109
55	435	143
60	505	192
65	550	228
70	615	285

$$L = \frac{AS^2}{1329} = KA$$

where:

S = the sight distance for the design speed (feet)
L = the length of vertical curve (feet)
A = the algebraic difference between grades (%)
K = the horizontal distance required to produce a 1% change in gradient



LENGTH OF CREST VERTICAL CURVE
(Minimum SSD Values)

Figure 4-4B

Table 4-4B

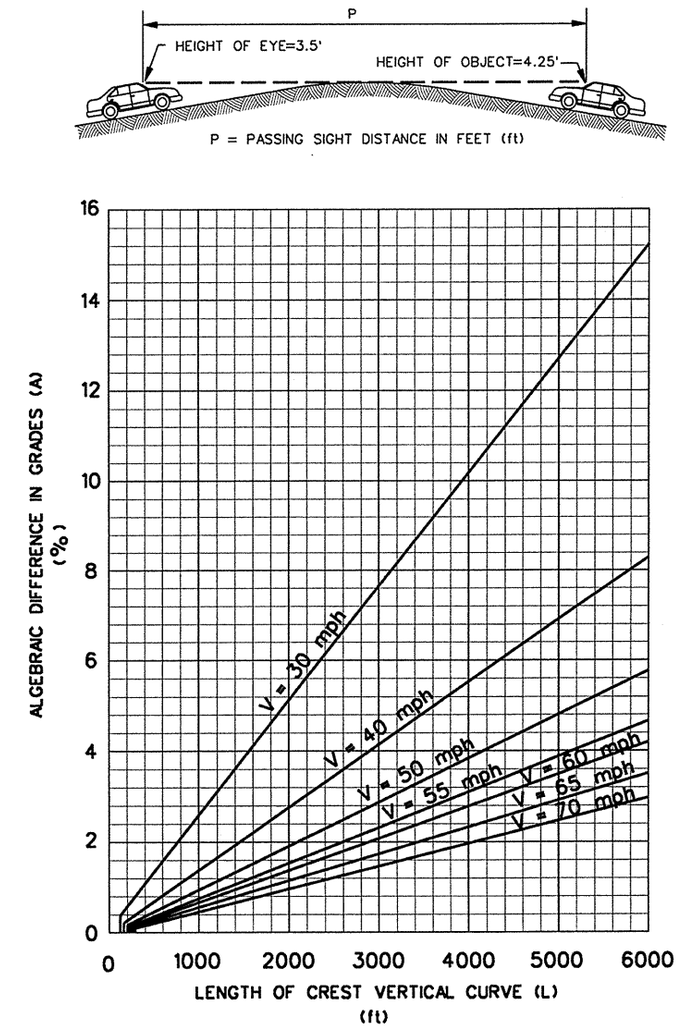
K-VALUES FOR CREST VERTICAL CURVES
(Passing Sight Distance)

Design Speed (mph)	Passing Sight Distance Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/3093$
30	1100	392
40	1500	728
50	1800	1048
55	2000	1294
60	2100	1426
65	2300	1711
70	2500	2021

$$L = \frac{AS^2}{3093} = KA$$

where:

S = the sight distance for the design speed (feet)
L = the length of vertical curve (feet)
A = the algebraic difference between grades (%)
K = the horizontal distance required to produce a 1% change in gradient



LENGTH OF CREST VERTICAL CURVE
(Passing Sight Distance)

Figure 4-4C

4-4.03 Sag Vertical Curves

The minimum length of a sag vertical curve, regardless of sight distance requirements, is $L_{min} = 3V$, where V equals the design speed in miles per hour.

4-4.03.01 Headlight Sight Distance

Nighttime driving conditions govern for sag vertical curves. The designated sight distance should be illuminated by the headlight beam with an assumed upward divergence of 1°. The height of the headlights is 2 ft and the height of object is 0 in. (i.e., the pavement surface). Table 4-4C and Figure 4-4D provide the criteria to determine the length of a sag vertical curve at various design speeds and minimum stopping sight distance values.

4-4.03.02 Comfort Criteria

In lighted areas on urban streets, the comfort criteria may be used for sag vertical curves. This is discussed in Chapter 14.

4-5.0 ESTABLISHING PROFILE GRADE

The profile grade normally will represent the finished surface elevation at the centerline of the roadway or at the median edge of the pavement for divided highways with a raised median. Profile grades may be computed manually or by a computer. Manual computation procedures are described in the following sections to ensure understanding of the concepts.

4-5.01 Grade Computations

Initial efforts for establishing the profile grade are recorded on a working profile. Using recommendations from the field and adhering to criteria in this *Manual*, the designer plots a tentative grade line on the working profile with the stationing and approximate elevation of each VPI clearly defined.

Starting at the beginning of the project, the designer should compute a grade which, when extended forward, will check reasonably close to the plotted elevation of the first VPI. The rate of grade (in percent) is computed by dividing the vertical difference by the horizontal distance and multiplying by 100. If practical, the percent grade should be computed and recorded to two decimal places.

For ease of computation, it is desirable to locate VPI's at multiples of 25 ft within the stationing (e.g., 5 + 75). VPI elevations should always be recorded to the nearest 0.01 ft.

After the first grade and VPI elevation have been established, the process is repeated between subsequent VPI's. Each grade and VPI elevation is recorded on the working profile.

Frequently, it is necessary to adjust a grade line after the initial grade computations have been made. When making this adjustment, it may be necessary to compute the new grade to five decimal places to accurately check-in to a previously established VPI elevation.

4-5.02 Vertical Curve Computations

After the tangent grades have been established, appropriate vertical curves must be designed for each VPI. The designer should comply with the criteria for sight distance and minimum curvature.

Table 4-4C

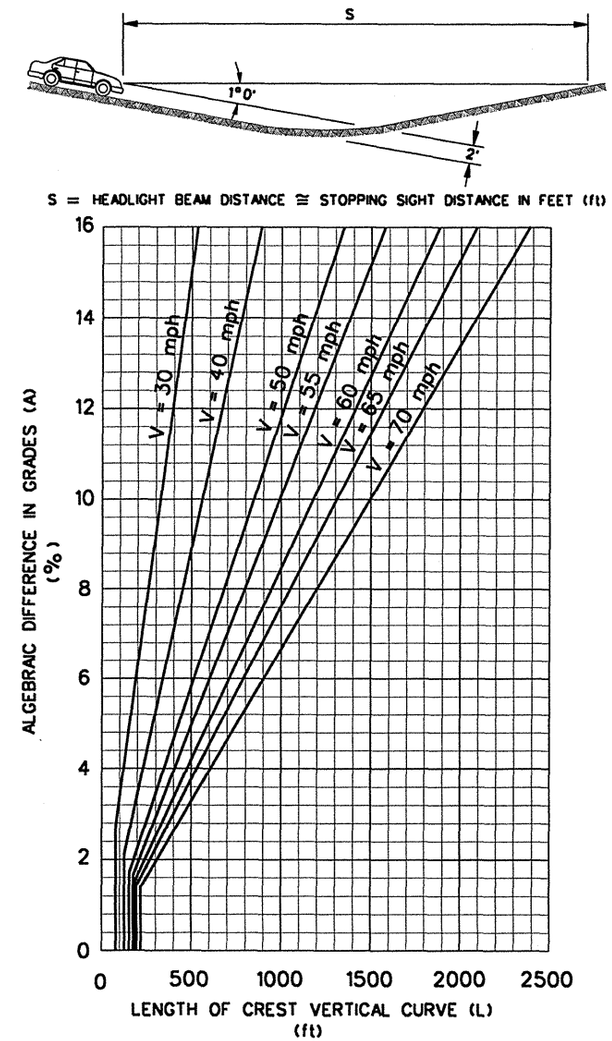
K-VALUES FOR SAG VERTICAL CURVES
(Minimum SSD Values)

Design Speed (mph)	Minimum SSD Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/(400 + 3.5S)$
30	185	33
40	275	56
50	380	84
55	435	99
60	505	118
65	550	131
70	615	149

$$L = \frac{AS^2}{(400 + 3.5S)} = KA$$

where:

S = the sight distance for the design speed (feet)
L = the length of vertical curve (feet)
A = the algebraic difference between grades (%)
K = the horizontal distance required to produce a 1% change in gradient



LENGTH OF SAG VERTICAL CURVE
(Minimum SSD Values)

Figure 4-4D

Vertical curve profile grade elevations are established by computing vertical offset distances between the tangent grade and the vertical curve at various points along the stationing. For crest vertical curves, these offset distances are subtracted from the tangent grade elevations; for sag vertical curves, the computed offsets are added to the tangent elevation.

Figure 4-4A illustrates a crest vertical curve; however, the formulas also are applicable to a sag vertical curve. It is important to recognize the plus or minus characteristics of the grades when computing algebraic differences and whether the offset distances will be added to or subtracted from the tangent elevations.

4-5.03 Recording Profile Grades

The established profile grade elevations should be recorded to the nearest 0.01 ft on the cross-section sheets and along the bottom of the plan profile sheets. In addition, the profile sheet should show the stationing and elevation at each VPC, VPI and VPT, the gradients between VPI's and the length of each vertical curve.

4-6.0 SPECIAL CONSIDERATIONS

4-6.01 Field Recommendations

Recommendations for the design of profile grades may accompany the field survey data. Special considerations may be required for items such as high-water levels, ditch flow lines, unusual soil conditions, road intersections, adjacent property development or other unusual conditions which may control profile grade elevations.

Profile grades should be designed to ensure adequate elevation above high-water levels for the selected design storm frequency (see Section 7-3.0). Freeboard clearance above the high-water level for the design storm frequency should be at least 4 ft for Interstate highways and 3 ft for all other roads. Also, the design storm frequency for the through lanes of an Interstate highway is 50 years.

4-6.02 Ties with Adjoining Projects

A smooth transition is needed between adjoining projects. Existing grade lines should be considered for a sufficient distance beyond the beginning and ending stations of a project to ensure adequate sight distance. Connections should be made which are compatible with the design speed of the new project and which can be used when the adjoining road section is reconstructed.

4-6.03 Balancing Earthwork

Unless there are circumstances which preclude this objective, the profile grade should be designed to achieve a balance of earthwork. Excavation from the normal cut sections should be adequate to construct the fills without a need for borrow material. Wasting excess material should also be minimized. Wherever practical, earthwork should be balanced at bridge ends, railroad crossings and major highway intersections.

Normally, balance points should not exceed a distance of approximately 3 miles. A reasonable earthwork balance usually can be attained by minor adjustments in the grade line, such as:

1. raising or lowering a tangent grade between two VPI's;
2. raising or lowering a VPI and adjusting the gradients to adjacent VPI's; or
3. lengthening or shortening a vertical curve.

When the excess or deficiency of excavation is known, the required amount of grade adjustment can be estimated reasonably close on the basis of average cross-section widths (outside limits of cuts and fills) and the length of grade that will be impacted.

4-6.04 Drainage Structures

Where there are special controls impacting drainage flow lines, consideration must be given to a profile grade design that will ensure at least minimum cover over drainage structures. Criteria for minimum cover are discussed in Chapter 7 "Drainage."

4-6.05 Major Structures

The design of profile grades must be carefully coordinated with bridge design and with the vertical clearance and high-water controls. The designer must adhere to the vertical clearances presented in Chapters 2, 11, 13 and 14.

Following the completion of the bridge design, there should be verification that the required clearance actually will be provided and that the grade controls shown on the bridge plans and on the road plans are identical.

4-6.06 Railroad Grade Crossings

The finished surface grade line must match the top of the rails at railroad grade crossings. In most instances, minor warping of the surface cross slopes will be required.

4-6.07 Adjacent Multilane Roadways

Normally, a common profile grade line is used for both roadways of divided highways where the median width is 64 ft or less.

For median widths in excess of 64 ft, independent profile grade lines may be desirable because of the natural terrain. However, an appreciable grade differential between the roadways should be avoided at crossovers for at-grade intersections. Traffic entering from the crossroad may make a wrong-way movement if the pavement of the far roadway is obscured because of grade differential. Required safety slopes in the median will place further restrictions on the grade differential.

This problem is further complicated when a divided highway will be provided by using the existing two lanes for one direction of travel and constructing two new lanes for the other direction. For safety, it is desirable to have approximately the same elevation on both roadways at crossroads and crossovers. However, in many instances, the existing two lanes may have poor vertical alignment, and attempts to match the elevations would sacrifice the desirable vertical alignment of the two new lanes. The location of crossovers may be adjusted slightly to improve the safety characteristics, but usually it is difficult to relocate a crossroad.

As a general rule in these situations, the grade of the crossover connections should not exceed 6%. Even without a crossover connection, large differences in elevation can cause problems in providing proper safety slopes. Table 4-6A suggests the maximum elevation differential between adjacent roadways for various horizontal distances between centerlines.

Profile grades at or near the maximum differential should be avoided where practical. Consideration should be given to reconstructing portions of the existing roadway at such locations.

Table 4-6A

ELEVATION DIFFERENTIAL BETWEEN ADJACENT ROADWAYS

Horizontal Distance ℄ to ℄ (ft)	Maximum Profile Grade Differential (ft)	
	With Crossover	Without Crossover
64	1.0	1.0
88	2.0	2.0
125	4.2	11.3
150	5.7	17.5
200	8.7	30.0
250	11.7	42.5

4-7.0 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 1990, 1994.

2. *Highway Capacity Manual*, Special Report No. 209, Transportation Research Board.

Chapter 5

Cross Sections and Earthwork

MISSISSIPPI DESIGN MANUAL

Chapter 5

CROSS SECTIONS AND EARTHWORK

Section		Page
5-1.0	PLOTTING THE EXISTING GROUND LINE	5-1
5-1.01	Field Survey Notes	5-1
5-1.02	Preparation for Computer Plotting	5-2
5-1.03	Preparation for Manual Plotting	5-3
5-1.04	Scales	5-3
5-1.05	Sheet Layout	5-3
5-1.06	Plotting	5-4
5-1.07	Sheet Identification	5-4
5-2.0	PLOTTING THE ROADWAY TEMPLATE	5-6
5-2.01	Typical Section Controls	5-6
5-2.02	Elevation Controls	5-6
5-2.03	Plotting	5-6
5-2.04	Drainage Structures	5-7
5-3.0	EARTHWORK COMPUTATION	5-7
5-3.01	Computer Computations	5-7
5-3.02	Manual Computations	5-8
5-3.02.01	Coordinate Method for End Areas	5-8
5-3.02.02	Strip Stick Method for Volumes	5-9
5-3.03	Balance Factor	5-9
5-3.04	Earthwork Calculations	5-10

Chapter 5

CROSS SECTIONS AND EARTHWORK

In combination with the horizontal alignment and vertical alignment, the cross section forms the third dimension of the highway. Cross-sectional views taken at right angles to the centerline present a visual relationship between the existing ground line and the typical section of the highway to be constructed. Cross sections also serve as a means for computing the earthwork quantities and designing the drainage. This chapter discusses guidelines for:

1. plotting the existing ground line;
2. plotting the roadway templates; and
3. earthwork computations.

5-1.0 PLOTTING THE EXISTING GROUND LINE

Before the detailed design of a highway can begin, the basic cross section sheets providing the shape and elevation of the existing ground lines must be prepared. Cross section notes from the field survey are used for this purpose. Cross sections are normally taken at 100-ft intervals and at other locations required by abrupt changes in terrain. Cross sections may be plotted manually or by the computer using the Roadway Design Software package.

5-1.01 Field Survey Notes

Typical field survey cross section notes are illustrated in Figure 5-1A. Complete survey information is necessary to assure accurate representation of the existing field conditions and, therefore, an accurate design. The following information is provided in Figure 5-1A:

1. The elevation control is from bench mark #1, a railroad spike in the base of a 10-in pine tree, 100 feet right of station 174+25. The bench mark elevation is 311.171 ft.

2. The first column provides the stationing of the cross sections and identifies bench marks (BM) and intermediate survey turning points (TP).

3. The second column provides backsight rod readings from an instrument set up to a BM or a TP.

4. The third column provides the height of instrument (HI) computed by adding the backsight rod reading to the established elevation for the BM or TP.

5. The fourth column provides the foresight rod readings from the HI to a subsequent BM or TP.

6. The fifth column provides the actual elevation of the TP computed by subtracting the foresight from the HI elevation.

7. The sixth column provides the BM elevation and notes to identify special features.

DATE 7-10-97 ROD MORRIS WEATHER FAIR I.D.
PICKERING ROD SPELL TYPE SECTION PRELIMINARY COL. 24
NOTES REEVES TAPE BURKS

B.M.	+	H.I.	−	ELEV.	DESCRIPTION
BM #1	4.18	315.35		311.17	R.R. SPIKE IN BASE 10-IN. PINE 100 FT. RT. STA. 174+25
T.P.	2.79	309.93	8.21	307.14	

25																	
26									0								
25	181+72					7.2	7.8	8.5	300.6 9.3	9.9	10.5	11.3					
26						150	100	50	0	50	100	150					
25	182+00			0.3	(+2.2)	(+2.1)	(+0.8)	4.9	8.7	301.0 8.9	8.9	9.5	10.5	11.3			
26				150	80	40	27	17	6	0	17	50	100	150			
25	183+00					3.0	1.6	2.4	303.6 6.3	7.7	4.2	10.1					
26						150	117	13	0	6	26	150					
25	184+00					5.2	3.8	4.3	302.2 7.7	8.1	6.8	7.3					
26						150	130	26	0	20	114	150					
25	FLOW LINE +79						4.0	6.7	F.L. OF DITCH 300.4 9.5	11.9	14.0						
26							150	75	0	75	150						
T.P.		6.22	312.38	3.77	306.16												
25	185+00					7.1	7.2	7.2	305.12 7.26	7.3	7.4	7.4					
26						150	100	50	0	50	100	150					

E.P. ①

Notes: ① *Edge of Pavement*

TYPICAL FIELD SURVEY NOTES FOR CROSS SECTIONS

Figure 5-1A

8. The cross section measurements are provided in the double rows adjacent to the station number. The bottom figures of each double row are the horizontal distances left and right of the centerline, and the top figures are the rod readings at those points. The actual elevation of the centerline is usually written above the rod reading.

The actual ground elevation at each point is calculated by subtracting the rod reading from the HI elevation. For example, the centerline elevation at station 181 + 72 is 300.6 (309.93-9.3) and 150 feet left of centerline, the elevation is 302.7 ft (309.93-7.2).

The accuracy required for the computed height of instrument (HI) shall be to the nearest 0.01 ft, and ground elevations shall be rounded to the nearest 0.1 ft. The elevation of bridges, the elevation of the existing roadway pavement, the flow line elevation of drainage structures, etc. shall be recorded to the nearest 0.01 ft.

5-1.02 Preparation for Computer Plotting

Most often, it is desirable to use computer plotting of cross sections. Cross section data is entered directly into the computer from the field survey notes. The computer will convert the recorded rod readings to ground line elevations which are then plotted. Before submitting the field survey notes to the computer section, the designer should:

1. Check the accuracy of all HI elevations.

2. Repeat all HI elevations with a red pencil in the third column at the top of each page.

3. Circle with a red pencil and mark "omit" or "do not punch" any data which should not be plotted or included for earthwork calculations.

4. Identify with a red pencil the beginning and ending limits.

5-1.03 Preparation for Manual Plotting

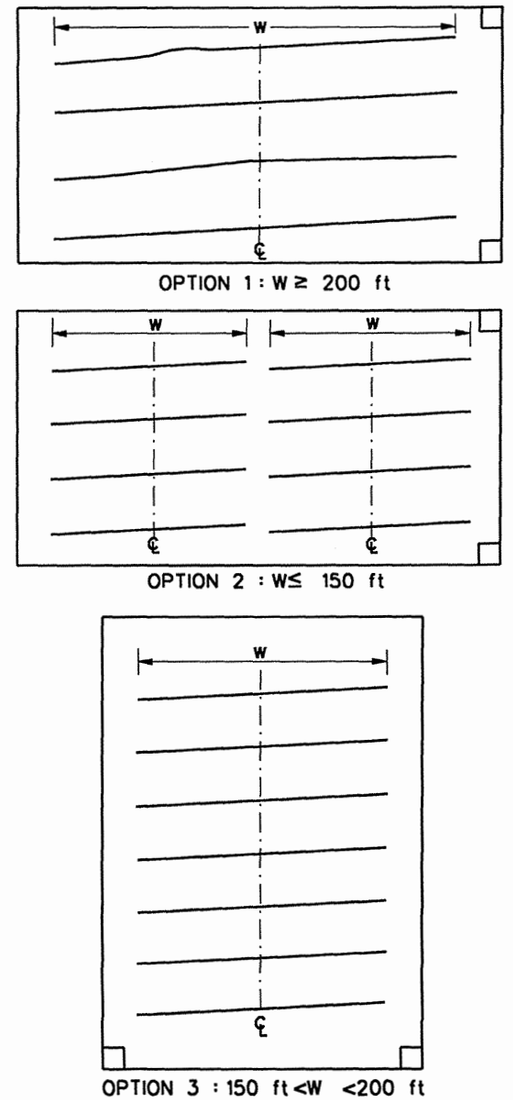
Manual plotting may be more convenient than computer plotting for special conditions. When cross sections are to be plotted manually, field survey data should be adjusted to facilitate the plottings. Each rod reading in the field survey notes is subtracted from (or, when a plus rod reading is used, added to) the applicable HI. The actual ground line elevations are recorded in the notes immediately above the corresponding rod reading. The designer should check the accuracy of all HI elevations and BM ties before computing the ground elevation at each point.

5-1.04 Scales

Cross sections will normally be plotted to a scale of 1 inch = 10 feet, both horizontally and vertically. In urban areas involving greater detail of design, the horizontal and vertical scales should be 1 inch = 5 feet. Other scales may be used in special situations. Any scale other than 1 inch = 10 feet should be provided below the project number block in the upper right corner of each cross section sheet.

5-1.05 Sheet Layout

The layout of a cross section sheet depends on the width of cross sections to be plotted and the horizontal scale selected. Figure 5-1B illustrates various options for cross section sheet layout and the total width of cross section, W. These layout options provide the best utilization of space on a sheet and, therefore, keep the total number of cross section sheets to a minimum.



CROSS SECTION SHEET LAYOUT

Figure 5-1B

5-1.06 Plotting

Figure 5-1C illustrates a typical cross section sheet with plotted sections of the existing ground line. Guidelines for the designer to follow in the plotting of a cross section sheet are listed below:

1. Cross sections should be plotted so that the survey stationing increases from the bottom to the top of each sheet.

2. Initial plotting should be with a sharp, hard pencil, and all plotted information should then be inked.

3. A zero (0) is placed on the bottom border of each sheet to identify the vertical line selected for the centerline of the cross sections.

4. Horizontal distances to the left and right of the centerline should be provided on the bottom border of the sheet at 10-ft intervals.

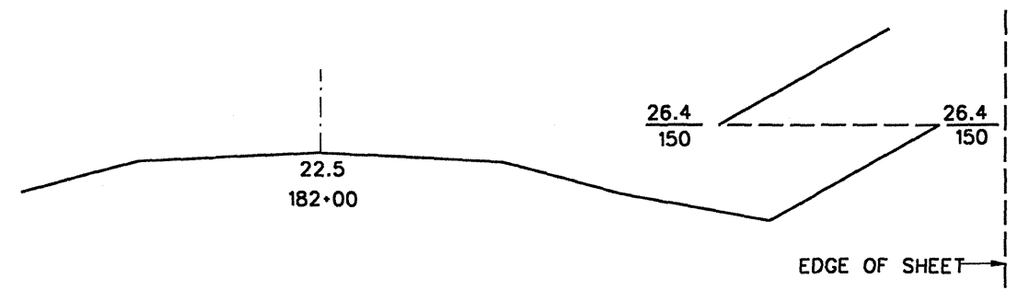
5. The accuracy required for the existing ground elevation at the centerline should be to the nearest 0.1 ft (0.01 ft in the case of existing pavement) and recorded immediately below the plotted cross section. The stationing of each cross section is recorded in larger size numerals directly below the centerline elevation.

6. The vertical spacing between plotted cross sections should be sufficient to avoid overlapping. The spacing will depend on the vertical scale used and on the height of fills or depth of cuts which must eventually be plotted on the cross section. Designers should refer to the working profile while plotting cross sections so that deep cuts or high fills can be anticipated and appropriate spacing may be provided between sections.

7. When an occasional cross section extends horizontally beyond the edge of the sheet, the slope line may be continued by an offset match line as illustrated in Figure 5-1D.

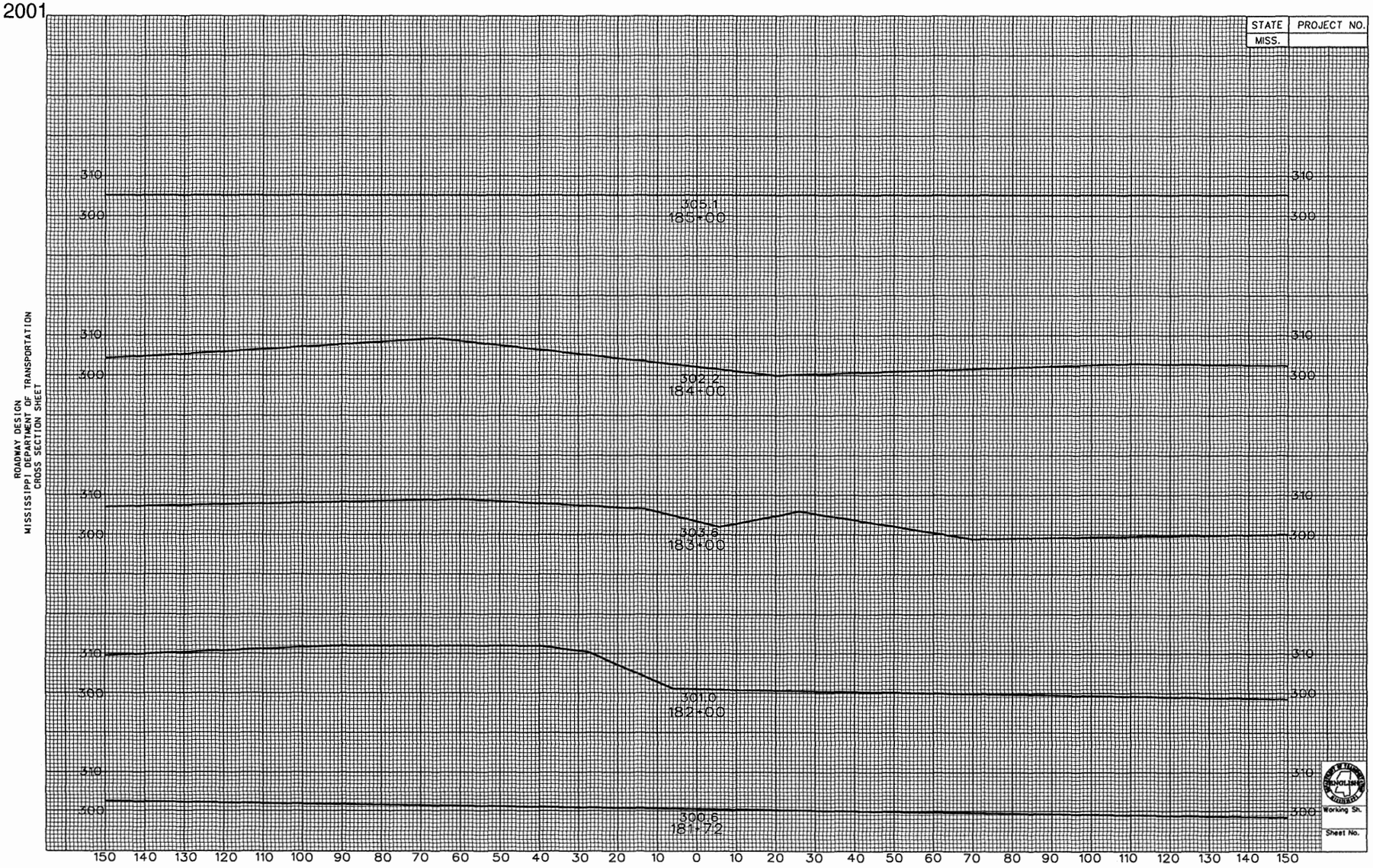
5-1.07 Sheet Identification

Each cross section sheet should be identified with the project number in the space provided in the upper right corner and with a sheet number in the space provided in the lower right corner of the sheet. Sequential sheet numbers, beginning with 901, should be assigned as cross section sheets are plotted. The entire mainline cross sections are sequentially plotted from beginning to end without interruption. Any supplemental cross sections, including ramps, channel changes, etc., should be grouped at the end of the mainline cross sections with each sheet clearly labeled in the lower right corner to identify the purpose and the location of the supplemental cross sections.



CROSS SECTION OFFSET MATCH LINE

Figure 5-1D



TYPICAL PLOTTED CROSS SECTIONS OF EXISTING GROUND LINE

Figure 5-1C

5-2.0 PLOTTING THE ROADWAY TEMPLATE

The roadway template is a graphical representation of the proposed roadway typical section superimposed on the plotted ground line cross section. The roadway template establishes the basis for computing the earthwork quantities. When earthwork is computed manually, the roadway templates should be initially plotted with a soft pencil because adjustments to the profile grade may be necessary to attain the desired balance of earthwork. When the computer is used for earthwork calculations, the roadway templates should not be plotted until all adjustments have been made and the final grades have been established.

5-2.01 Typical Section Controls

The proposed typical section, or sections, for a project define the basic geometric criteria of the roadway template and include:

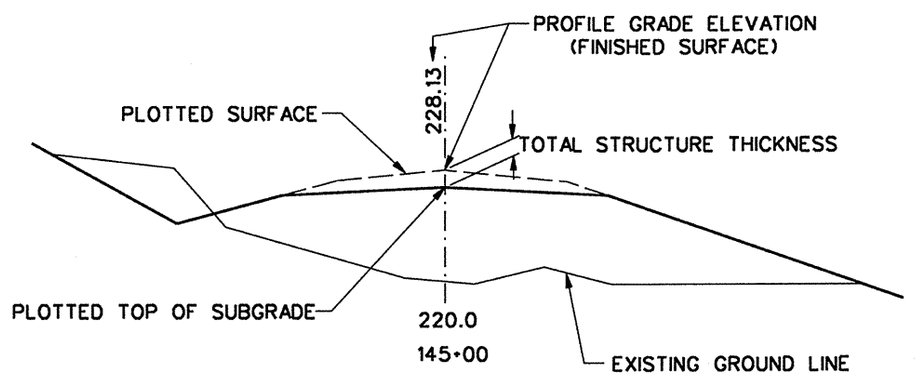
1. cross slopes,
2. safety slopes,
3. cut and fill slopes,
4. location of ditch line, and
5. vertical distance from profile grade to subgrade (structure thickness).

5-2.02 Elevation Controls

The elevation of each roadway template is controlled by the computed profile grade elevation for the corresponding station. For 2-lane roadways and for symmetrical multilane roadways having common profile grades, the elevation should be recorded vertically above the point representing the profile grade point of the template. In the case of multilane roadways having separate profile grades, the elevation should be recorded above the profile grade point of each roadway template. The accuracy of the grade elevation shall be to the nearest 0.01 ft.

5-2.03 Plotting

A typical plotted roadway template is illustrated in Figure 5-2A. The principal objective in plotting the roadway template is to define the shape and limits of roadway excavation and embankment sections relative to the existing ground line. It is desirable to plot the surfacing elements of the template (broken line) to clearly illustrate the conditions of pavement width transition, superelevation, intersections, ramps, etc. In urban areas, the roadway surface is plotted to define relationships between the curbs and the adjacent ground lines and to define drop inlets and other drainage structures. It is not mandatory to plot the structure portion of the template (solid line) although, to assist in the computation of earthwork quantities, the top of the subgrade should be plotted directly at an elevation equal to the profile grade minus the structure thickness.



TYPICAL PLOTTED ROADWAY TEMPLATE

Figure 5-2A

5-2.04 Drainage Structures

All minor drainage structures will be plotted to scale on the appropriate cross sections to assist the designer in the physical design of the drainage facility. The cross sections are used to establish flow lines and estimate lengths of culverts, ends of structures, connecting side ditches, etc. The design criteria and physical standards for drainage structures are discussed in Chapter 7.

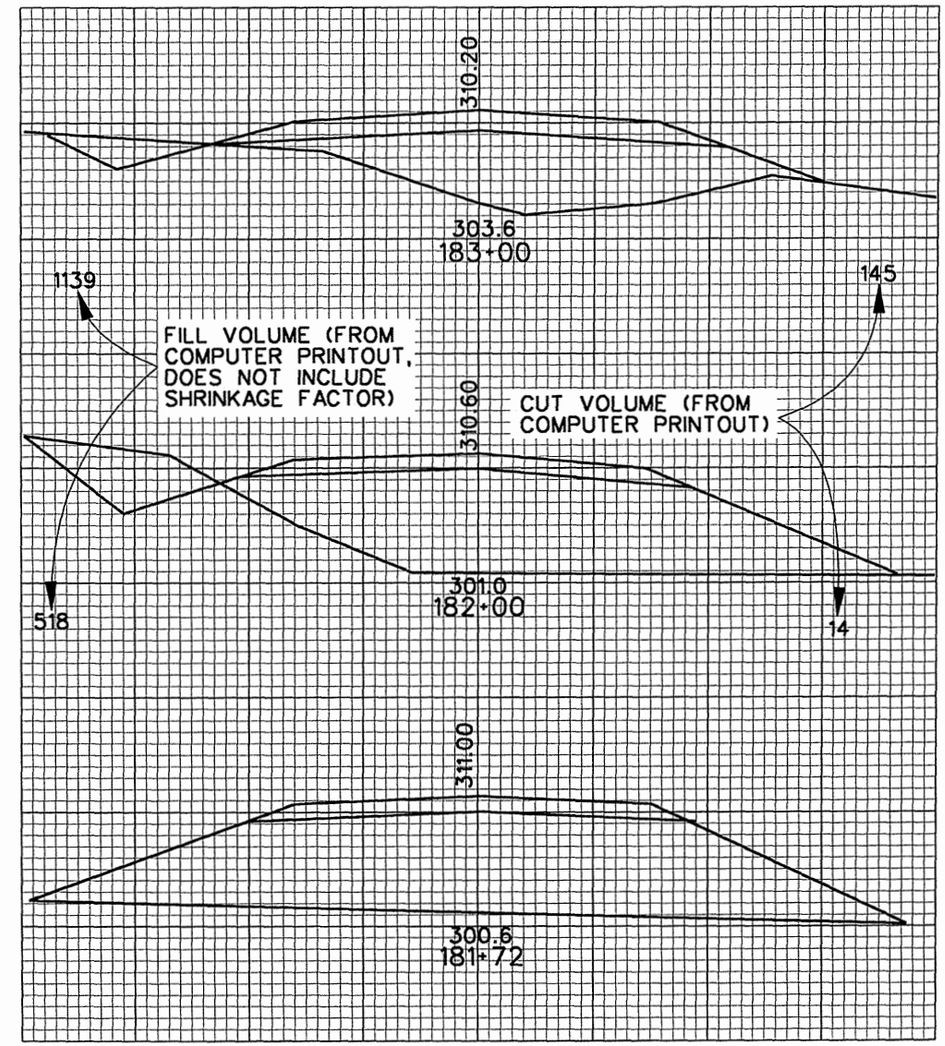
5-3.0 EARTHWORK COMPUTATION

5-3.01 ~~Computer Computations~~ Computer Computations

The Department's instruction manual provides the designer with the required input for computer earthwork programs. Computer computations of earthwork volumes are performed based on data entered from the field survey cross section notes and from data entered by the designer relative to profile grades and typical sections. The printed output from the computer consists of:

1. profile grade elevations,
2. profile gradients in percent,
3. superelevation rates,
4. descriptions of each roadway template including slope stakes,
5. cut and fill areas of each roadway template,
6. earthwork volumes,
7. mass ordinates,
8. plot of the mass ordinates, and
9. plot of the slope stakes.

When the desired earthwork balance has been attained and the final profile grades have been established, the roadway templates should be plotted and the earthwork volumes recorded on the cross section sheets. A typical cross section with computer earthwork volumes recorded is illustrated in Figure 5-3A. Fill volumes should be



TYPICAL CROSS SECTION WITH COMPUTER EARTHWORK VOLUMES

Figure 5-3A

placed on the left, and cut volumes should be placed on the right of the corresponding cross sections.

5-3.02 Manual Computations

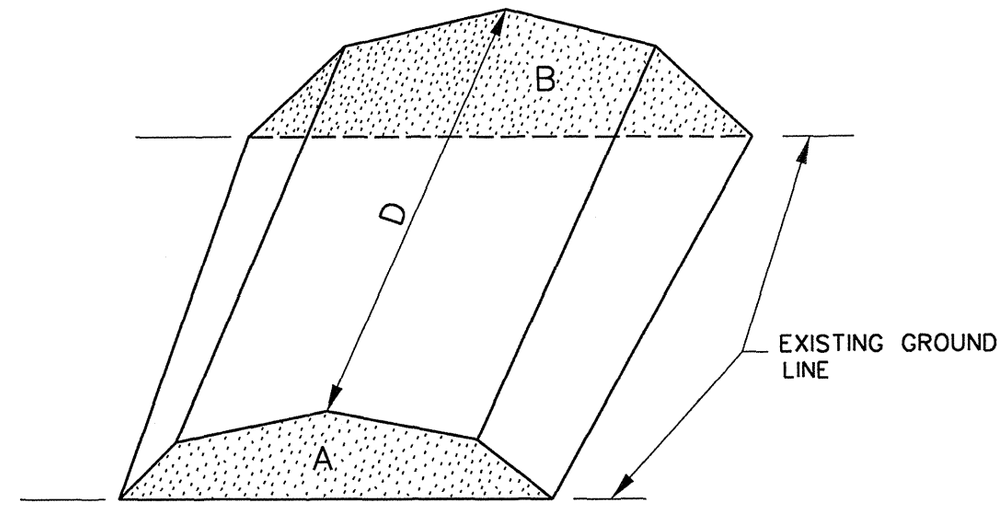
Manual computations of earthwork volumes are performed based on the end areas calculated between the prepared existing ground line and information plotted by the designer relative to the profile grade and typical sections. The end areas of each cross section can be measured with a planimeter or computed mathematically by breaking the area into several smaller geometric shapes or by the coordinate method (see Section 5-3.02.01). The concept of average end areas is illustrated in Figure 5-3B.

5-3.02.01 Coordinate Method for End Areas

A typical cross section illustrating the coordinate method for end areas is presented in Figure 5-3C. Vertical and horizontal coordinates are identified for each irregular breakpoint of the area. The upper figure at each point is the elevation, and the lower figure is the horizontal distance from the reference line (usually the centerline).

The principle of the coordinate method is to multiply the difference in elevation of two adjacent points by the sum of the horizontal distances of the points from the reference line. The sign of the computation is plus (+) if the grade slopes upward and minus (-) if the grade slopes downward. The end area of the section is one-half of the algebraic sum of the individual computations.

A convenient procedure is to start at the ground elevation at the centerline and perform the computations in a counterclockwise direction returning to the reference line. The process is then repeated in a clockwise direction for the left portion of the area. This sequence is illustrated by the arrows in the figure. Example 5-3.1 provides the end area computation based on the information presented in Figure 5-3C.

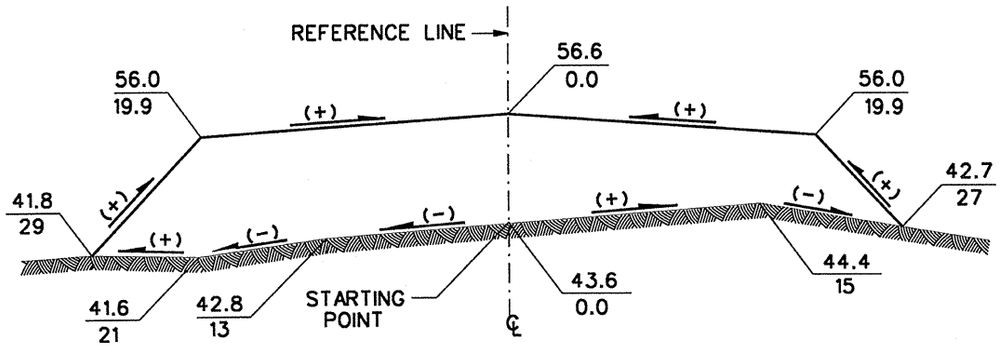


$$V = D\left(\frac{A + B}{2}\right) \div 27 \text{ or } V = \frac{D\,(A + B)}{54} \qquad \textit{(Equation 5-3.1)}$$

Where: V = *Volume between sections (yd^3)*
A *and* B = *End areas of adjacent sections (ft^2)*
D = *Distance between sections (ft)*

AVERAGE END AREAS

Figure 5-3B



COORDINATE METHOD FOR END AREAS

Figure 5-3C

5-3.02.02 Strip Stick Method for Volumes

A commonly accepted technique within the Department for earthwork computation is the "strip stick" method. This method maintains the basic concept of the average end area; however, the design of the strip stick permits measurements of irregular sections so that the cumulative readings on the sticks may be converted to volumes without the intermediate step of recording the actual end areas.

Two "strip sticks" are available to the designer. One applies to cross sections with a horizontal and vertical scale of 1 inch = 10 ft, and the other applies to cross sections with a horizontal and vertical scale of 1 inch = 5 ft. If scales other than these are used, conversion factors must be applied to the strip stick readings to reflect the proportionate difference in actual end areas.

Example 5-3.1

Given: See Figure 5-3C for the appropriate vertical and horizontal coordinate values.

Problem: Determine the end area of the given cross section by the coordinate method.

Solution: From Figure 5-3C:

$$A = 0.5[(0.8)(15)-(1.7)(42)+(13.3)(46.9)+(0.6)(19.9)]+ 0.5[-(0.8)(13)-(1.2)(34)+(0.2)(50)+(14.2)(48.9)+(0.6)(19.9)]$$

$$= 0.5[12 - 71.4 + 623.27 + 11.94] + 0.5[-10.4 - 40.8 + 10.0 + 694.38 + 11.94]$$

$$= 288.16 + 332.56$$

$$= 620.72 \text{ ft}^2$$

End areas computed by the coordinate method can be used in Equation 5-3.1 of Figure 5-3B to compute volumes between the cross sections.

5-3.03 Balance Factor

The designer should apply a balance factor to the earthwork computations to account for the conditions of shrinkage and swell. If these conditions are not considered, the theoretical volumes of cut and fill when balanced on paper will not balance during field construction. For shrinkage, there are two methods of applying the balance factor:

1. increase the computed embankment quantity by the balance factor to determine the amount of excavation actually needed to construct the embankments; or

2. decrease the computed excavation quantity by the balance factor to determine the quantity of embankment that will be produced.

The first method is used when a project is to be balanced with excavation from within the roadway prism. The second method is generally used when the roadway prism cannot be balanced and other borrow excavation must be obtained.

The balance factor will vary with location and soil characteristics. A recommended balance factor should be provided by the District.

5-3.04 Earthwork Calculations

The cuts and fills will be shown on the plan/profile sheets. Any special borrow or excess excavation due to unsuitable material or high-volume change soil will be shown on cut and fill lines. The sheet totals will include cut, fill, borrow and/or excess. Special borrow and excess amounts will not be included in cut and fill totals. The borrow and/or excess that results from cuts and fills will be shown on the estimated quantity sheet. Figures 5-3D and 5-3E illustrate example earthwork calculations of excess and borrow conditions, respectively. The last plan/profile sheet will include a block listing the cumulative totals from previous plan/profile sheets.

Earthwork quantities will be shown on plan/profile sheets whenever practical. However, if plan/profile sheets cannot be used to show earthwork, then separate earthwork sheets may be developed. Original and finish grade lines must be provided on both the plan/profile sheets and the earthwork sheets if separate earthwork sheets are utilized.

When less than approximately 15% of the required embankment material is available from excavation within the right-of-way, the excavation is identified as Estimated State Furnished Excavation (ESFE) and becomes the property of the contractor. It must be excavated, used in the embankment if suitable, and paid for as borrow excavation (i.e., Furnished Material Excavation or FME). The embankment volume is normally totaled every 1000 ft and no balance factor is applied.

ESTIMATED EARTHWORK QUANTITIES						
Work Sheet Number	Plan & Profile		Borrow Excavation (yd^3)	Excess Excavation (yd^3)	Unclassified Excavation (yd^3)	Remarks
	Cut (yd^3)	Fill (yd^3)				
3	206,111	67,257	4,442①	51,550①		High-Volume Change Soil
4	78,886	129,291	3,322①	31,446①		High-Volume Change Soil
5	172,436	8,925	11,185①	127,627①		High-Volume Change Soil
Subtotal	457,433	205,473	18,949①	210,623①		High-Volume Change Soil
(Cut)-(Fill)(1+SF)=Excess						
(457,433)-(205,423)(1.25)=				314,950		From Cuts & Fills
(Fill)(1+SF)=Unclassified						
(205,433)(1.25)=					256,841	From Cuts & Fills
PROJECT TOTAL			18,949	525,573	256,841	

Notes:

① Due to the removal and replacement of high-volume change soil.

EXAMPLE EARTHWORK CALCULATION
(Excess Condition)

Figure 5-3D

ESTIMATED EARTHWORK QUANTITIES						
Work Sheet Number	Plan & Profile		Borrow Excavation (yd^3)	Excess Excavation (yd^3)	Unclassified Excavation (yd^3)	Remarks
	Cut (yd^3)	Fill (yd^3)				
3	32,229	147,116	4,442 ①	51,550①		High-Volume Change Soil
4	20,706	155,872	3,322①	31,446①		High-Volume Change Soil
5	15,614	26,106	11,185①	127,627①		High-Volume Change Soil
Subtotal	68,549	329,094	18,949①	210,623①		High-Volume Change Soil
Cut = Unclassified =					68,549	From Cuts & Fills
(Fill)-(Cut)/(1+SF) = Borrow						
(329,094)-(68,549)/(1.25) =			274,255			From Cuts & Fills
PROJECT TOTAL			293,204	210,623	68,549	

Notes:

① Due to the removal and replacement of high-volume change soil.

EXAMPLE EARTHWORK CALCULATION
(Borrow Condition)

Figure 5-3E

Chapter 6

At-Grade Intersections/Interchanges

MISSISSIPPI DESIGN MANUAL

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ROADWAY DESIGN

Chapter 6

AT-GRADE INTERSECTIONS/INTERCHANGES

Section		Page
6-1.0	GENERAL DESIGN CONTROLS	6-1
6-1.01	Design Vehicles	6-1
6-1.02	Horizontal Alignment	6-1
6-1.03	Profile	6-2
6-1.04	Capacity and Level of Service	6-3
6-1.05	Types of Intersections	6-3
6-1.06	Intersection Spacing	6-3
6-2.0	TURNING RADII DESIGN	6-3
6-2.01	Selection of Design Vehicle	6-3
6-2.02	Turning Design Criteria	6-4
6-2.02.01	Rural	6-4
6-2.02.02	Urban	6-4
6-2.03	Encroachment	6-6
6-2.04	Other Intersection Factors	6-6
6-3.0	TURNING ROADWAYS	6-8
6-4.0	AUXILIARY LANES	6-8
6-4.01	Warrants for Right-Turn Lanes	6-8
6-4.02	Warrants for Left-Turn Lanes	6-9
6-4.03	Length of Auxiliary Turning Lanes	6-9
6-4.03.01	Taper Length	6-9
6-4.03.02	Application of Deceleration Length	6-12
6-4.03.03	Storage Length	6-13
6-4.03.04	Design Length for Extension of Through Lanes	6-14
6-4.03.05	Additional Design Considerations	6-15
6-5.0	CONTINUOUS TWO-WAY LEFT-TURN LANES	6-19
6-5.01	Guidelines for Application	6-19
6-5.01.01	Typical Sites	6-19
6-5.01.02	Functional Class	6-19
6-5.01.03	Traffic Volumes	6-19
6-5.01.04	Design Speed	6-19
6-5.02	Design Criteria	6-20
6-5.02.01	Lane Width	6-20
6-5.02.02	Intersection Treatment	6-20
6-5.02.03	Traffic Control	6-21
6-6.0	INTERSECTION SIGHT DISTANCE (ISD)	6-21
6-6.01	General	6-21
6-6.02	Background	6-21
6-6.02.01	Basic Criteria	6-21
6-6.02.02	Sight Flare Criteria	6-22
6-6.03	Application (Stop-Controlled Intersections)	6-22
6-6.03.01	Vehicle Entering Major Roadway	6-22
6-6.03.02	Straight Through Crossing Vehicle	6-25
6-6.03.03	All-Way Stop	6-25
6-6.04	Signal-Controlled Intersections	6-26
6-6.05	Stopped Vehicle Turning Left	6-26
6-6.06	Sight Flare Criteria	6-28
6-6.06.01	General	6-28
6-6.06.02	Urban Signalized Intersections	6-28
6-6.07	Examples of ISD Applications	6-28
6-7.0	CHANNELIZATION	6-31
6-7.01	Type	6-31
6-7.02	Design of Islands	6-32
6-7.02.01	Types of Islands	6-32
6-7.02.02	Minimum Size	6-32
6-7.02.03	Delineation	6-32
6-7.02.04	Approach Treatment	6-34
6-7.02.05	Offsets	6-34
6-7.03	Turning Roadways	6-34
6-7.03.01	Guidelines	6-34
6-7.03.02	Curvature	6-34
6-7.03.03	Deceleration and Acceleration Lanes	6-34
6-7.03.04	Cross-Slope Rollover	6-36
6-7.03.05	Width of Roadway	6-36
6-8.0	MEDIAN OPENINGS	6-38
6-8.01	Warrants/Spacing	6-38
6-8.01.01	Freeways	6-38
6-8.01.02	Non-Freeways	6-38
6-8.02	Design	6-38
6-8.02.01	Turning Radii	6-38
6-8.02.02	Shape of Median Ends	6-45
6-8.02.03	Length of Opening	6-45
6-8.03.04	U-turns	6-46
6-8.03.05	Skewed Intersections	6-46
6-9.0	RURAL DRIVEWAYS	6-46
6-10.0	INTERCHANGES	6-50
6-10.01	Warrants	6-50
6-10.01.01	Guidelines	6-50
6-10.01.02	New/Revised Access Points on the Interstate System	6-51
6-10.02	Interchange Types	6-51
6-10.03	Freeway/Ramp Junctions	6-52
6-10.03.01	Exit Ramps	6-52
6-10.03.02	Entrance Ramps	6-53
6-10.04	Access Control	6-56
6-11.0	REFERENCES	6-58

Chapter 6

AT-GRADE INTERSECTIONS/INTERCHANGES

This Chapter discusses the geometric criteria incorporated into the design of at-grade intersections and grade-separated interchanges. Intersections and interchanges are important elements of the highway system. The operational efficiency, the capacity, the safety and the cost of the system depend largely upon their design. The primary objective of intersection and interchange design is to reduce potential conflicts between vehicles, bicycles and pedestrians while providing for the convenience, ease and comfort of those traversing the intersection or interchange.

6-1.0 GENERAL DESIGN CONTROLS

6-1.01 Design Vehicles

The shape and sharpness of turns at intersections and the widths of turning roadways must be designed to accommodate the characteristics of vehicular operation. The designer should select the design vehicle based on the largest vehicle that will regularly use the intersection. The basic design vehicles used for intersection design are:

1. P — Passenger car and light panel and pickup trucks.
2. SU — Single-unit truck or small bus.
3. BUS — Transit bus.
4. WB-40 — Semitrailer combination with an overall wheelbase of 40 ft.
5. WB-50 — Semitrailer combination with an overall wheelbase of 50 ft.
6. WB-62 — Semitrailer combination with an overall wheelbase of 62 ft.

The SU vehicle is generally the smallest vehicle used in the design of intersections. This reflects that, even in residential areas, delivery trucks will be negotiating turns with some frequency. On facilities accommodating regular truck traffic, one of the semitrailer combinations should be used for design. The selection of the design vehicle is also based on the area type and functional classification of the intersecting highways. Section 6-2.01 discusses the selection of a vehicle specifically for the turning radii designs.

6-1.02 Horizontal Alignment

Roadways should intersect on tangent sections. The intersection of roadways on sharp curves should be avoided wherever practical because superelevation complicates the intersection design and the curvature may contribute to sight distance problems.

Roadways should also intersect at right angles. Roads intersecting at acute angles are undesirable for several reasons:

1. Vehicular turning movements become more restricted.

2. The accommodation of large trucks may require additional pavement and channelization.

3. The exposure time to vehicles and pedestrians crossing the main traffic flow is increased.

4. The driver's angle of sight for one of the sight triangles becomes restricted.

Desirably, the angle of intersection will be within 10° of perpendicular; the maximum will be 30°. This amount of skew can often be tolerated. The impact on sight lines and turning movements will not be large. However, intersections outside of this range may warrant geometric improvements such as realignment or greater corner sight distance. Figure 6-1A illustrates various angles of intersection and potential improvements to the alignment.

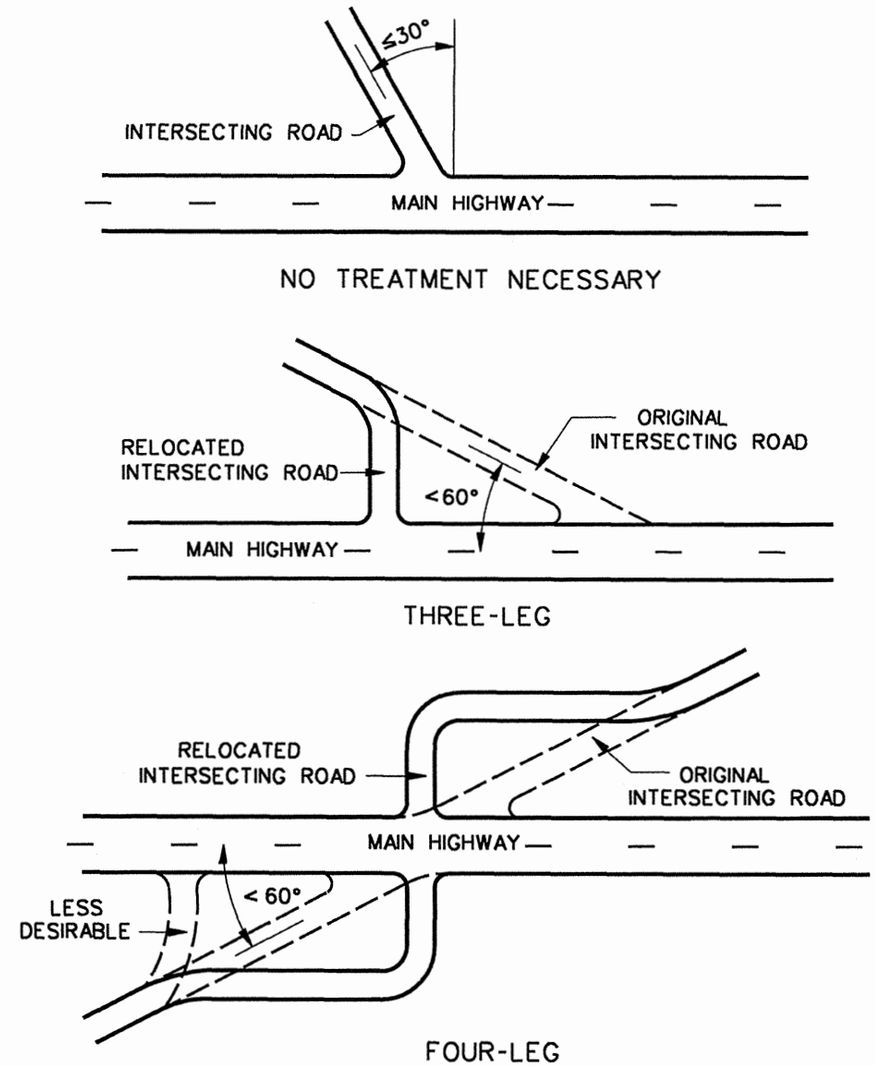
Avoid constructing short radii horizontal curves on side road approaches to achieve right-angle intersections. These curves result in increased lane infringements because drivers tend to smooth out curves by encroaching into a portion of the opposing travel lane. Also, the traffic control devices at the intersection may be obscured resulting in the need for advance signing.

6-1.03 Profile

Avoid combinations of grade lines that make vehicular control difficult at intersections. Substantial grade changes should be avoided, but it is not always practical to do so.

The gradients of intersecting highways should be as flat as practical on those sections that will be used for storage space for stopped vehicles. This is sometimes referred to as the storage platform. Most vehicles with either manual or automatic shifts must have the brakes applied to stand still while the motors are running, unless they are stopped on a gradient flatter than 1 percent.

Intersections with stop control should maintain the profile and cross section of the major road through the intersection. In these cases, the minor road should be transitioned (or warped) to fit the profile of the major road. At signalized intersections or at intersections where the roadways are nearly equal in importance, transition both roadways to a plane section. Where compromises are necessary between the two, the smoother riding characteristics should be provided for the roadway with higher traffic volumes and operating speeds.



TREATMENTS FOR SKEWED INTERSECTIONS
Figure 6-1A

6-1.04 Capacity and Level of Service

A capacity analysis should be performed before performing the detailed design of any intersection. This analysis will influence several geometric design features including number of approach lanes, lane widths, channelization and number of departure lanes. The designer should select a level of service and a future design year, typically 20 years for new construction and reconstruction. Level-of-service recommendations are provided in the Department's tables of geometric design criteria in Chapters 2, 11, 13 and 14. Once the level of service and design traffic volumes are determined, the designer should use the *Highway Capacity Manual* for the detailed capacity analysis.

6-1.05 Types of Intersections

Intersections at-grade are usually the 3-leg (T- or Y-shape) or the 4-leg design. Individual intersections may vary in size and shape and may be non-channelized, flared or channelized. The principal factors which affect the selection of intersection type and its design characteristics are the DHV, turning movements, the character or composition of traffic, the design speed and the angle of intersection.

6-1.06 Intersection Spacing

Short distances between intersections should be avoided if practical because they tend to impede traffic operations. For example, if two intersections are close together and require signalization, they may need to be considered as one intersection for signalization purposes. To operate safely, each leg of the intersection may require a separate green cycle, thereby greatly reducing the capacity for both intersections. To operate efficiently, signalized intersections should be a minimum of 1325 ft apart.

In addition, short gaps between opposing "T" intersections should be avoided. Drivers tend to encroach into the opposing lanes (corner cutting) so that they can make their turning maneuvers in one movement. In general, all new intersections should preferably be at least 400 ft apart.

6-2.0 TURNING RADII DESIGN

Turning radii treatments for intersections at-grade are important design elements. They influence the operation of the intersection, safety and construction costs. Turning radii design often does not receive sufficient attention; therefore, the designer should ensure that the design is compatible with the intersection operations.

6-2.01 Selection of Design Vehicle

In general, the turning radii at an intersection should be based on the largest vehicle that will regularly use the intersection. Types of design vehicles are discussed in Section 6-1.01. The designer should evaluate the area type and functional classification of intersecting highways. Table 6-2A presents the desirable and minimum design vehicle selection for rural intersections; Table 6-2B provides the criteria for urban intersections. Selection of the applicable design vehicle should be based on the functional class of the highway which the vehicle is turning onto. For example, in an urban area a collector and an arterial highway may intersect. The desirable design vehicle turning from the collector to the arterial is a WB-50; from the arterial to the collector, it is a WB-40.

The WB-62 design vehicle (large truck) is allowed on the Interstate highway system and other roads as designated on the National Truck Network. These trucks must have reasonable access to truck facilities near designated highways. Therefore, the WB-62 may be allowed to make right turns at some at-grade intersections, and the designer should consider this possibility. The designer should exercise judgment when deciding which intersections should be designed according to the turning characteristics of the large truck. Some intersections (e.g., those near truck stops) are obvious candidates. The designer should also consider whether or not an intersection may in the future need to accommodate the large truck.

Table 6-2A

SELECTION OF DESIGN VEHICLE
(Rural Intersections)

Functional Class	Design Vehicle	
	Desirable	Minimum
Freeway Ramps	WB-62	WB-50*
Arterials	WB-50	WB-50
Collectors	WB-50	WB-40
Local Roads	WB-40	SU

* WB-62 must be physically able to make the turn.

Table 6-2B

SELECTION OF DESIGN VEHICLE
(Urban Intersections)

Functional Class	Design Vehicle	
	Desirable	Minimum
Freeway Ramps	WB-62	WB-50*
Arterials	WB-50	WB-40
Collectors	WB-40	SU
Local Roads	SU	P

* WB-62 must be physically able to make the turn.

Note: On bus transit routes, a BUS is the desirable design vehicle for turning radii.

Some portions of an intersection may be designed with one design vehicle and other portions with another vehicle. For example, it may be desirable to design physical characteristics (curbs, islands) for one design vehicle but provide painted channelization for a smaller vehicle. This will provide a positive indicator for the more frequent turning vehicles.

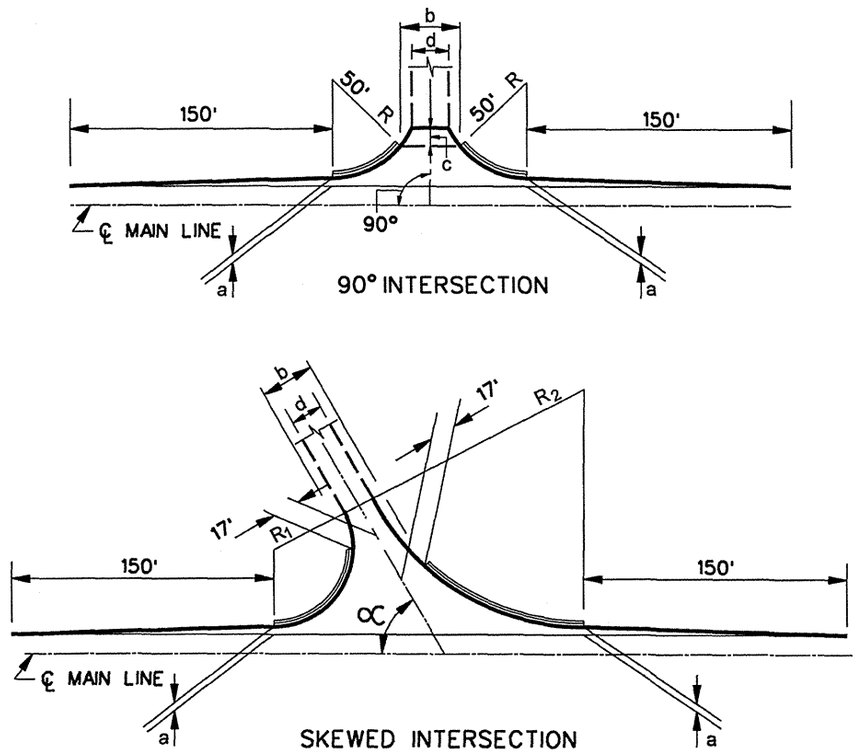
6-2.02 Turning Design Criteria

6-2.02.01 Rural

For rural intersections, the design for right turns will normally provide a simple curve with a taper offset onto the major road. Figure 6-2A illustrates a typical design for both a 90° and a skewed rural intersection. These designs provide a relatively easy maneuver for vehicles exiting or entering the major road.

6-2.02.02 Urban

For urban intersections, the designer will normally use a simple curve radius for the design of the pavement edge or for the curb line. Table 6-2C provides the curb radii recommendations for various design vehicles. These criteria are adequate for low-speed turning movements. The designer should check all proposed designs with the applicable vehicular turning template. Where it is desirable for vehicles to turn at a higher speed, such as for high-volume turns or turns off high-volume streets, larger radii or turning roadways may be warranted. Turning roadways are discussed in Section 6-7.03.



a = Shoulder Width. *Note: Where shoulder width is less than 5 ft, use 5-ft minimum.*
b = 34 ft Minimum
c = 10 ft Minimum
d = Variable (20-ft Minimum)
α = Angle of Intersection (80° - 90° Desirable, 60° Minimum) *Note: Relocate intersection where angle is less than 60°.*
R_1 = 50 ft
R_2 = Variable (radius point opposite smaller radius point)

SIMPLE RURAL INTERSECTION WITH A TAPER OFFSET
(Non-Channelized)

Figure 6-2A

Table 6-2C

RECOMMENDED CURB RADII
(Urban Intersections)

Design Vehicle	Radius
P	20 ft*
SU	35 ft
WB-40	40 ft
BUS	45 ft
WB-50	50 ft
WB-62	65 ft

**In residential areas, a radius of 15 ft is acceptable.*

Notes:

1. *Criteria in the table are applicable to intersections where:*
   a. *very little encroachment onto adjacent lanes can be tolerated (see Section 6-2.03 for further discussion),*
   b. *the angle of turn is 90° ± 20°, and*
   c. *parking lanes or shoulders are <u>not</u> available to provide additional width for the turning maneuver (see Section 6-2.04 for further discussion).*
2. *Where a simple radius is considered inadequate for the intersection turn, the designer should consider using the alternative criteria presented in* A Policy on Geometric Design of Highways and Streets *(e.g., simple radius with taper offsets, 3-centered curves).*
3. *For large radii, the designer should consider providing a turning roadway; see Section 6-3.0.*

6-2.03 Encroachment

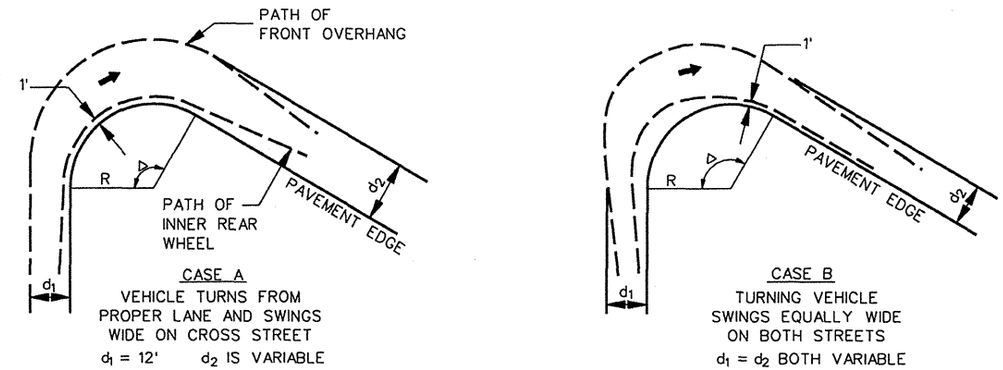
The criteria in Table 6-2C may not be attainable at some intersections. In these cases, the designer must consider the encroachment which can be tolerated and the frequency for which this encroachment is acceptable. Several factors will influence this assessment including traffic volumes, truck traffic, functional classification, 1-way or 2-way operation, available roadway width and traffic control.

Table 6-2D provides guidelines for determining the encroachment that can be tolerated for various conditions. Figure 6-2B provides the information necessary to determine the amount of encroachment by a turning vehicle. The value will vary with the angle of turn, the design vehicle and the radius of the curb line.

6-2.04 Other Intersection Factors

Where on-street parking is allowed, the additional roadway width will decrease the turning problems for large vehicles at intersections with small radii. Parking must be restricted in advance of the PC and beyond the PT for the simple curve radius design. The designer should evaluate each intersection on a case-by-case basis. For turning movements, parking should generally be prohibited for approximately 15 ft before the PC and approximately 30 ft beyond the PT. Even for radii as small as 15 ft, a WB-50 can negotiate the turn with little encroachment into the adjacent lane, provided the parking restrictions are enforced. For truck maneuvers, the presence of a shoulder 8 ft to 12 ft in width will have the same impact as a parking lane.

Pedestrians are affected by the selection of the turning radii. The greater the radius, the further pedestrians must walk in the roadway. This may lead to, for example, the decision to use a turning roadway to provide a pedestrian refuge. Smaller turning radii, typically less than 30 ft, can substantially reduce the vehicular turning speed and reduce the open pavement area for pedestrians crossing the road. This is especially important to handicapped individuals. However, these radii may cause encroachment problems. These trade-offs must be considered when designing the intersection.



d_1 = MAXIMUM DISTANCE BETWEEN THE PAVEMENT EDGE (BEFORE THE TURN) AND THE VEHICLE OVERHANG.
d_2 = MAXIMUM DISTANCE BETWEEN THE PAVEMENT EDGE (AFTER THE TURN) AND THE VEHICLE OVERHANG.

Angle of Turn Δ	Design Vehicle	d_2 ft) for Cases A and B Where:									
		R = 15 ft		R = 20 ft		R = 25 ft		R = 30 ft		R = 40 ft	
		A	B	A	B	A	B	A	B	A	B
30°	SU	14	13	14	13	13	13	13	13	13	13
	BUS	22	17	19	17	19	17	19	17	18	17
	WB-40	14	14	14	14	14	14	14	14	14	14
	WB-50	20	17	20	17	20	17	19	16	18	16
	WB-62	—	—	—	—	—	—	—	—	27	17
60°	SU	19	16	19	16	17	15	16	15	14	14
	BUS	28	21	26	20	24	20	23	19	22	18
	WB-40	24	19	22	19	21	19	19	18	17	16
	WB-50	31	22	27	21	28	20	25	19	22	18
	WB-62	—	—	—	—	—	—	—	—	30	22
90°	SU	26	20	23	18	19	16	17	15	13	13
	BUS	38	23	33	22	30	22	25	21	22	18
	WB-40	31	22	27	21	23	21	19	18	17	16
	WB-50	42	22	37	24	32	22	29	21	22	18
	WB-62	—	—	—	—	—	—	—	—	39	23
120°	SU	34	22	27	19	21	18	17	16	13	13
	BUS	46	28	40	25	32	23	26	19	19	18
	WB-40	37	23	29	22	24	22	19	18	17	16
	WB-50	50	29	43	28	36	27	30	26	22	18
	WB-62	—	—	—	—	—	—	—	—	26	22
150°	SU	40	25	32	21	22	19	17	16	12	12
	BUS	48	28	40	25	32	23	22	18	17	16
	WB-40	39	24	29	22	23	22	19	18	17	16
	WB-50	53	31	46	28	36	27	28	26	22	18
	WB-62	—	—	—	—	—	—	—	—	20	18

Note: P design vehicle turns within 12-ft width where R = 15 ft or more. No parking on either street.

CROSS STREET WIDTH OCCUPIED BY A TURNING VEHICLE
Figure 6-2B

Table 6-2D

GUIDELINES FOR ENCROACHMENT FOR RIGHT TURNS

Turn Made From	Turn Made Onto	Acceptable Encroachment for Design Vehicle for Road/Street Onto Which Turn Made
Arterial	Arterial Collector Local	No encroachment into opposing lanes of travel 1 ft encroachment into opposing lanes of travel 2 ft encroachment into opposing lanes of travel
Collector	Arterial Collector Local	No encroachment into opposing lanes of travel 2 ft encroachment into opposing lanes of travel 3 ft encroachment into opposing lanes of travel
Local	Arterial Collector Local	No encroachment into opposing lanes of travel 3 ft encroachment into opposing lanes of travel 4 ft encroachment into opposing lanes of travel

Notes:

1. *See Tables 6-2A and 6-2B for the design vehicle selection. The encroachment criteria refer to the desirable design vehicle.*

2. *Before the turn is made, the design vehicle is assumed to be in the outermost through travel lane or exclusive right-turn lane, whichever applies. It is assumed that the vehicle does not encroach onto adjacent lanes on the road/street from which the turn is made (i.e., Case "A" in Figure 6-2B).*

3. *When determining the acceptable encroachment, the designer should also consider turning volumes, through volumes and the type of traffic control at the intersection. In all cases, the desirable encroachment will be zero into the opposing lanes of travel.*

4. *Check all proposed designs with the applicable vehicular turning template.*

6-3.0 TURNING ROADWAYS

It is often desirable to design at-grade intersections to allow turning vehicles to operate at above minimum speeds. Turning roadways are channelized areas (separated by an island) which allow a moderate-speed right turn away from the intersection area. Section 6-7.03 discusses the design guidelines and warrants for turning roadways.

6-4.0 AUXILIARY LANES

Drivers leaving a highway at an intersection are usually required to reduce their speed before turning. Drivers that enter a highway at an intersection accelerate until the desired open-road speed is reached. Where deceleration or acceleration occurs in the through travel lanes, it disrupts the flow of through traffic. To minimize the potential hazard, the use of auxiliary lanes may be warranted for at-grade intersections to improve the level of service of the highway.

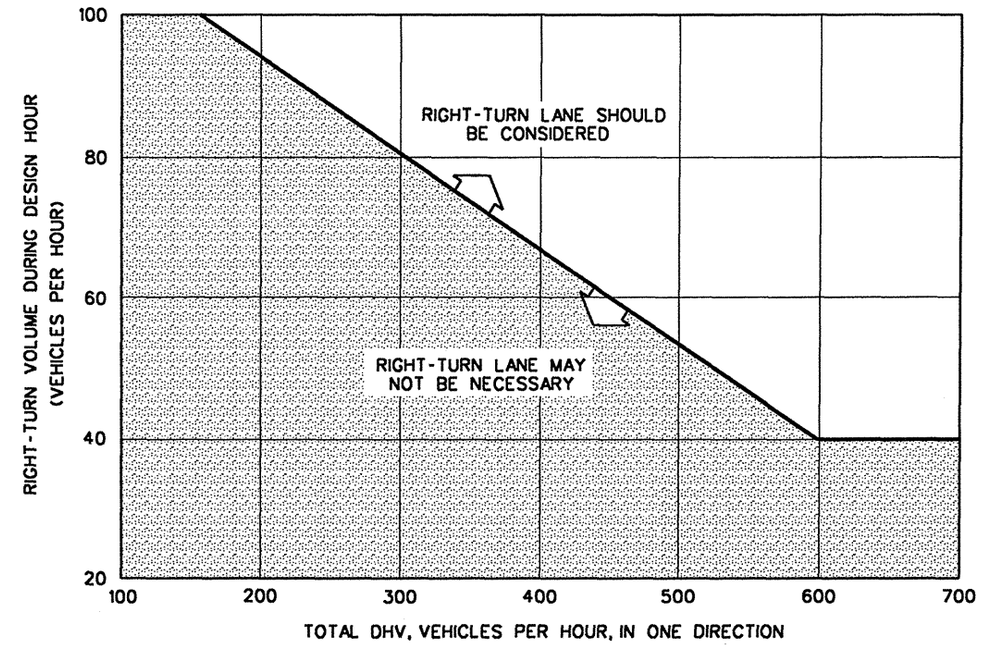
6-4.01 Warrants for Right-Turn Lanes

The use of right-turn lanes at intersections can significantly improve operations. Exclusive right-turn lanes should be provided:

1. at any unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria in Figure 6-4A;

2. at any intersection where a capacity analysis determines a right-turn lane is necessary to meet the level-of-service criteria; or

3. at any intersection where the accident experience, existing traffic operations or engineering judgment indicates a significant hazard related to right-turning vehicles.



Note: For highways with a design speed below 50 mph and a DHV < 300 and Right Turns > 40, an adjustment should be used. To read the vertical axis of the chart, subtract 20 from the actual number of right turns.

Example

Given: Design Speed = 45 mph
DHV = 250 vph
Right Turns = 100 vph

Problem: Determine if a right-turn lane is warranted.

Solution: To read the vertical axis, use 100 - 20 = 80 vph. The figure indicates that a right-turn lane is not warranted unless other factors (e.g., high accident rate) indicate a lane is needed.

RIGHT-TURN LANE WARRANTS AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS

Figure 6-4A

6-4.02 Warrants for Left-Turn Lanes

The accommodation of left turns is often the critical factor in proper intersection design. Left-turn lanes influence both the level of service and intersection safety. Exclusive left-turn lanes should be provided:

1. at all median openings (cross overs) on divided urban and rural highways with a median wide enough to accommodate a left-turn lane;

2. at any unsignalized intersection on a 2-lane highway which satisfies the criteria in Figures 6-4B, 6-4C or 6-4D;

3. at any unsignalized intersection on a 4-lane undivided highway which satisfies the criteria in Figure 6-4E;

4. at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria; or

5. at any intersection where the accident experience, existing traffic operations, adverse geometrics (e.g., restricted sight distance) or engineering judgment indicate a significant hazard related to left-turning vehicles.

The following example illustrates how to use Figures 6-4B through 6-4D to determine the need for a left-turn lane based on volume warrants.

6-4.03 Length of Auxiliary Turning Lanes

The length of a right- or left-turn lane at an intersection should allow safe deceleration and storage of turning vehicles outside of the through lanes. This improves safety and the intersection level of service. The length of auxiliary lanes will be determined by a combination of its taper length (L_T), deceleration length (L_D) and storage length (L_S). The following sections discuss the application of these elements. The minimum overall length of an auxiliary turn lane should be 200 ft.

Figure 6-4F illustrates typical auxiliary lanes at an intersection and their segments. Note that, in the Figure, L_D starts at the beginning of the taper.

6-4.03.01 Taper Length

Tapers should be as long and as smooth as practical so that drivers will be encouraged to use the full length of the taper. At restricted sites, the taper length may be shortened to provide a greater full-width length of the auxiliary lane.

The recommended taper length (L_T) for deceleration lanes is 150 ft. In restricted urban areas, this may be reduced to 75 ft.

Example 6-4.1

Given: Design Speed = 60 mph
V_A = 400 vph
V_O = 200 vph
Left turns in V_A = 10%

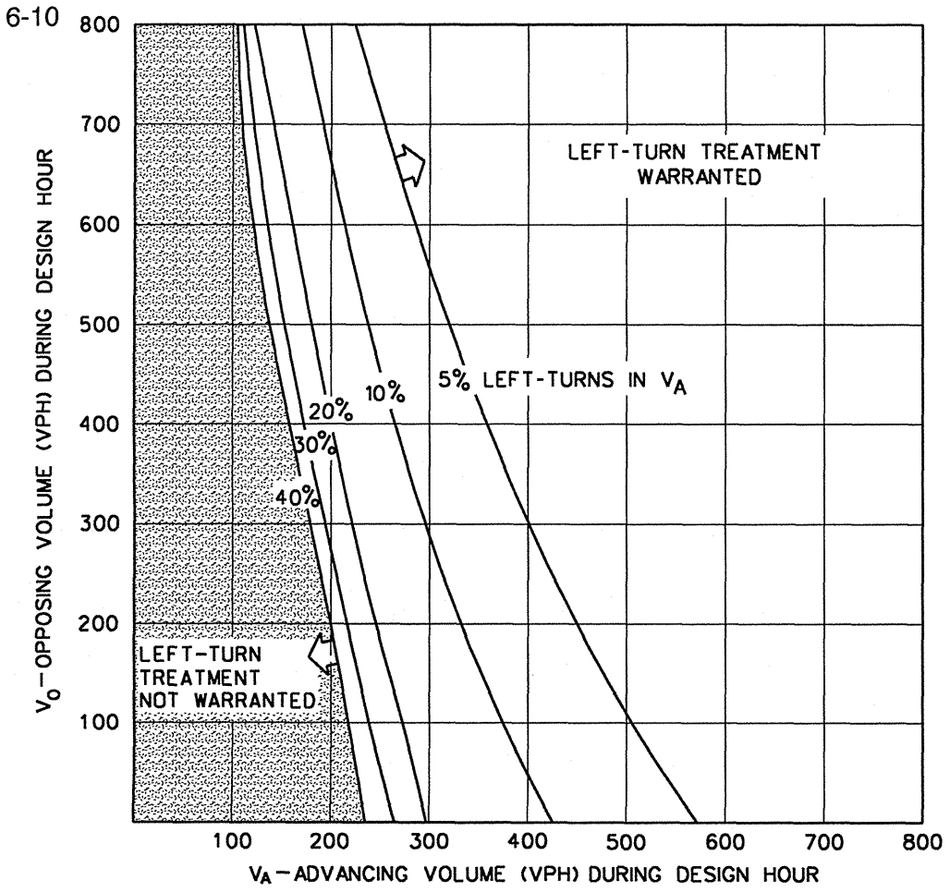
Problem: Determine if a left-turn lane is warranted.

Solution:

Step 1: Use the 10% curve on Figure 6-4B.

Step 2: Read V_A (400 vph) and V_O (200 vph) into the chart and locate the intersection point.

Step 3: The point in Step 2 is located to the right of the curve in Step 1. Therefore, a left-turn lane is warranted.



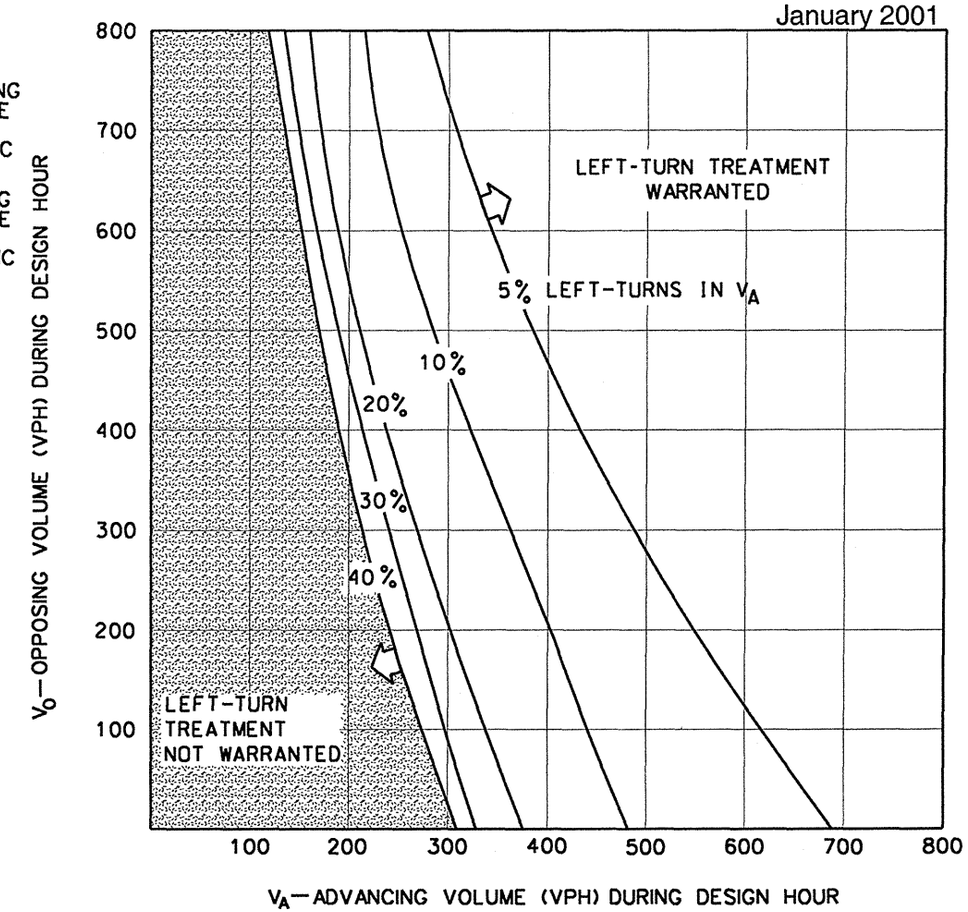
V_A = TOTAL ADVANCING TRAFFIC VOLUME INCLUDING ALL TURNING TRAFFIC

V_O = TOTAL OPPOSING TRAFFIC VOLUME INCLUDING ALL TURNING TRAFFIC

Instructions:

1. *The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*
2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*
3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

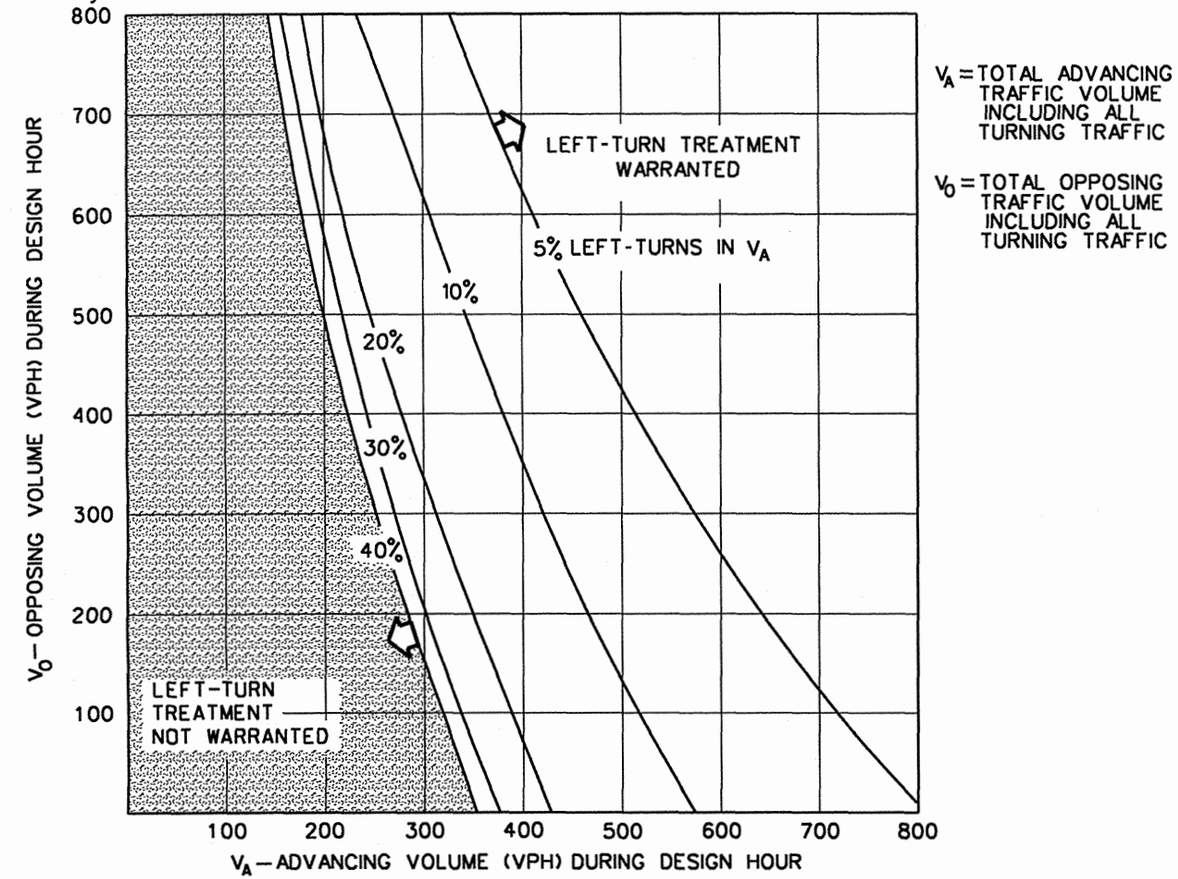
VOLUME WARRANTS FOR LEFT-TURN LANE AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (60 mph)
Figure 6-4B



Instructions:

1. *The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*
2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*
3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME WARRANTS FOR LEFT-TURN LANE AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (50 mph)
Figure 6-4C



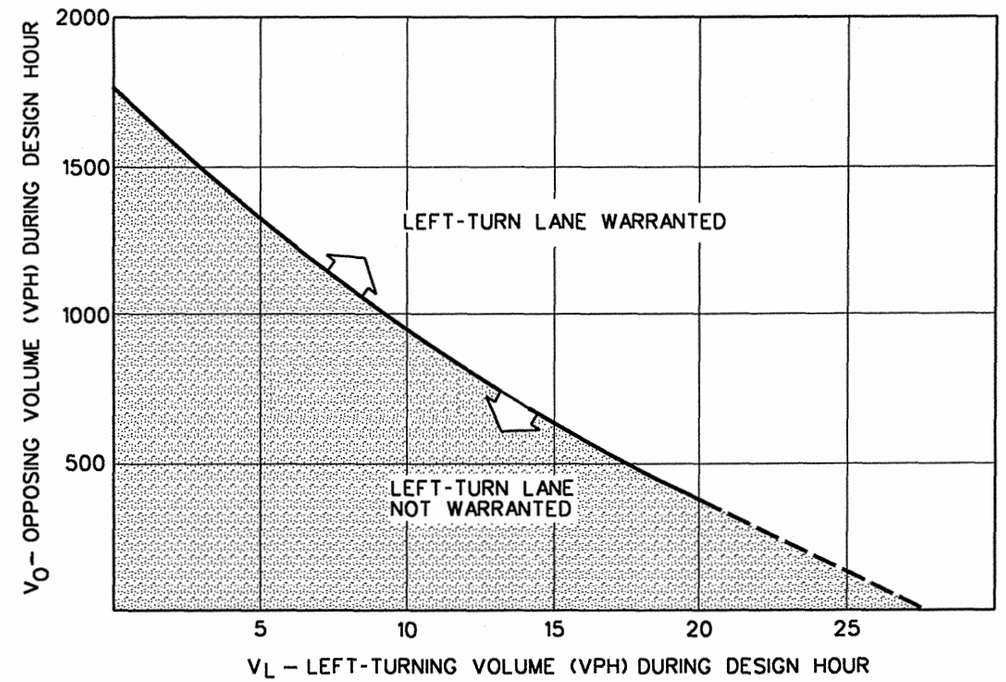
V_A = TOTAL ADVANCING TRAFFIC VOLUME INCLUDING ALL TURNING TRAFFIC

V_O = TOTAL OPPOSING TRAFFIC VOLUME INCLUDING ALL TURNING TRAFFIC

Instructions:

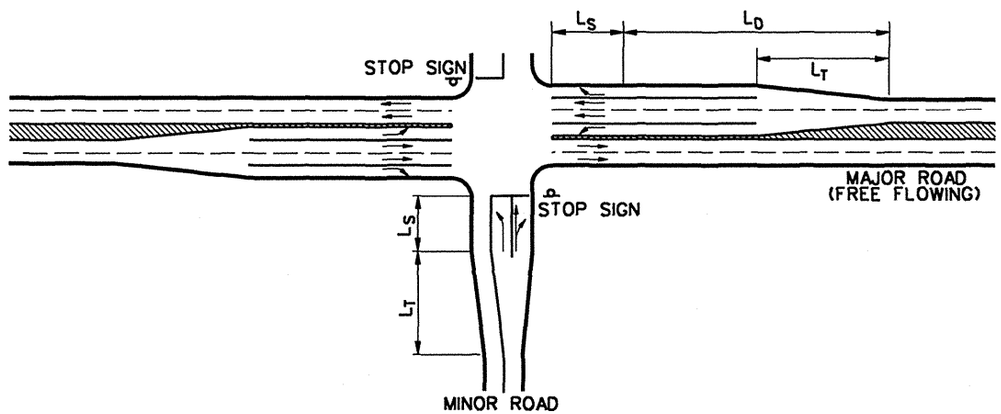
1. *The family of curves represent the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*
2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*
3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME WARRANTS FOR LEFT-TURN LANE AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS (40 mph)
Figure 6-4D



Note: When $V_O < 400$ vph (dashed line), a left-turn lane is not normally warranted unless the advancing volume (V_A) in the same direction as the left-turning traffic exceeds 400 vph ($V_A > 400$ vph).

VOLUME WARRANTS FOR LEFT-TURN LANE ON 4-LANE UNDIVIDED HIGHWAYS
Figure 6-4E



Note: *The schematic of the major road (free flowing) also applies to all legs of a signalized intersection.*

Key: L_T = Taper Length
L_D = Deceleration Length
L_S = Storage Length

TYPICAL AUXILIARY LANES AT AN INTERSECTION
Figure 6-4F

6-4.03.02 Application of Deceleration Length

Table 6-4A presents criteria for the application of the deceleration length for turning lanes at an intersection. The intersection must add the appropriate storage length (L_S) to the deceleration length (L_D) to obtain the full length for design of the turning lane. The minimum lengths are more appropriate on facilities with low traffic volumes, in restricted areas or on 3R projects.

Table 6-4A

RECOMMENDED LENGTHS OF TURNING LANES FOR DECELERATION (L_D)

Highway Design Speed (mph)	L_D (ft)	
	Desirable	Minimum
30	235	120
40	315	165
50	435	265
55	480	310
60	530	370
70	615	465

Notes:

1. *The desirable condition assumes that deceleration starts as the vehicle enters the taper. The initial speed is equal to the average running speed.*

2. *The minimum condition assumes that the vehicle has decelerated to 5 mph below the average running speed in the through lane before entering the taper.*

3. *L_D values from the table include the taper length (L_T).*

4. *L_D values from the table do <u>not</u> include the storage length (L_S).*

5. *The L_D component of the turning lane only applies to signalized intersections (except the truncated leg of a T-intersection) and the free-flowing legs of a stop-controlled intersection.*

6-4.03.03 Storage Length

The storage length (L_S) for turning lanes should be sufficient to store the number of vehicles likely to accumulate in the design hour. The storage length is determined by the type of traffic control governing the turning movement and the requirement for bypassing stopped vehicles. Recommended storage length criteria are as follows:

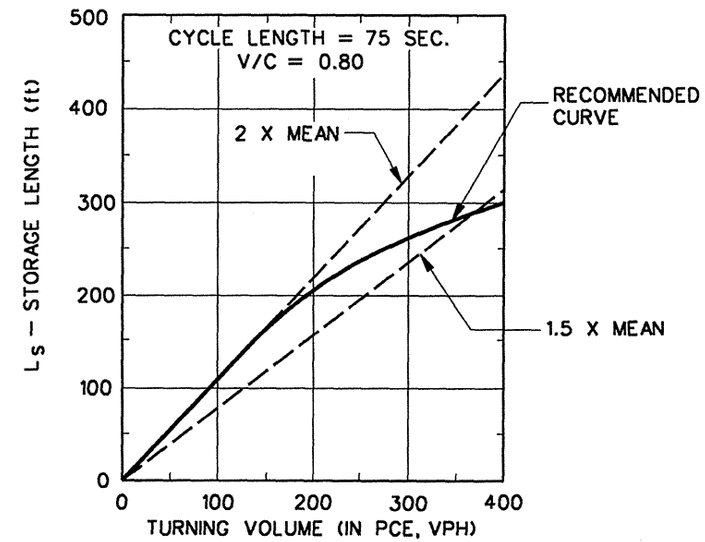
1. Signalized Intersections. The designer should consider the following in determining the recommended storage length for signalized intersections:

   a. The storage length should be based on 1.5 (minimum) to 2.0 (desirable) times the average number of vehicles that would store per cycle during the design hour.

   b. Figure 6-4G illustrates an alternative method to determine the recommended storage length for left-turn lanes, or right-turn lanes where right-turn-on-red is prohibited, at a signalized intersection when the *v/c* ratio is known. The values obtained from the figure are for a cycle length of 75 seconds and a *v/c* ratio of 0.80. For other values, the designer should multiply the length obtained in the figure by an adjustment factor found in the accompanying table with Figure 6-4G. The *v/c* ratio is determined by a capacity analysis as described in the *Highway Capacity Manual.*

   c. Where right-turns-on-red are permitted or where separate right-turn signal phases are provided, the length of the right-turn lane may be reduced due to less accumulation of turning vehicles.

2. Unsignalized Intersections. The storage length should be the number of turning vehicles likely to arrive in an average two-minute period within the design hour. Table 6-4B provides recommended storage lengths for right and left turns at an unsignalized intersection.



Storage Length Adjustment Factors

v/c RATIO, *X*	CYCLE LENGTH, *C* (SEC)				
	60	70	80	90	100
0.50	0.70	0.76	0.84	0.89	0.94
0.55	0.71	0.77	0.85	0.90	0.95
0.60	0.73	0.79	0.87	0.92	0.97
0.65	0.75	0.81	0.89	0.94	1.00
0.70	0.77	0.84	0.92	0.98	1.03
0.75	0.82	0.88	0.98	1.03	1.09
0.80	0.88	0.95	1.05	1.11	1.17
0.85	0.99	1.06	1.18	1.24	1.31
0.90	1.17	1.26	1.40	1.48	1.56
0.95	1.61	1.74	1.92	2.03	2.14

Notes: 1. *Figure applies to exclusive left-turn lanes and exclusive right-turn lanes where right-turns- on-red are not allowed.*

2. *See minimum storage length discussion in Section 6-4.03.03.*

3. *To determine the v/c ratio and the passenger car equivalent (PCE) values, see the Highway Capacity Manual.*

RECOMMENDED STORAGE LENGTH FOR SIGNALIZED INTERSECTIONS

Figure 6-4G

Table 6-4B

RECOMMENDED STORAGE LENGTHS (L_s) FOR UNSIGNALIZED INTERSECTIONS

DHV (vph)	L_S (ft)
≤ 60	50 - 75
61 - 120	100
121 - 180	150
> 180	200 or greater

Note: See minimum storage length discussion in Section 6-4.03.03.

3. Minimum Length. The minimum storage length for turning lanes should be sufficient to store at least two vehicles. Where the truck traffic is 10 percent or less, 50 ft is the minimum storage length. Where the truck traffic is greater than 10 percent, 100 ft is the minimum. However, for right-turn lanes at unsignalized intersections in low-volume areas, the driver will likely be able to turn without delay. In these cases, the storage length may be assumed to be zero. At signalized intersections with the right-turn-on-red allowance, the minimum storage length of a right-turn lane is 25 ft.

Example 6-4.2 illustrates the use of Table 6-4A and Figure 6-4G to determine the total length of a left-turn lane at a signalized intersection.

Example 6-4.2

Given: Signalized 4-leg intersection
Left-turn volume = 150 vph (DHV)
Cycle Length = 80 sec; v/c ratio = 0.70
Design speed of highway = 40 mph

Problem: Determine the length of a left-turn lane

Solution: The total length of the turn lane including taper equals $L_D + L_S$.

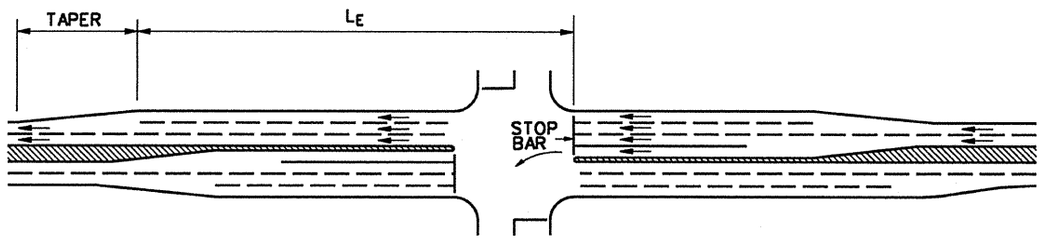
From Table 6-4A (40 mph), L_D = 165 ft (including taper).

The minimum L_S is based on 1.5 x mean, or L_S = 120 ft unadjusted. From Figure 6-4G, the storage length adjustment factor = 0.92. Therefore, L_S = 0.92 (120) = 110 ft.

$L_D + L_S$ = 165 ft + 100 ft = 275 ft

6-4.03.04 Design Length for Extension of Through Lanes

Intersections are sometimes widened to meet the level-of-service criteria, especially at signalized intersections. Any additional through lanes or combination turning and through lanes should be extended beyond the intersection. The distance (L_E) should be sufficient for vehicles to return to the through lanes without affecting the highway operation. Figure 6-4H provides the recommended lengths (L_E) for vehicles to return to the through lane beyond a widened intersection. For through lanes extended beyond the intersection, the recommended taper is 150 ft for design speeds of 45 mph and below and 300 ft for design speeds greater than 45 mph.



Design Speed (mph)	L_E (ft)
30	190
40	380
50	760
55	980
60	1170
70	1590

Notes:

1. *L_E is that distance required by the vehicle to accelerate from a stop to 5 mph below the design speed.*
2. *The taper distance should be 150 ft for design speeds of 45 mph and below and 300 ft for design speeds of 50 mph or greater.*

EXTENSION OF ADDITIONAL THROUGH LANES

Figure 6-4H

6-4.03.05 Additional Design Considerations

The designer should consider several other factors when designing an auxiliary turning lane:

1. Channelized Left-Turn Lane. If a left turn is required on a two-lane highway, it should desirably be designed as a fully channelized left-turn lane. A typical channelized left turn is illustrated in Figure 6-4I. Generally, left-turn deceleration and storage bays will be designed symmetrically about the highway centerline.

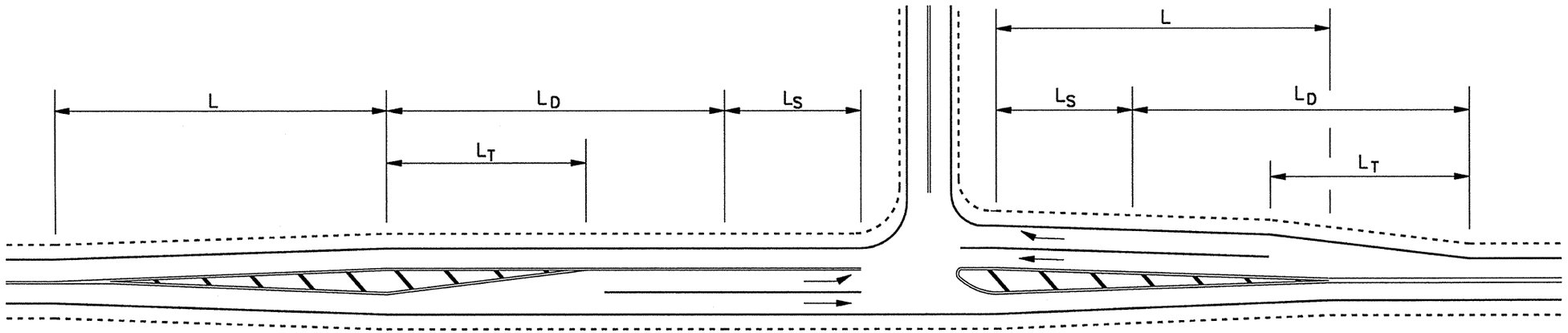
2. Dual-Turn Lanes. Where the proper length of a turn lane cannot be provided or becomes prohibitive, the designer may consider a dual-turn lane. Generally, a dual-turn lane, approximately 50 percent as long as a single-turn lane, will operate comparably. However, to operate properly, the intersecting road must have two lanes of traffic to accept the turning vehicles.

3. Offset Left-Turn Lanes. For medians wider than 18 ft, it is desirable to align the left-turn lane so that it will reduce the width of the divider to 4 ft to 8 ft immediately before the intersection, rather than to align it parallel and adjacent to the through lane. This alignment will place the vehicle waiting to make the turn as far to the left as practical, thus providing improved visibility to the opposing through traffic. The advantages of aligning the left-turn lanes are:

   a. better visibility to opposing through traffic;

   b. decreased probability of a conflict between opposing left-turn movements within the intersection; and

   c. more left-turn vehicles served in a given period of time, especially at a signalized intersection.



The taper distance is calculated from:

$L = WS$ ($S \geq 45$), or $L = WS^2/60$ ($S \leq 40$)

Where:

L = taper length (ft)
W = transition width (ft)
S = design speed (mph)

L_S = storage length (ft)
L_D = deceleration length (ft)
L_T = taper length:
- 150 ft (desirable)
- 75 ft (restricted urban areas)

CHANNELIZED LEFT-TURN LANE FOR TWO-LANE HIGHWAYS

Figure 6-4I

Offset designs may be either a parallel or taper design; see Figure 6-4J. The parallel design may be used at signalized and unsignalized intersections. However, the taper design is primarily only used at signalized intersections. Offset turn lanes should be separated from the adjacent through traveled way by painted or raised channelization.

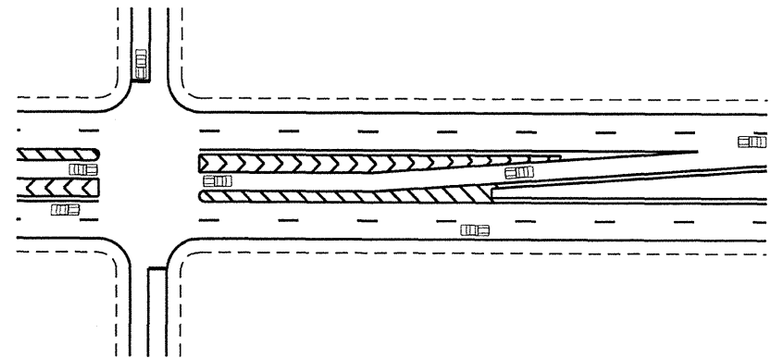
4. Parking Restrictions. A right-turn lane in an urban area may require parking restrictions beyond the usual restricted distances from the intersection as discussed in Sections 6-2.04 and 14-2.06.03.

5. Width. The width of the turn lane should be according to the functional class and urban/rural location of the intersection. Chapter 2 presents criteria for rural highways; Chapter 14 presents criteria for urban streets.

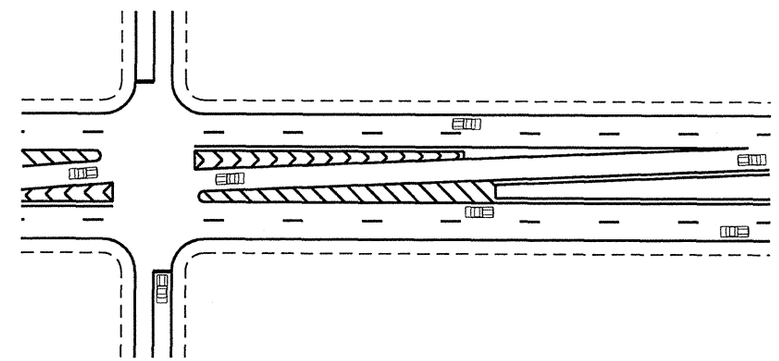
6. Intersection Sight Distance. See Section 6-6.05 for intersection sight distance criteria for a left-turning vehicle.

7. Median Openings. Median openings should be designed according to the criteria in Section 6-8.0.

8. Pavement Markings. Typical development and pavement markings of a left-turn lane are illustrated in Figure 6-4K. The MUTCD should also be used to determine the proper pavement markings.

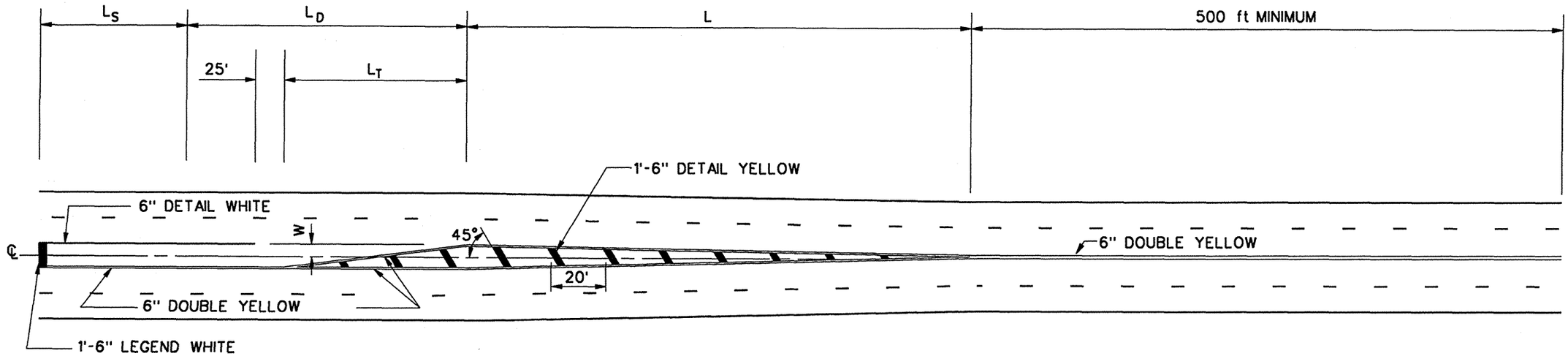


a) PARALLEL-OFFSET TURN LANE



b) TAPERED-OFFSET TURN LANE

OFFSET TURN LANES
Figure 6-4J



L_S = storage length (ft)
L_D = deceleration length (ft)
L_T = taper length:
- 150 ft (desirable)
- 75 ft (restricted urban areas)

$L = WS$ ($S \geq 45$), or $L = WS^2/60$ ($S \leq 40$)

Where:

L = taper length (ft)
W = transition width (ft)
S = design speed (mph)

TYPICAL PAVEMENT MARKINGS
(Left-Turn Lanes)
Figure 6-4K

6-5.0 CONTINUOUS TWO-WAY LEFT-TURN LANES

Continuous two-way left-turn lanes (CTWLTL) are used as a cost-effective method to accommodate a continuous left-turn demand and to reduce delay and accidents. These lanes typically reduce conflicts on roadways which were originally intended to serve the through movement but are now used to serve the demand for accessibility created by changes in adjacent land use. The designer should evaluate the information presented in the following sections when considering the implementation of a CTWLTL.

6-5.01 Guidelines for Application

6-5.01.01 Typical Sites

The typical sites where a CTWLTL should be considered include:

1. areas with a high number of driveways per block,
2. areas of high-density commercial development, and
3. areas with substantial midblock left turns.

The applicability of the CTWLTL is a function of the traffic conditions resulting from the adjacent land use. The designer should evaluate the area to determine the relative benefit of a CTWLTL as compared to alternative access techniques. For example, a CTWLTL may encourage more strip development. If this is not desirable, a raised median is an alternative treatment.

6-5.01.02 Functional Class

An undivided 2-lane or 4-lane urban arterial is the most common type of highway selected for the implementation of a CTWLTL.

6-5.01.03 Traffic Volumes

Through traffic volumes and the capacity of major intersections should be investigated to determine the through requirements at the midblock area of the highway. For application, the design year for the project should be used to determine the traffic volume. As general guidance, the following should be used:

1. On existing 2-lane roadways, a CTWLTL should be advantageous for traffic volumes between 5,000 and 12,500 ADT.

2. On existing 4-lane highways, a CTWLTL should be advantageous for traffic volumes between 10,000 and 25,000 ADT.

3. For traffic volumes greater than 25,000 ADT, a raised median may be a more appropriate treatment than a CTWLTL. The designer will need to consider the extent of development, number of traffic signals, number of driveway entrances, gaps available in the oncoming traffic and the use of dual left-turn lanes at intersections to determine the appropriate selection.

4. Pedestrian crossing volumes are also a consideration because of the large paved area which must be traversed.

6-5.01.04 Design Speed

The design speed on a highway facility is a critical factor for consideration in CTWLTL applications. General criteria dictates that design speeds up to 50 mph will accommodate a CTWLTL. Design speeds higher than 50 mph should be investigated more carefully considering the number of accidents, number of driveways, traffic signals, etc. The designer should also contact the district for further guidance.

6-5.02 Design Criteria

6-5.02.01 Lane Width

Recommended lane widths for a CTWLTL are presented in Table 6-5A.

Table 6-5A

RECOMMENDED LANE WIDTHS FOR A CTWLTL

Desirable	Minimum
14 ft	12 ft

Note: In industrial areas with heavy truck traffic, the desirable lane width is 16 ft.

Existing highways that warrant the installation of a CTWLTL are often located in areas of restricted right-of-way. Conversion of the existing cross section may be difficult; however, narrow lane widths should be avoided except for low traffic volumes. To obtain the necessary lane width, the designer may have to consider several alternatives including:

1. removing an existing raised median,
2. reducing the width of existing through lanes,
3. reducing the number of existing through lanes,
4. eliminating existing parking lanes,
5. eliminating or reducing the width of existing shoulders, and/or
6. acquiring additional right-of-way to expand the pavement width by the amount needed for the CTWLTL.

6-5.02.02 Intersection Treatment

At all intersections with public roads, a CTWLTL must either 1) be terminated in advance to allow the development of an exclusive left-turn lane, or 2) be extended up to the intersection area. The preferred treatment is to terminate the CTWLTL and provide an exclusive left-turn lane. Consideration may be given to extending the CTWLTL up to the intersection under the following conditions:

1. Signals. At signalized intersections, a CTWLTL should be terminated because these intersections will typically warrant an exclusive left-turn lane. At unsignalized intersections, the CTWLTL may be extended through the intersection because an exclusive left-turn lane is usually not required.

2. Turning Volumes. The left-turn demand into the intersecting road is a factor in determining the proper intersection treatment. The following may be used as general guidance: If the minimum storage length will govern (Section 6-4.03.03), then it may be warranted to extend the CTWLTL up to the intersection (i.e., provide no exclusive left-turn lane).

3. Minimum Length of CTWLTL. The CTWLTL should have sufficient length to operate properly, and the intersection treatment will determine the length of the CTWLTL. The minimum length will be influenced by through traffic volumes, turning volumes and operating speeds on the highway. The following guidance may be used:

a. On facilities with lower speeds and/or lower traffic volumes, the minimum length of a CTWLTL should be 500 ft.

b. On facilities with higher speeds and/or higher traffic volumes, the minimum length of a CTWLTL should be 1000 ft.

4. Operational/Safety Factors. Extending the CTWLTL up to an intersection could result in operational or safety problems. Some drivers may, for example, pass through the intersection in the CTWLTL and turn left beyond the intersection into a driveway. If operational or safety problems are known or anticipated at an intersection, this is a factor in determining the proper intersection treatment.

6-5.02.03 Traffic Control

A CTWLTL requires proper signing and marking to reduce indecision and misuse. Criteria for signing and marking have been developed and are presented in the MUTCD and in Figure 9-9C.

6-6.0 INTERSECTION SIGHT DISTANCE (ISD)

6-6.01 General

At each intersection the potential exists for vehicles to conflict with one another when entering, exiting or crossing the intersection. The designer should provide sufficient sight distance for a driver to perceive these potential conflicts and to perform the necessary actions needed to negotiate the intersection safely.

Because all intersections on State highways are either stop controlled or signalized, no guidelines are provided for no control or yield-controlled intersections. For these types of intersections, the designer is referred to the AASHTO *A Policy on Geometric Design of Highways and Streets* for guidance.

6-6.02 Background

6-6.02.01 Basic Criteria

The Department uses gap acceptance as the conceptual basis for its intersection sight distance (ISD) criteria at stop-controlled intersections. The intersection sight distance is obtained by providing clear sight triangles both to the right and left as shown in Figure 6-6A. The length of legs of these sight triangles are determined as followings:

1. Minor Road. The length of leg along the minor road is based on two parts. The first is the location of the driver's eye on the minor road. This is typically assumed to be 15 ft from the edge of traveled way for the major road and in the center of the lane on the minor road; see Figure 6-6A. The second part is based on the distance to the center of the vehicle on the major road. For right-turning vehicles, this is assumed to be the center of the closest travel lane from the left. For left-turning vehicles, this is assumed to be the center of the closest travel lane for vehicles approaching from the right. See Figure 6-6A.

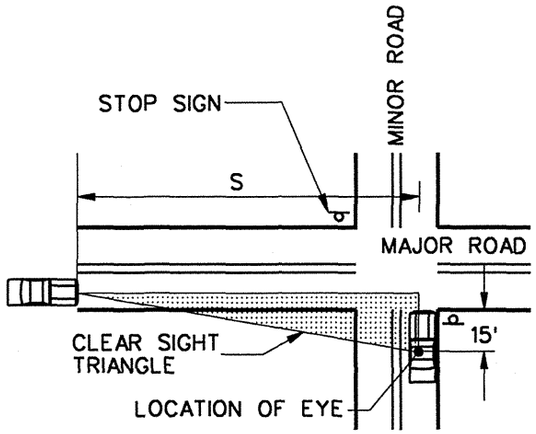
2. Major Road. The length of the sight triangle leg or ISD along the major road is determined using the following equation:

$$S = 1.467\, V_{major}\, t_c \qquad \text{(Equation 6-6.1)}$$

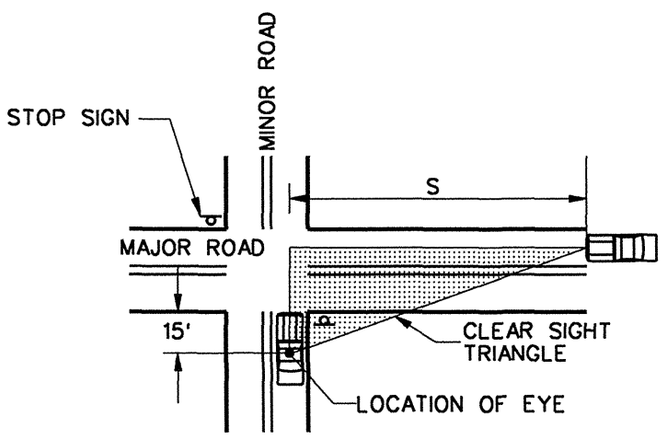
where:

S = *length of sight triangle leg along major road (ft)*
V_{major} = *design speed of major road (mph)*
t_c = *critical gap for entering the major road (sec)*

The critical gap time (t_c) varies according to the design vehicle, the grade on the minor road approach, the number of lanes on the major roadway, the type of operation and the intersection skew. Section 6-6.07 presents several examples on the application of ISD.



CLEAR SIGHT TRIANGLE FOR VIEWING TRAFFIC APPROACHING FROM THE LEFT



CLEAR SIGHT TRIANGLE FOR VIEWING TRAFFIC APPROACHING FROM THE RIGHT

CLEAR SIGHT TRIANGLES (STOP-CONTROLLED) INTERSECTIONS
Figure 6-6A

Within this clear sight triangle, if practical, the objective is to remove or lower any object that obstructs the driver's view. These objects may include buildings, parked or turning vehicles, trees, hedges, tall crops, unmowed grass, fences, retaining walls and the existing ground line. In addition, where an interchange ramp intersects the major road or crossroad near a bridge on a crest vertical curve, objects such as bridge parapets, piers, abutments or the crest vertical curve itself may restrict the clear sight triangle.

6-6.02.02 Sight Flare Criteria

In addition to the ISD criteria based on gap acceptance, the driver on the major roadway should have sufficient sight distance to see vehicles approaching the stop sign to avoid a collision if the approaching vehicle violates the traffic control. The sight flare criteria apply to intersections which exceed the Department's threshold traffic volumes. See Section 6-6.06.

6-6.03 Application (Stop-Controlled Intersections)

Where 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 assuming that the approaching vehicle comes into view as the stopped vehicle begins it departure. The following sections discuss the application of the Department's ISD methodology at stop-controlled intersections.

6-6.03.01 Vehicle Entering Major Roadway

To determine the intersection sight distance for vehicles turning left or right onto the major road, the designer should use Equation 6-6.1 and the critical gap times (t_c) presented in Table 6-6A. Table 6-6B, which solves Equation 6-6.1, provides the ISD values for all design vehicles on two-lane, level facilities. The designer should also consider the following:

Table 6-6A

GAP ACCEPTANCE TIMES
(Right or Left Turn From Minor Road)

Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger Car	7.5
Single-Unit Truck	9.5
Tractor/Semitrailer	11.5

Table 6-6B

TWO-LANE INTERSECTION SIGHT DISTANCES
(Right or Left Turn from Minor Road)

Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
20	225	280	340
25	280	350	425
30	335	420	510
35	390	490	595
40	445	560	675
45	500	630	760
50	555	700	845
55	610	770	930
60	665	840	1015
65	720	910	1100
70	775	980	1185

Note: These ISD values assume a minor road approach grade ≤ +3%.

1. Turn Maneuver. There is only a minimal difference in the gap acceptance times between the left- and right-turning drivers. Therefore, only one critical gap time is provided.

2. Multilane Facilities. For multilane facilities, the gap acceptance times presented in Table 6-6A should be adjusted to account for the additional distance required by the turning vehicle to cross the additional lanes or median. The following will apply:

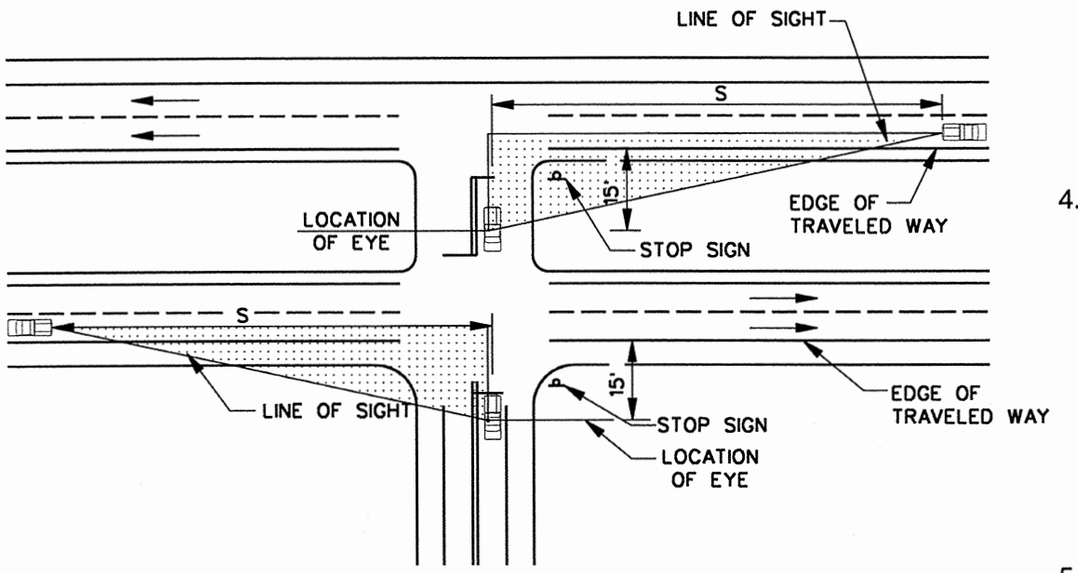
   a. Left-Turns. For left turns onto multilane highways, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, in excess of one, to be crossed by the turning vehicle. Assume that the left-turning driver will enter the left travel lane on the far side of the major road. For example, the gap acceptance time for a passenger car turning left onto an undivided six-lane facility would be 7.5 seconds plus 0.5 seconds for each of the two additional lanes needed to be crossed. The total gap time required is therefore 8.5 seconds.

   b. Right Turns. Because the turning vehicle is assumed to be turning into the nearest right through lane, no adjustments to the gap times are required.

   c. Medians. Depending on the median width, additional time may need to be added to the base gap time; see Comment #3 below.

3. Medians. For a multilane facility which does not have a median wide enough to store a stopped vehicle, divide the median width by 12 ft to determine the corresponding number of lanes, and then use the criteria in Comment #2a above to determine the appropriate time factor.

   On multilane facilities with a median wide enough to store a stopped vehicle, the designer should evaluate the move in two steps (see Figure 6-6B):



INTERSECTION SIGHT DISTANCE
(Divided Facilities)
Figure 6-6B

a. First, with the vehicle stopped on the minor road (the bottom portion in Figure 6-6B), use the gap acceptance times and distances for a vehicle turning right (Tables 6-6A and 6-6B) to determine the applicable ISD. Under some circumstances, it may be necessary to check the crossing maneuver to determine if it is the critical movement. Crossing criteria are discussed in Section 6-6.03.02.

b. Then, with the vehicle stopped in the median (top portion in Figure 6-6B), assume a two-lane roadway design and use the gap acceptance times and distances for vehicles turning left (Tables 6-6A and 6-6B) to determine the applicable ISD.

4. Approach Grades. If the approach grade on the minor road exceeds +3%, add the following times to the basic gap acceptance times in Table 6-6A:

a. Left Turns. Multiply the percent grade on the approach by 0.2 and add this to the base time gap.

b. Right Turns. Multiply the percent grade on the approach by 0.1 and add this to the base time gap.

Use the adjusted t_c in Equation 6-6.1 to determine S.

5. Trucks. At some intersections (e.g., near truck stops, interchange ramps, grain elevators), the designer may want to use the truck as the design vehicle for determining the ISD. The gap acceptance times (t_c) for single-unit and tractor/ semitrailer trucks are provided in Table 6-6A. ISD values for level, two-lane roadways are presented in Table 6-6B.

6. Height of Eye/Object. The height of eye for passenger cars is assumed to be 3.5 ft above the surface of the minor road. The height of object (approaching vehicle on the major road) is also assumed to be 3.5 ft. An object height of 3.5 ft assumes that a sufficient portion of the oncoming vehicle must be visible to identify it as an object of concern by the minor road driver. If there is a

sufficient number of trucks to warrant their consideration, assume an eye height of 8 ft for a tractor/semitrailer and 6 ft for single-unit trucks and buses. If a truck is the assumed entering vehicle, the object height will be 3.5 ft for the passenger car on the major road.

7. Skew. At skewed intersections where the intersection angle is less than 60°, adjustments may need to be made to account for the extra distance the vehicle needs to travel across opposing lanes. Using the procedures discussed in Comment #2 in Section 6-6.03.01 and/or Section 6-6.03.02, determine the appropriate ISD value based on this extra travel distance.

8. Examples. For examples on the application of ISD, see Section 6-6.07.

6-6.03.02 Straight Through Crossing Vehicle

In the majority of cases, the intersection sight distance for turning vehicles typically will provide adequate sight distance to allow a vehicle to cross the major road. However, in the following situations, the crossing sight distance may be the more critical movement:

1. where left and/or right turns are not permitted from a specific approach and the crossing maneuver is the only legal or expected movement (e.g., indirect left turns);

2. where the design vehicle must cross more than six travel lanes or, with medians, the equivalent distance; or

3. where a substantial volume of heavy vehicles cross the highway and there are steep grades on the minor road approach.

Use Equation 6-6.1 and the gap acceptance times (t_c) and the adjustment factors in Table 6-6C to determine the ISD for crossing maneuvers. Where medians are present, include the median width in the overall length to determine the applicable gap time. Divide this width by 12 ft to determine the corresponding number of lanes for the crossing maneuver.

Table 6-6C

GAP ACCEPTANCE TIMES
(Crossing Maneuvers)

Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger Car	6.5
Single-Unit Truck	8.5
Tractor/Semitrailer	10.5

Adjustments:

1. *Multilane Highway. Where the design vehicle is crossing a major road with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane in excess of two. See the discussion in Section 6-6.03.02 for additional guidance.*

2. *Approach Grade. If the approach grade on the minor road exceeds +3%, multiply the percent grade of the minor road approach by 0.2 and add it to the base gap acceptance time.*

6-6.03.03 All-Way Stop

At intersections with all-way stop control, provide sufficient sight distance so that the first stopped vehicle on each approach is visible to all other approaches. The ISD criteria for left- or right-turning vehicles as discussed in Section 6-6.03.01 are not applicable in this situation. Often, intersections are converted to all-way stop control to address limited sight distance at the intersection. Therefore, providing additional sight at the intersection is unnecessary.

6-6.04 Signal-Controlled Intersections

At signalized intersections, provide sufficient sight distance so that the first vehicle on each approach is visible to all other approaches. Traffic signals may be used at high-volume intersections to address accidents related to restricted sight distances. Therefore, the ISD criteria for left- or right-turning vehicles as discussed in Section 6-6.03.01 is typically not applicable at signalized intersections. However, where right-turn-on-red is allowed, ensure that the ISD as presented in Section 6-6.03.01 for a stop-controlled, right-turning vehicle is available to the left. If not, this may warrant restricting the right-turn-on-red movement. In addition, if the traffic signal is placed on two-way flash operation during off-peak or nighttime conditions, provide the ISD criteria as discussed in Section 6-6.03.01 for a stop-controlled intersection.

6-6.05 Stopped Vehicle Turning Left

At all intersections, regardless of the type of traffic control, the designer should consider the sight distance needs for a stopped vehicle turning left from the major road. This is illustrated in Figure 6-6C. The driver must see straight ahead for a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection. In general, if the major highway has been designed to meet the stopping sight distance criteria, intersection sight distance only will be a concern where the major road is on a horizontal curve, where there is a median, or where there are opposing vehicles making left turns at an intersection.

Use Equation 6-6.1 and the gap acceptance times (t_c) from Table 6-6D to determine the applicable intersection sight distances for the left-turning vehicle. Where the crossing vehicle must cross more than one lane, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane in excess of one. Where medians are present, the designer will need to consider their effect in the same manner as discussed in Section 6-6.03. Table 6-6E provides the ISD values for all design vehicles and two common left-turning situations.

Table 6-6D

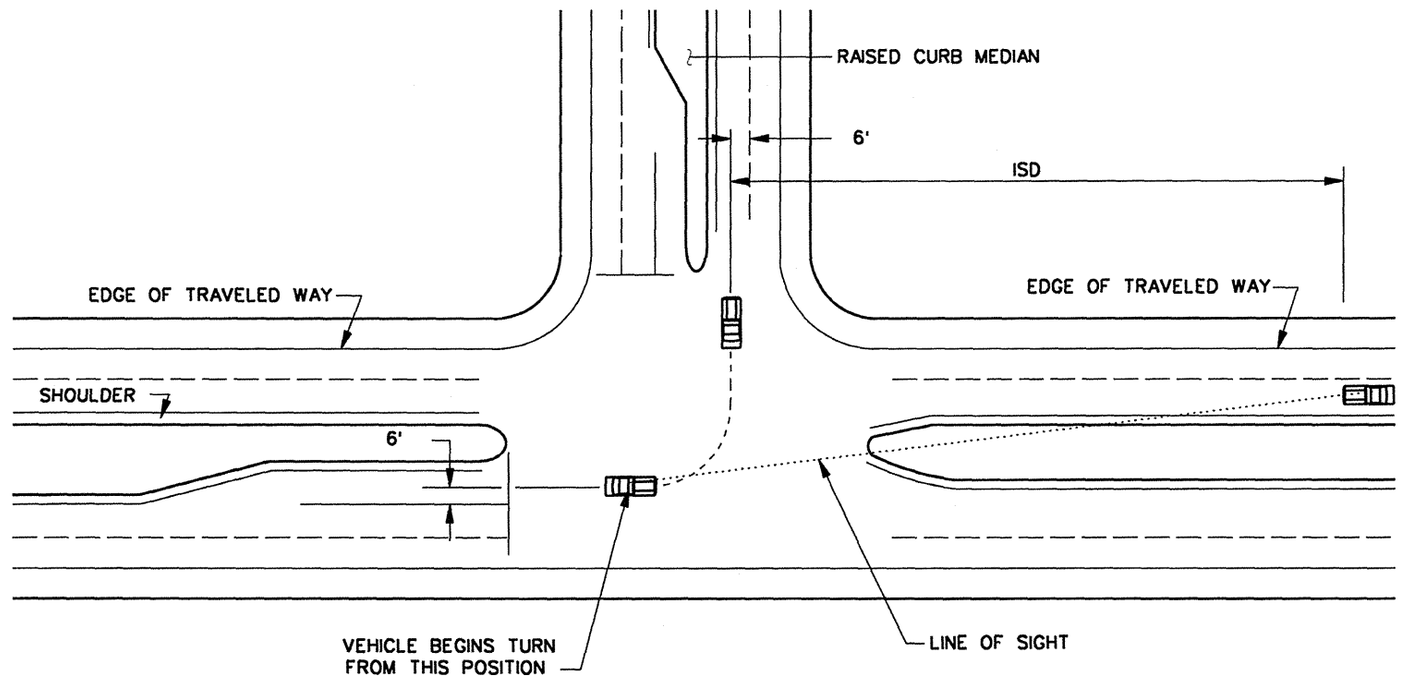
GAP ACCEPTANCE TIMES
(Left-Turning Vehicles from Major Road)

Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger Car	5.5
Single-Unit Truck	6.5
Tractor/Semitrailer	7.5

Table 6-6E

INTERSECTION SIGHT DISTANCES
(Left-Turning Vehicles from Major Road))

Design Speed (V_{major}) (mph)	ISD (ft)					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes
20	165	180	195	215	220	240
25	205	220	240	265	275	300
30	245	265	290	320	330	360
35	285	310	335	370	385	425
40	325	335	385	425	440	485
45	365	400	430	480	495	545
50	405	440	480	530	550	605
55	445	485	525	585	605	665
60	485	530	575	635	660	725
65	525	575	620	690	715	785
70	565	620	670	740	770	845



Note:

1. *See Table 6-6E for ISD values.*
2. *See Section 6-6.05 for discussion and application.*

INTERSECTION SIGHT DISTANCE FOR A STOPPED VEHICLE TURNING LEFT
(On Major Road)

Figure 6-6C

6-6.06 Sight Flare Criteria

6-6.06.01 General

Figure 6-6D illustrates the sight flare criteria for undivided and divided highways. These criteria apply to intersections where the DHV of the major highway exceeds 300 and the current ADT of the intersecting roadway exceeds 300. These criteria will determine a sight triangle which should be free of all sight obstructions. These include buildings, large trees, etc. Right-of-way at the intersection should be acquired to avoid the future introduction of sight obstructions. However, at existing intersections it may be impractical to remove existing obstructions within the sight triangle, especially in urban areas.

6-6.06.02 Urban Signalized Intersections

At urban signalized intersections, a sight flare is desirable but not required. The designer should consider the following:

1. Improvements. Do not show a sight flare in any quadrants that contain some type of improvement (e.g., house, gas station). However, include a normal construction flare on the plans if there is no right-of-way restriction. The construction flare will be labeled *No Access*. It is desirable to extend the *No Access* along the right-of-way line to the end of the taper where the site investigation determines this is feasible.

2. No Improvements. Show a sight flare on the plans for any quadrants that do not contain improvements. The construction flare will be labeled *No Access* and *(Design Speed) Sight Flare*; see Figure 6-6D.

6-6.07 Examples of ISD Applications

The following three examples illustrate the application of the ISD criteria.

Example 6-6.1

Given: Minor road intersects a 4-lane highway with a CTWLTL.
Minor road is stop controlled.
Design speed of the major highway is 50 mph.
All travel lane widths are 12 ft.
The CTWLTL width is 14 ft.
Trucks are not a concern.

Problem: Determine the intersection sight distance to the left and right from the minor road.

Solution: The following steps will apply:

1. For the vehicle turning right, the ISD to the left can be determined directly from Table 6-6B. For the 50 mph design speed, the ISD to the left is 555 ft.

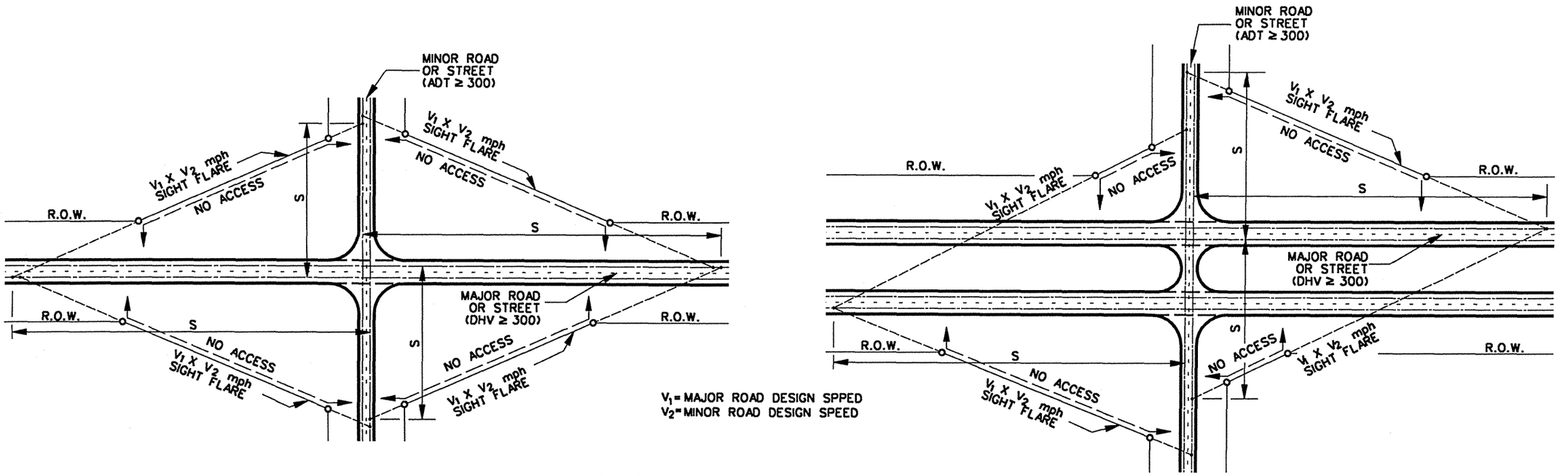
2. For the vehicle turning left, the ISD must reflect the additional time required to cross the additional lanes; see Comment #2 in Section 6-6.03.01. The following will apply:

   a. First determine the extra width required by the one additional travel lane and the CTWLTL and divide this number by 12 ft:

   $$\frac{(12 + 14)}{12} = 2.2 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required:

   (2.2 lanes)(0.5 sec/lane) = 1.1 seconds



Design Speed (mph)	S (ft)
30	185
40	275
50	380
55	435
60	505
65	550
70	615

Note: Design speed for sight flare must be shown on the plans (e.g., 60 x 40 mph SIGHT FLARE*).*

SIGHT FLARE CRITERIA AT INTERSECTIONS
Figure 6-6D

c. Add the additional time to the basic gap time of 7.5 seconds and insert this value into Equation 6-6.1:

$$S = (1.467)(50)(7.5 + 1.1) = 631 \text{ ft}$$

Provide an ISD of 635 ft to the right for the left-turning vehicle.

3. Check the crossing vehicle, as discussed in Section 6-6.03.02. The following will apply:

a. First determine the extra width required by the two additional travel lanes and the CTWLTL and divide this number by 12 ft:

$$\frac{(12 + 12 + 14)}{12} = 3.2 \text{ lanes}$$

b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required:

$$(3.2 \text{ lanes})(0.5 \text{ sec/lane}) = 1.6 \text{ seconds}$$

c. Add the additional time to the basic gap time of 6.5 seconds and insert this value into Equation 6-6.1:

$$S = (1.467)(50)(6.5 + 1.6) = 594 \text{ ft}$$

The 594 ft for the crossing maneuver is less than the 635 ft required for the left-turning vehicle and, therefore, is not the critical maneuver.

Example 6-6.2

Given: Minor road intersects a 4-lane divided highway.
Minor road is stop controlled.
Design speed of the major highway is 60 mph.
All travel lane widths are 12 ft.
The median width is 125 ft.
Trucks are not a concern.

Problem: Determine the intersection sight distance to the left and right from the minor road.

Solution:

1. For the vehicle turning right, the ISD to the left can be determined directly from Table 6-6B. For the 60 mph design speed, the ISD to the left is 665 ft.

2. Determine if the crossing maneuver is critical; see Section 6-6.03.02. No adjustments are required to the base time of 6.5 seconds. Therefore, use Equation 6-6.1 directly:

$$S = (1.467)(60)(6.5) = 572 \text{ ft}$$

The crossing maneuver is less than the right-turning maneuver and, therefore, is not critical.

3. For the vehicle turning left, assume the passenger car is stopped in the median; see Figure 6-6B. The ISD to the right can be determined directly from Table 6-6B. For the 60 mph design speed, the ISD to the left is 665 ft. The crossing manuever will not be critical.

Example 6-6.3

Given: Minor road intersects a 2-lane highway.
Minor road is stop controlled.
Design speed of the major highway is 60 mph.
All travel lane widths are 12 ft.

The approach grade on the minor road is 4.5%.
Tractor/semitrailer trucks are a concern.

Problem: Determine the intersection sight distance to the left and right from the minor road.

Solution: The following steps will apply:

1. For the left-turning vehicle, the base gap acceptance time from Table 6-6A is 11.5 seconds. Add the additional time due to the approach grade (0.2 seconds per percent grade) to the base gap time; see Comment #4 in Section 6-6.03.01:

   (0.2)(4.5) + 11.5 = 12.4 seconds

   Then, using Equation 6-6.1:

   S = (1.467)(60)(12.4) = 1091 ft

2. The ISD for the right-turning vehicle is determined similarly:

   (0.1)(4.5) + 11.5 = 12 seconds

   Then, using Equation 6-6.1:

   S = (1.467)(60)(12.0) = 1056 ft

3. The crossing manuever will not be critical.

6-7.0 CHANNELIZATION

6-7.01 Type

Most at-grade intersections with low-volume roads will be simple, non-channelized intersections. As the significance of the intersecting road increases, there may be a need for various channelization treatments. Note that turning roadways are a part of channelized intersections. Table 6-7A presents general criteria which may be used to make an initial determination for channelization needs. The designer must evaluate each intersection to determine the applicability of the criteria in Table 6-7A.

Table 6-7A

CHANNELIZATION WARRANTS

Current ADT (Crossroad)	Channelization Treatment	
	Crossroad	Main Highway
Under 300	No Channelization	Left-turn lanes on 4-lane divided only
300-1000	No Channelization	Left-turn lanes on all 4-lane facilities only
	Simple Channelization	Left-turn lanes
*1000-2000	Channelize with left-turn lanes	Left-turn lanes
	Channelize with left-turn lanes	Left-turn lanes and right-turn lanes
Over 2000	Conduct signal warrant analysis; Conduct capacity analysis.	

** Turning movement count is required at all intersections where crossroad ADT is 1000 or more.*

Note: See Section 6-4.02 for left-turn warrants.

At intersections where the approaching traffic volumes are approximately equal and a signal is not warranted, it may be warranted to install all-way stop control. The MUTCD presents minimum traffic volume warrants for all-way stop control intersections.

Additionally, the designer should use the methodology in Chapter 10 of the *Highway Capacity Manual* to determine if the all-way stop can accommodate the intersection traffic volumes at the desired level of service. Note that all-way stops should not be used where either roadway is a divided facility.

6-7.02 Design of Islands

The edges of traffic channels at intersections are formed and delineated by traffic islands. The design of traffic islands should consider the site-specific functions, including definition of vehicular paths, separation of traffic movements, prohibition of movements, protection of pedestrians and placement of traffic control devices. Islands can be grouped into the following functional classes. Most islands serve two or all of these functions:

1. Directional Islands. Control and direct traffic movements and should guide the driver into the proper channel.

2. Divisional Islands. Separate opposing traffic flows, alert the driver to the crossroad ahead and regulate traffic through the intersection. These islands are often introduced at intersections on undivided highways and are particularly advantageous in controlling left turns at skewed intersections and in preventing wrong-way turns into right-turning traffic lanes.

3. Refuge Islands. Refuge islands at or near crosswalks aid or protect pedestrians crossing a wide roadway. These islands may be required for pedestrians at intersections where complex signal phasings are used. Refuge islands may also serve as areas for the installation of traffic control devices.

The designer should adhere to the criteria in the following sections when designing islands. The designer should also refer to the MUTCD for additional design guidance.

6-7.02.01 Types of Islands

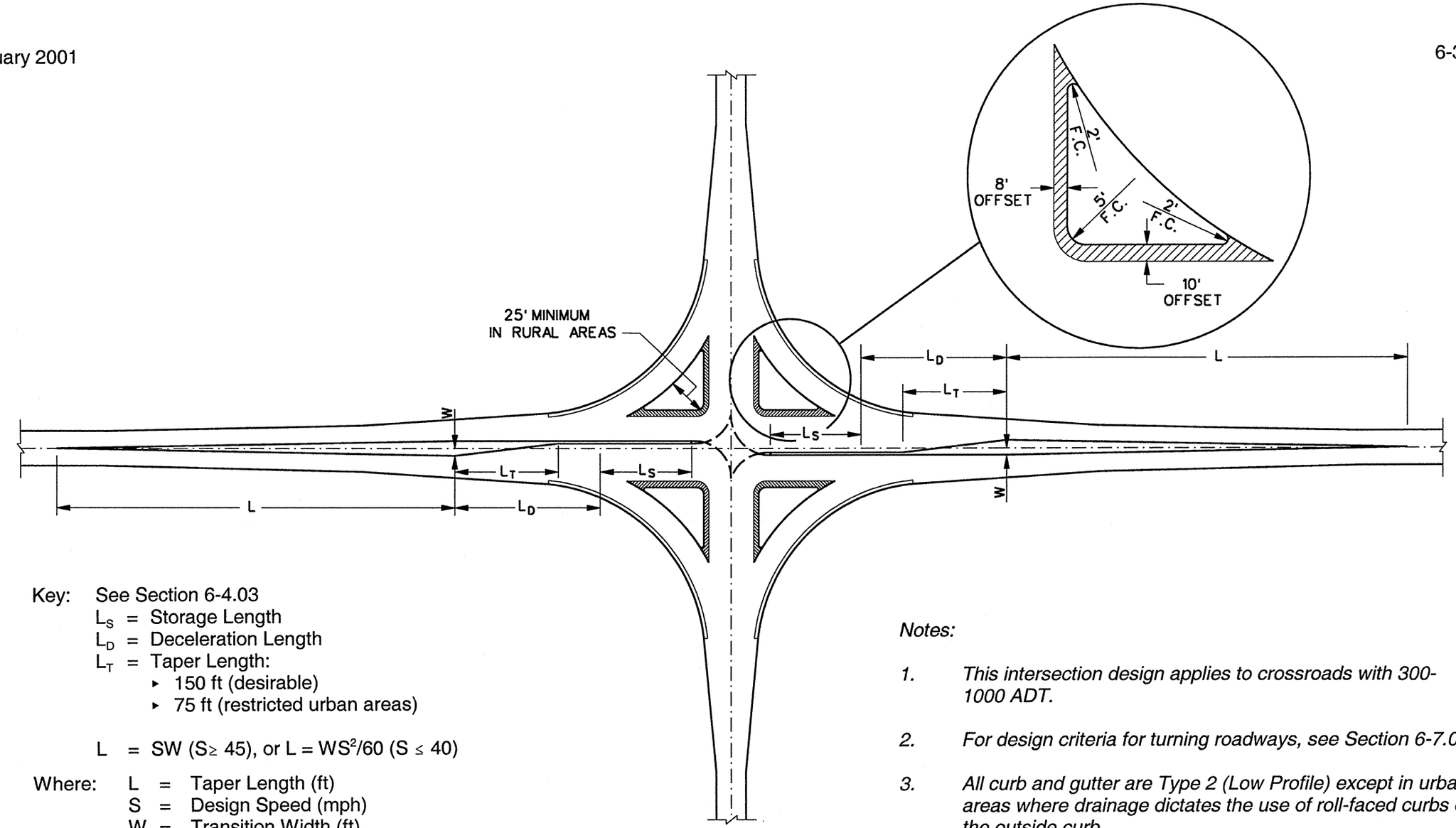
Traffic islands may be flush or raised, paved or turf, and triangular or elongated. Raised islands formed by curbs should be used where pedestrian traffic is significant and where traffic control devices are needed within the island. Type 2 (Low Profile) Curb and Gutter should be used in most cases. However, traffic islands with barrier curbs are acceptable where the design speed is 45 mph or less. It may be desirable to provide lighting where raised islands are used. Flush (painted) islands are appropriate in urban areas where speeds are low and space is limited.

6-7.02.02 Minimum Size

Traffic islands should be large enough to command the driver's attention. Island shapes and sizes vary from one intersection to another. For triangular islands, the minimum size is 50 ft^2 at urban intersections and a 25-ft width at rural intersections (see Figure 6-7A). Elongated islands should be not less than 3 ft wide and 20 ft long. In restricted areas, an elongated island may be 2 ft wide. Divisional islands on high-speed facilities should be at least 100 ft in length.

6-7.02.03 Delineation

Traffic islands should be delineated based on their size, location and function. Raised islands outlined by curbs can be applied universally; this is the most positive means of delineation. Islands delineated by mountable curb, pavement markings, rumble bars or buttons should be used in urban areas where space is limited or on highways with higher design speeds. Non-paved areas formed by pavement edges, supplemented by delineators or by earth treatment adjacent to the pavement, should be used in rural areas where there is sufficient space.



Key: See Section 6-4.03

L_S = Storage Length

L_D = Deceleration Length

L_T = Taper Length:

- 150 ft (desirable)
- 75 ft (restricted urban areas)

L = SW (S≥ 45), or $L = WS^2/60$ (S ≤ 40)

Where: L = Taper Length (ft)

S = Design Speed (mph)

W = Transition Width (ft)

Notes:

1. *This intersection design applies to crossroads with 300-1000 ADT.*
2. *For design criteria for turning roadways, see Section 6-7.03.*
3. *All curb and gutter are Type 2 (Low Profile) except in urban areas where drainage dictates the use of roll-faced curbs on the outside curb.*

TYPICAL LAYOUT
(Channelized Intersection)

Figure 6-7A

6-7.02.04 Approach Treatment

Raised islands with mountable curbs are acceptable where hazardous conditions exist. Approach and trailing corners of curbed islands should be constructed with 2-ft radii measured to the face of curb. The corner radius should be 5 ft to the face of curb, see Figure 6-7A. For good delineation, pavement markings, roughened pavement or rumble strips should be placed in advance of the island approach to warn the driver. This is especially important at the approach to median curbed islands in the direct line of approaching traffic.

6-7.02.05 Offsets

Traffic islands should be offset from the edge of the traveled way. The offset for major roads will be 10 ft and for minor roads 8 ft. This distance is measured from the edge of the traveled way to the face of curb. No additional offsets are required for auxiliary lanes.

6-7.03 Turning Roadways

6-7.03.01 Guidelines

The designer should consider providing a turning roadway where:

1. it is desirable to allow right turns at speeds greater than 10 mph,
2. the angle of turn is greater than 110°,
3. the volume of right turns is high or the turning movement is from a high-volume roadway,
4. the selected design vehicle is a semi-trailer combination, or
5. there is a need to provide a pedestrian refuge area.

The following sections discuss the design details of a turning roadway.

6-7.03.02 Curvature

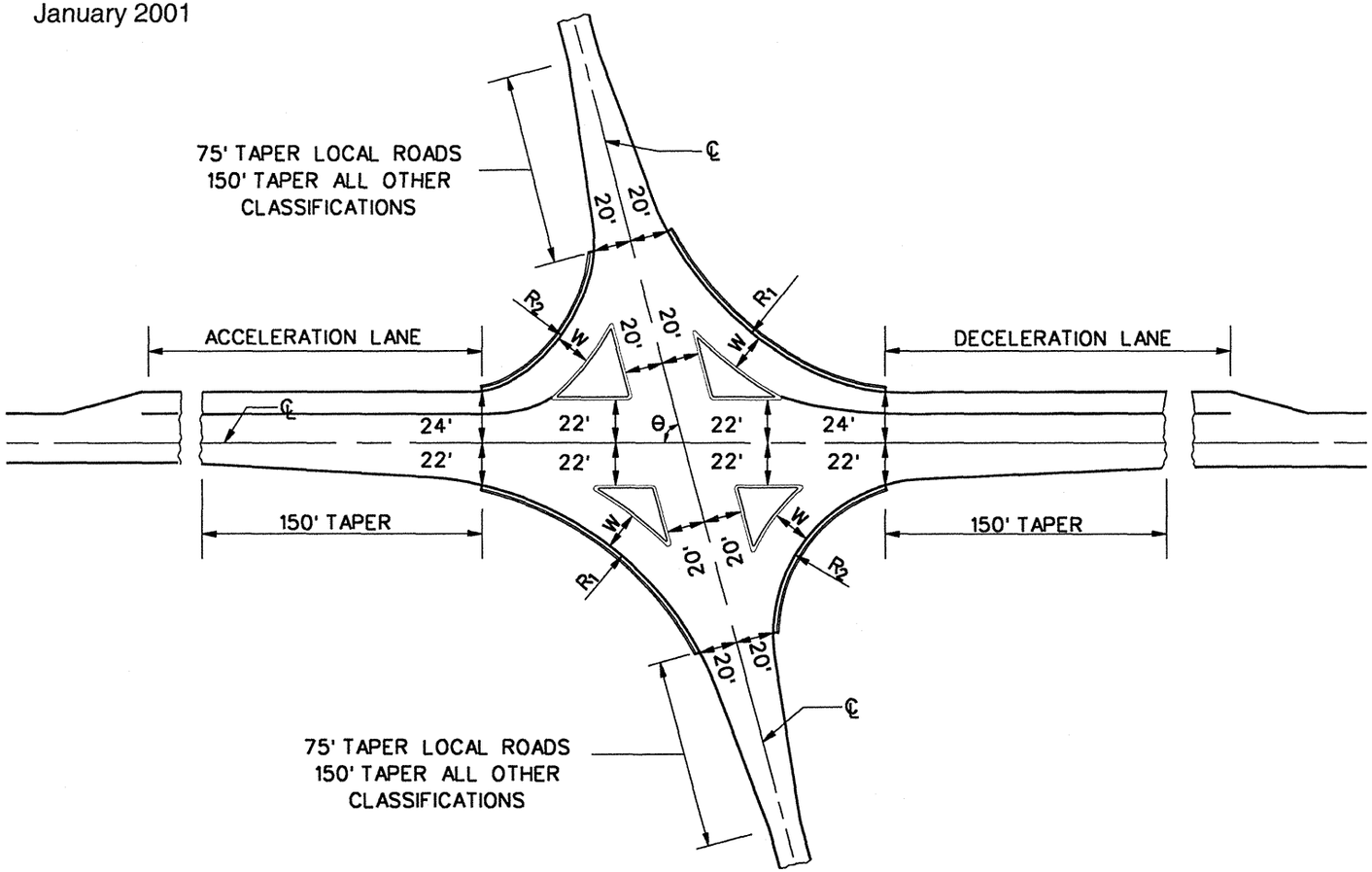
The radii of a turning roadway should be coordinated with the controlling geometry of the intersection. Figure 6-7B illustrates a typical example of a turning roadway intersection using a simple curve with a taper offset. In the figure, as the angle of intersection (θ) decreases, the turning radius (R_1) increases and the turning radius (R_2) decreases. Proper design radii can be determined by entering the chart at the intersecting angle on the ordinate and reading the appropriate design radii from the curves labeled R_1 and R_2.

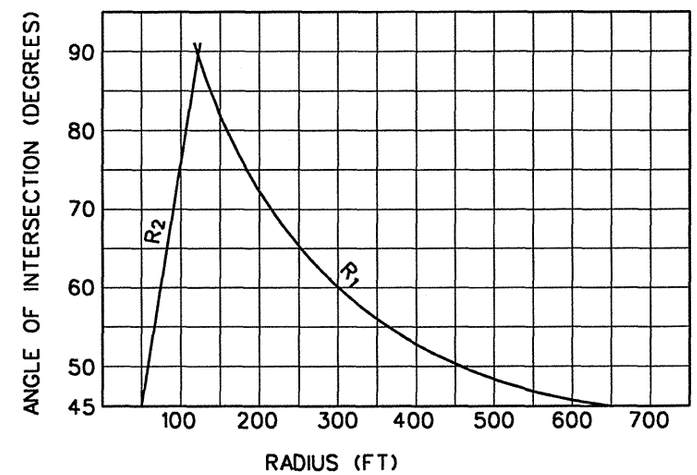
6-7.03.03 Deceleration and Acceleration Lanes

In the design and application of deceleration and acceleration lanes, the designer should consider the following:

1. Warrants. Consider the following guidelines:

   a. Deceleration Lanes. Deceleration lanes provide the driver with a length of roadway to decelerate to a lower design speed before entering the turning roadway. This will remove the decelerating vehicle from the mainline. Factors which will determine the warrant for a deceleration lane include mainline design speed, turning roadway design speed, urban/rural location, type of traffic control, through volumes, turning volumes, number of trucks and two-lane/multi-lane facility. Table 6-7B provides guidelines for deceleration lane warrants at turning roadways based on traffic volumes. If the combination of mainline volumes and turning volumes exceeds these criteria during the design hour, a deceleration lane may be warranted.





Note: Where $\theta = 90°$, $R_1 = R_2 =$ 115 ft to 130 ft.

Notes:

1. *R_1 is always the larger angle of turn.*
   R_2 is always the smaller angle of turn.
   R_1 and R_2 are measured to the edge of the turning roadway.

2. *W = Width of turning roadway. See Section 6-7.03.05.*

Relation Between Intersecting Angle and Radii (Based on 100-ft taper on local road, 150-ft taper on main line and maintaining a 25-ft minimum island width).

TURNING ROADWAY INTERSECTIONS
Figure 6-7B

Table 6-7B

GUIDELINES FOR DECELERATION LANES AT TURNING ROADWAYS

Total vph in One Direction	Right-Turning vph
<200	(Not Warranted)
200 - 300	120
300 - 400	100
400 - 500	85
500 - 600	70
>600	50

Example 6-7.1

Given: DHV = 350 vph
Right Turns = 105 vph

Problem: Determine if a deceleration lane to the turning roadway is warranted.

Solution: According to Table 6-7B, the right-turning volume threshold is 100 vph where the 1-way DHV is 350. Therefore, with a turning volume of 105 vph, a deceleration lane to the turning roadway may be warranted. Section 6-7.03.03 lists other factors that should be considered.

b. Acceleration Lanes. Acceleration lanes provide a driver with a length of roadway to accelerate to the design speed of the mainline and safely merge into the traffic flow. However, acceleration lanes from turning roadways may not be properly used by many drivers. They should not be used at unchannelized intersections. At channelized intersections, the designer should determine the use of acceleration lanes on a case-by-case basis considering traffic volumes, highway speed, grades, sight distance, urban/rural location, type of traffic control, etc.

2. Design. Once warranted, a deceleration or acceleration lane for a turning roadway should provide sufficient length for a driver to comfortably reduce speed or accelerate without interference to the through traffic of the intersection. Recommended design lengths of deceleration and acceleration lanes for turning roadways are provided in Table 6-7C. The width of these lanes should be equal to the width of the adjacent through lane.

6-7.03.04 Cross-Slope Rollover

The cross-slope rollover of a turning roadway is the algebraic difference between the cross slopes of the through lane and the turning roadway. The maximum algebraic difference in pavement cross slope for various design speeds is provided in Table 6-7D.

6-7.03.05 Width of Roadway

Widths of turning roadways are determined primarily by the selected deign vehicle and the radius of the controlling curve. Table 6-7E provides the recommended design widths of pavement for turning roadways.

Table 6-7C

RECOMMENDED LENGTHS OF ACCELERATION OR DECELERATION LANES FOR TURNING ROADWAYS

Design Speed of Approaching Highway (mph)	Design Speed of Turning Roadway (mph)						
	Stop Condition	15	20	25	30	35	40
	Deceleration Length (ft)						
30	235	185	160	140			
40	315	295	265	235	185	155	
50	435	405	385	355	315	285	225
55	480	445	430	400	375	335	275
60	530	500	490	460	430	410	340
65	570	540	530	490	480	430	380
70	615	590	570	550	510	490	430
	Acceleration Length (ft)						
30	190						
40	380	320	250	220	140		
50	760	700	630	580	500	380	160
60	1170	1120	1070	1000	910	800	590
70	1590	1540	1500	1410	1330	1230	1010

Notes:

1. *The deceleration or acceleration occurs both within the taper and the full-width lane up to the PC or PCC of the controlling curve (i.e., the lengths in the table will include the taper length).*

2. *The recommended taper length for deceleration lanes is 150 ft. However, in restricted urban areas, this may be reduced to 75 ft.*

3. *The recommended taper length for acceleration lanes are 150 ft for highway design speeds of 40 mph and below and 300 ft for design speeds greater than 40 mph.*

Table 6-7D

MAXIMUM ALGEBRAIC DIFFERENCE IN PAVEMENT CROSS SLOPE AT TURNING ROADWAYS

Design Speed of Turning Roadway (mph)	Maximum Algebraic Difference in Cross Slope (percent)
≤ 20	5 - 8
25 - 30	5 - 6
≥ 35	4 - 5

Table 6-7E

RECOMMENDED DESIGN WIDTHS OF PAVEMENT FOR TURNING ROADWAYS

Radius on Inner Edge of Pavement R(ft)	Width of Turning Roadways (ft)				
	P	SU	WB-40	WB-50	WB-62
50	13	18	23	26	26
75	13	17	19	22	25
100	13	16	18	21	23
150	12	16	17	19	21
200	12	16	16	17	19
300	12	15	16	17	18
400	12	15	16	16	17
500	12	15	15	16	17
Tangent	12	15	15	15	15

Notes:

1. *If barrier curb is used on one side, then a curb offset of 1 ft should be added to the table value.*

2. *If barrier curb is used on both sides, then a curb offset of 2 ft (1 ft on each side) should be added to the table value.*

3. *For selection of the applicable design vehicle, see Tables 6-2A and 6-2B.*

6-8.0 MEDIAN OPENINGS

6-8.01 Warrants/Spacing

6-8.01.01 Freeways

Emergency crossovers on rural freeways are normally provided where interchange spacing exceeds 5 miles to avoid long travel distances for emergency law enforcement and maintenance vehicles. Between interchanges, emergency crossovers are spaced at 3 miles to 4 miles intervals. Maintenance crossovers may be required at one or both ends of interchange facilities, depending on interchange type, to facilitate maintenance operations. Do not locate maintenance or emergency crossovers closer than 1500 ft to the end of an exit or entrance taper of a ramp or to any structure. See Figure 6-8A. Crossovers should only be located where above-minimum stopping sight distance is provided and, preferably, should not be located within curves requiring superelevation.

6-8.01.02 Non-Freeways

Median openings are provided on all divided highways with partial control of access (Type 2) or control by regulation (Type 3) provided that the openings are sufficiently spaced. Department policy for median opening spacing, measured from center of opening to center of opening, is as follows:

1. Urban Areas. Space median openings no closer than 880 ft apart.
2. Rural Areas. Space median openings no closer than 1760 ft apart.

Generally, median openings on divided facilities are provided for all existing public streets and highways, provided the median openings are not spaced closer than the above criteria. Waiver of these minimum distances can be made only where existing streets or highways with existing traffic patterns of a prominent degree must cross the median divider. Median openings provided for public streets and highways then become control points for determining spacing and location of intermediate median openings. The maximum number of equally spaced intermediate openings should be provided between the control points.

In addition to the Department's policy, the following applies to median opening spacings on non-freeways:

1. Interchanges. Do not place the first median opening any closer than 850 ft from the tip of taper closest to the centerline of the median opening. See Figure 6-8B.

2. Signal Coordination. Median openings (both signalized and unsignalized) must not impair the traffic signal coordination of the overall facility.

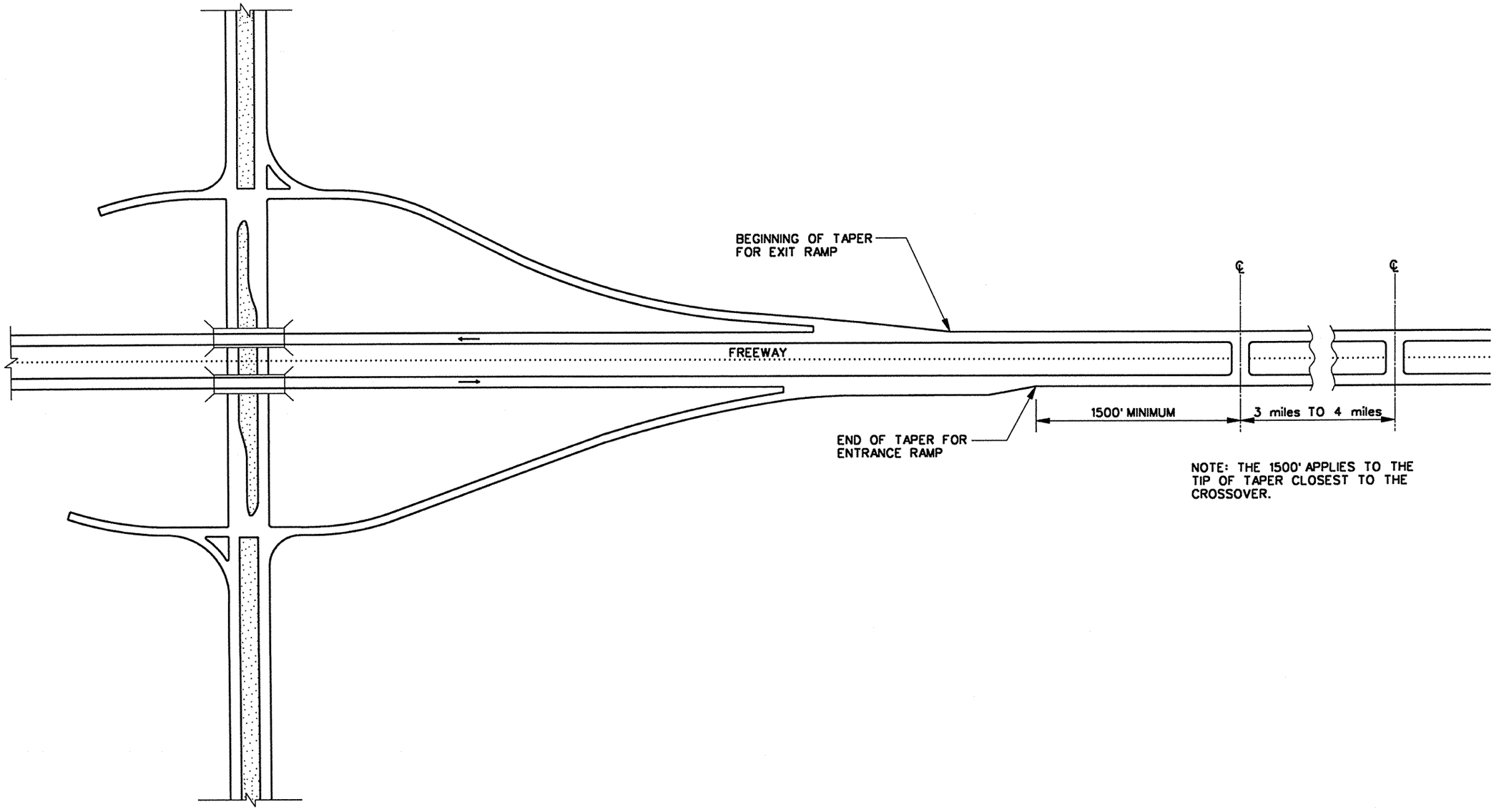
3. Geometrics. The location of median openings should, where practical, be based on favorable geometrics. For example, they should not be located in areas of restrictive sight distance (e.g., on a horizontal curve or near the apex of a crest vertical curve).

6-8.02 Design

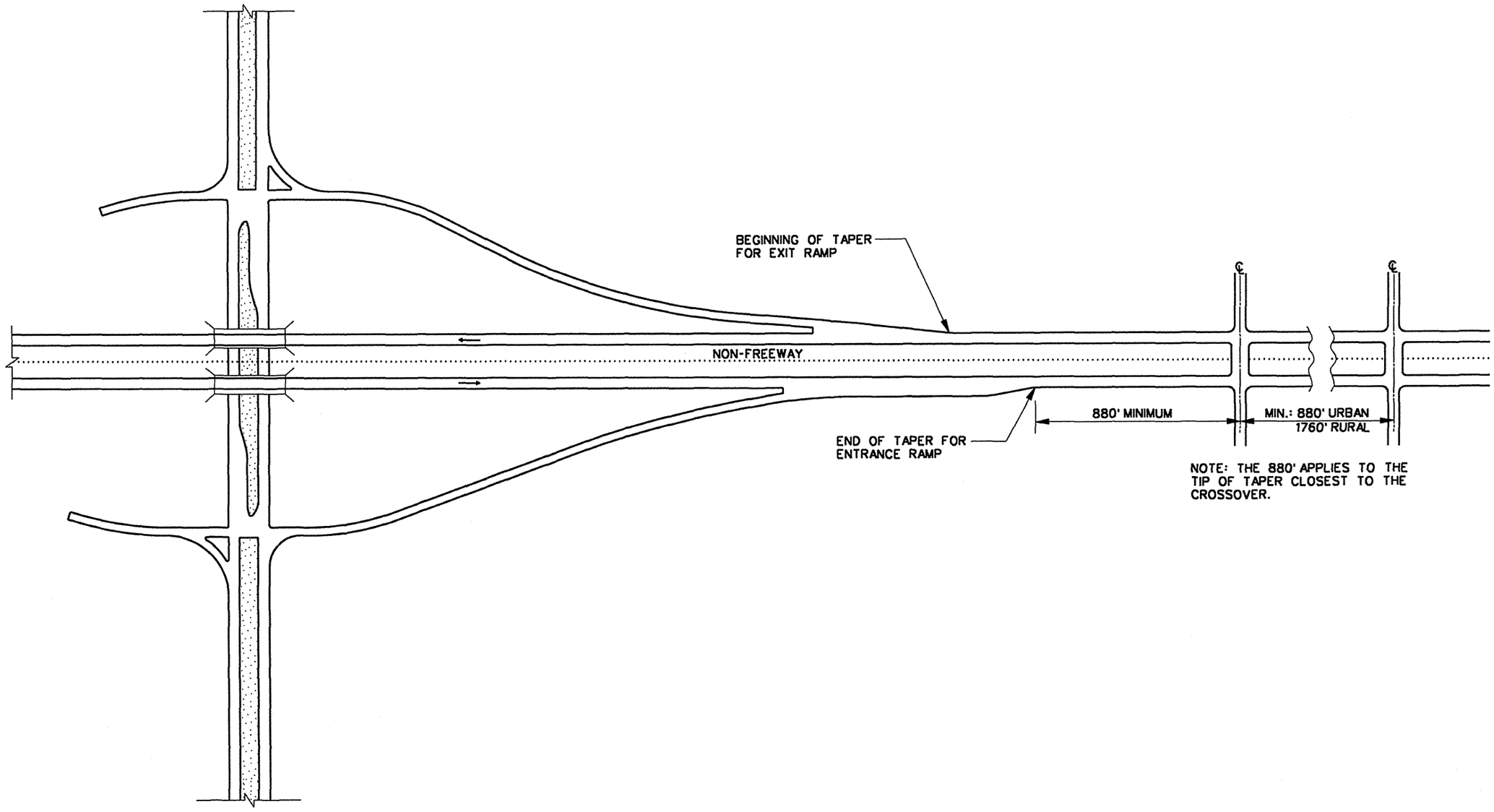
Figures 6-8C, 6-8D, 6-8E and 6-8F provide the Department's design criteria for median openings for various median widths.

6-8.02.01 Turning Radii

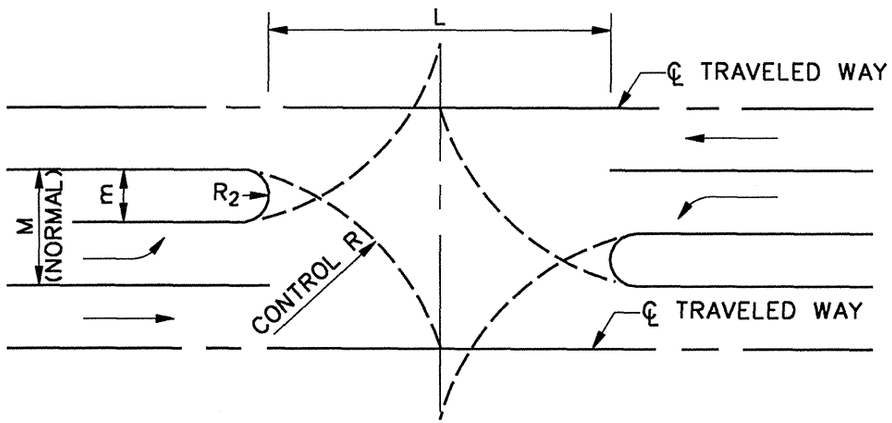
Median openings should be designed to properly accommodate left-turning vehicles, which follow essentially the same path as vehicles making a right turn. Turning radii for the intersection layout are based on the selection of the design vehicle, the turning characteristics of the design vehicle, the acceptable encroachment and the angle of turn. The discussion in Section 6-2.0 on these factors also applies to median openings.



LOCATIONS OF MAINTENANCE/EMERGENCY CROSSOVERS ON FREEWAYS
Figure 6-8A



LOCATIONS OF MEDIAN OPENINGS ON NON-FREEWAYS
Figure 6-8B



m (ft)	L (ft)	
	SU	WB-40
4	96	146
6	94	144
8	92	142
10	90	140

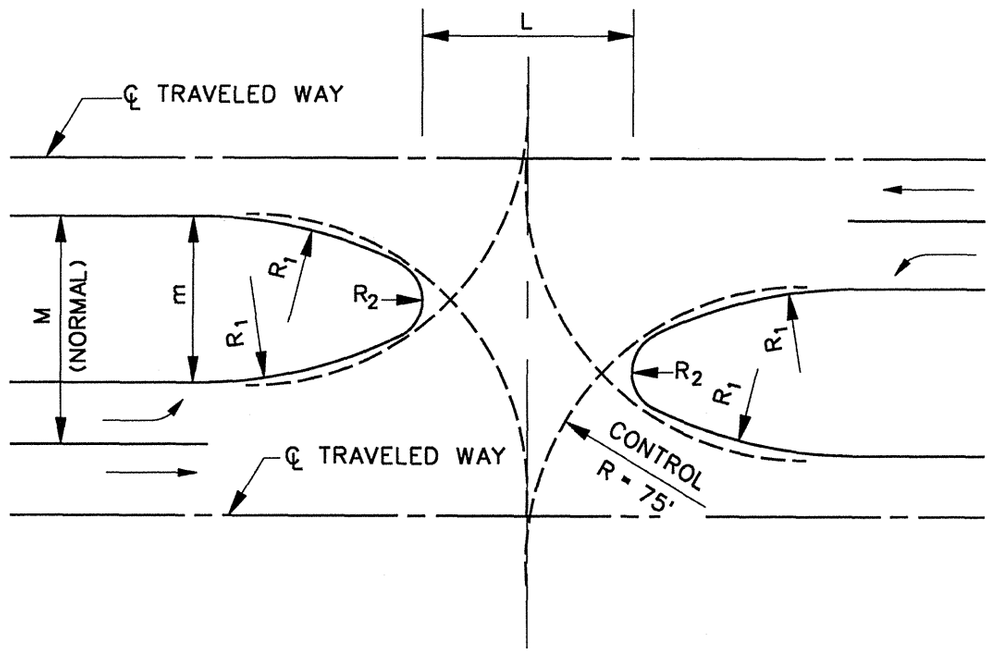
Key: M = median width measured between the two edges of the inside travel lanes.

m = width of divider (raised or flush) remaining after the widths of the left-turn lane (if any) and/or shoulders have been subtracted from the median width, M.

Notes:

1. *According to Table 6-8A, the semicircular end is typically used when m < 10 ft.*
2. *Control R is dependent on the turning radii of the design vehicle selected.*
3. *$R_2 = m/2$*
4. *Typically the SU design vehicle is used for these median openings.*
5. *These suggested criteria should only be used as starting points for the design of the median opening. See the discussion in Section 6-8.02.03 for more information on the design of the opening.*

MEDIAN OPENINGS
(Semicircular End)
Figure 6-8C



m (ft)	L (ft)
10	105
12	100
14	96
16	92
20	85
24	78
28	73
32	67
36	62
40	57
≥54	40

Key: M = median width measured between the two edges of the inside travel lanes.

m = width of divider (raised or flush) remaining after the widths of the left-turn lane (if any) and/or shoulders have been subtracted from the median width, M.

Notes:

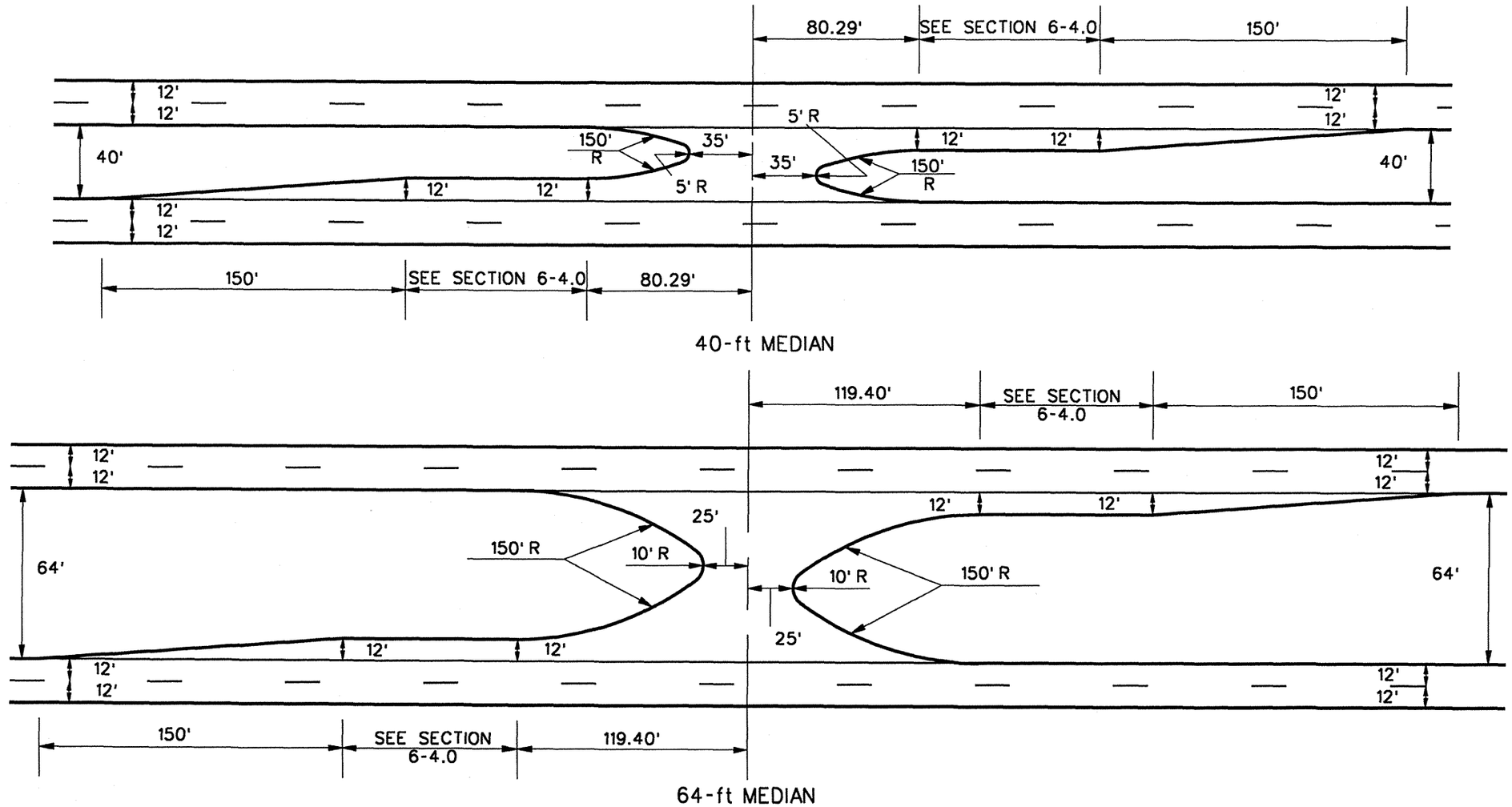
1. *According to Table 6-8A, the bullet-nose is typically used when 10 ft ≤ m ≤ 64 ft.*

2. $R_2 \cong m/5$

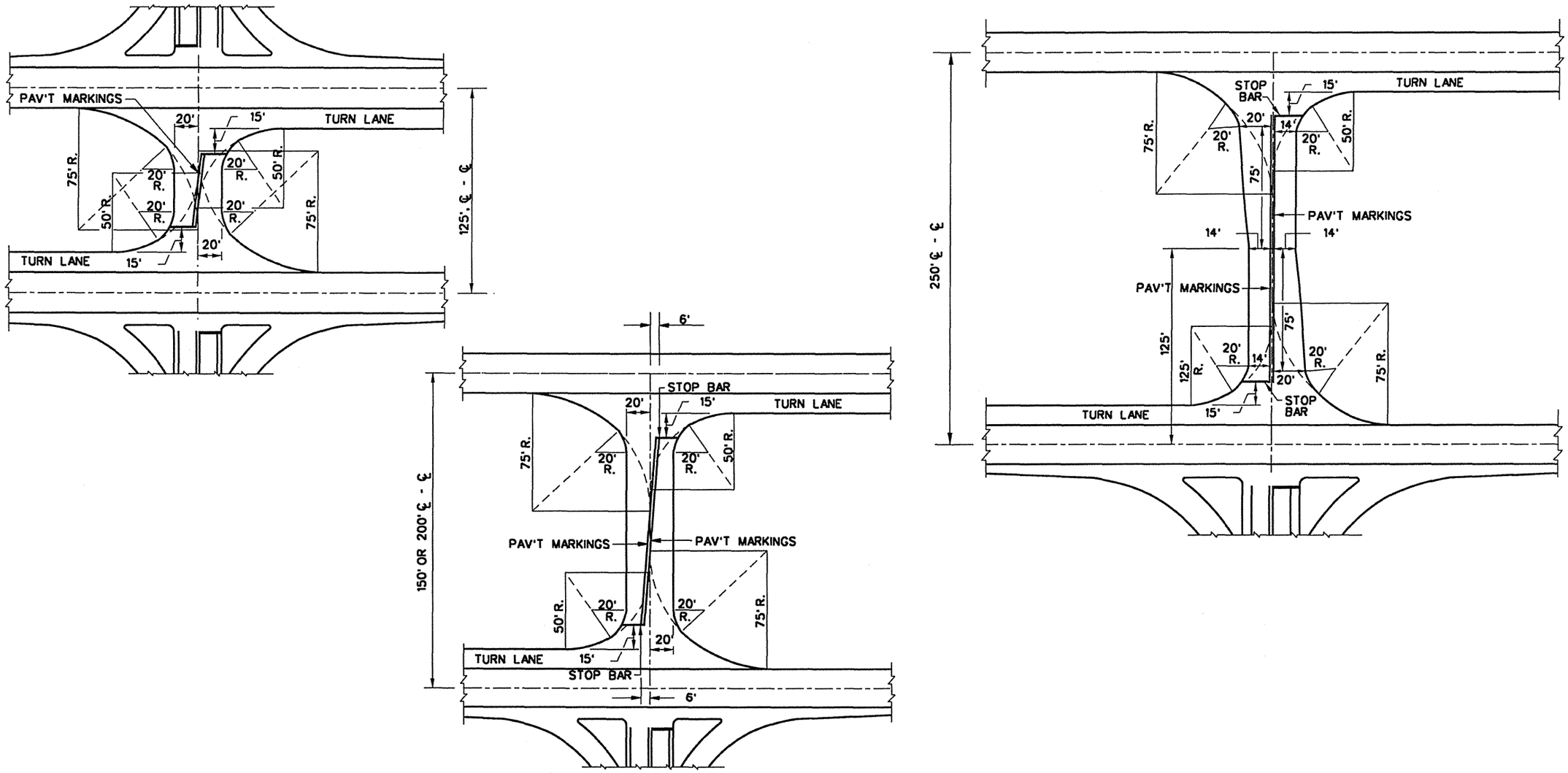
3. *These suggested criteria should only be used as starting points for the design of the median opening. They are predicated on a WB-40 design vehicle. See the discussion in Section 6-8.02.03 for more information on the design of the opening.*

4. $R_1 = 150$ *ft*

MEDIAN OPENINGS
(Bullet-Nose End)
Figure 6-8D



TYPICAL MEDIAN OPENINGS AT INTERSECTIONS
(M ≤ 64 ft)
Figure 6-8E



TYPICAL MEDIAN OPENINGS WITH OR WITHOUT INTERSECTIONS
(M > 64 ft)
Figure 6-8F

The criteria in Figures 6-8D, 6-8E and 6-8F will accommodate all design vehicles. For narrow medians, the criteria in Figure 6-8C should be used to determine the length of median openings (L) for the selected design vehicle. These criteria will allow the vehicle to make the left turn at 5 - 10 mph without encroachment onto the adjacent travel lane.

6-8.02.02 Shape of Median Ends

The shape of median ends at openings depends on the width of the divider (ft) either between the two inside travel lanes or between the left-turn and the opposing inside travel lane. This width is in contrast to the median width (M), which is measured between the edges of the two inside travel lanes and, therefore, includes the width of left-turn lanes, if present.

The most common types of median ends are the semicircular end and the bullet-nose end. Recommended criteria for the selection of the median end shape based on "m" are provided in Table 6-8A.

Figure 6-8C provides recommended design criteria for the semicircular end, and Figure 6-8D presents design criteria for the bullet-nose end. Figure 6-8E provides typical median openings for 40-ft and 64-ft medians. Median openings for median widths greater than 64 ft are treated as separate intersections. Figure 6-8F illustrates typical median openings for M > 64 ft. Designs for median crossovers for M < 64 ft are considered U turns (see Section 6-8.02.04).

6-8.02.03 Length of Opening

The length of a median opening should properly accommodate the turning path of the design vehicle. The length of opening for any divided highway should be equal to or greater than the width of the intersecting road. Recommended lengths of median openings are presented in Figure 6-8C for the semicircular end and Figure 6-8D for the bullet-nose end. They should be used only as a starting point. Each median opening will be evaluated individually to determine its proper design. The designer should consider the following factors in the evaluation:

1. Turning Templates. The designer should check the proposed design with the turning templates for the design vehicles likely to use the intersection. Consideration should be given to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

2. Nose Offset. At 4-leg intersections, traffic passing through the median opening (going straight) will pass the nose of the median end (semicircular or bullet nose). To provide a sense of comfort for these drivers, the offset between the nose end and through travel lane (extended) should be at least 2 ft.

3. Intersection Symmetry. The 3 or 4 legs of the intersection may have various combinations of through travel lanes and turning lanes. The designer should give some consideration to the overall symmetry of the intersection. This, in turn, may affect the design of the median opening.

4. Location of Crosswalks. Desirably, pedestrian crosswalks will intersect the median to provide some refuge for pedestrians. Therefore, the median opening design should be coordinated with the location of crosswalks.

Table 6-8A

SELECTION OF MEDIAN END SHAPE

m (ft)	Median End Shape
m < 10 ft	Semicircular
10 ft ≤ m ≤ 64 ft	Bullet Nose
m > 64 ft	Treated as separate intersections

6-8.02.04 U-turns

As discussed in Section 6-8.01, median openings designed to accommodate U-turns only are sometimes needed on divided highways. Preferably, a vehicle should be able to begin and end the U-turn on the inner lanes next to the median. This design also allows a stopped vehicle to be fully protected within the median area. The sight distance should be adequate to provide for the unexpected U-turn maneuver on an access-controlled highway. Figure 6-8G provides the minimum recommended median widths for U-turn maneuvers and various design vehicles.

6-8.02.05 Skewed Intersections

Preferably median crossovers should not be skewed. Skewed median opening require larger openings which may require special channelization and other adjustment to the left-turn lanes. Each skewed design should be designed separately to determine the appropriate layout.

6-9.0 RURAL DRIVEWAYS

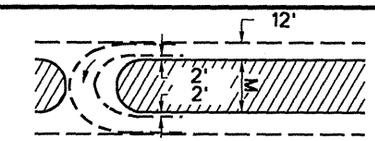
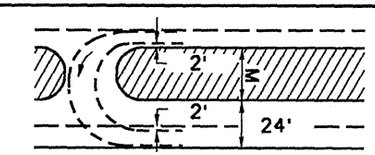
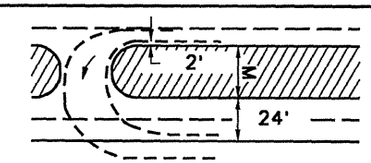
Table 6-9A and Figure 6-9A present design criteria for rural driveways. Section 14-2.07 presents criteria for urban driveways. The *Standard Roadway Design Drawings* present other design details. In addition, several Department SOP's provide information related to driveways:

1. SOP No. MND-03-00-00-000 "Processing of Permit Applications."

2. SOP No. MND-03-03-01-000 "Driveway Connections."

3. SOP No. MND-03-04-01-000 "Construction and Maintenance of Driveway, County Road and Municipal Street Connections to State Highways."

4. SOP No. MND-04-03-01-000 "Driveway and Street Connections, Median Openings, Frontage Roads."

Type of Maneuver		M - Min. Width of Median (ft) for Design Vehicle				
		P	SU	BUS	WB-40	WB-50
		Length of Design Vehicle (ft)				
		19	30	40	50	55
Inner Lane to Inner Lane		30	63	63	61	71
Inner Lane to Outer Lane		18	51	51	49	51
Inner Lane to Shoulder		8	41	41	39	49

MINIMUM DESIGNS FOR U-TURNS

Figure 6-8G

Table 6-9A

RECOMMENDED DESIGN CRITERIA FOR RURAL DRIVEWAYS

Driveway Design Element		Driveway Type	Functional Classification		
			Arterial	Collector	Local Road
Corner Spacing (C)		All	(1)	(1)	(1)
Turning Radii (R) (2)		Residential	10 ft	10 ft	10 ft
		Commercial/Industrial	15 ft - 30 ft	15 ft - 30 ft	15 ft - 30 ft
Width (W) (3)		Residential	16 ft	16 ft	16 ft
		Commercial/Industrial	30 ft - 50 ft	30 ft - 50 ft	30 ft - 50 ft
Maximum Grades	G_1	All	Cut Section: 2% - 4% (down); Fill Section: Shoulder Slope (4% Typical) (down)		
	G_2	Residential	15%		
		Commercial/Industrial	10%		
Change in Grade without Vertical Curve (ΔG)		All	Maximum: 12%	Maximum: 15%	Maximum: 15%
Driveway Side Slopes (4)	Within Clear Zone	All	$V \geq 50$ mph — Des: 10:1, Max: 6:1 $V \leq 45$ mph — Max: 3:1	$V \geq 50$ mph — Des: 10:1, Max: 6:1 $V \leq 45$ mph — Max: 3:1	$V \geq 50$ mph — Des: 10:1, Max: 6:1 $V \leq 45$ mph — Max: 3:1
	Outside Clear Zone		Maximum: 3:1	Maximum: 3:1	Maximum: 3:1
Sight Distance		All	(5)	(5)	(5)

Note: See Figure 6-9A for a definition of terms.

RURAL DRIVEWAYS

Footnotes to Table 6-9A

1. Corner Spacing (C). No part of a driveway entrance or exit is permitted within the turning radius of an intersection (see Section 6-2.0) or within the sight triangles (flares) of an intersection (see Section 6-6.0). If practical, greater distances for corner spacing should be provided.

2. Turning Radii. The radii in the table assume a shoulder width of 2 ft or less. See Section 6-2.0 for a discussion of the effects of wider shoulders on acceptable turning radii.

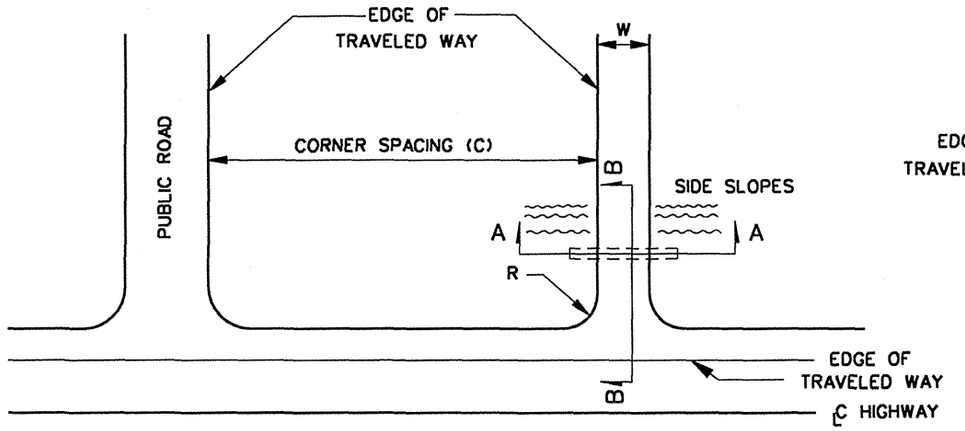
3. Widths. The following apply:

   a. These widths apply to the total, all-weather surface width available.

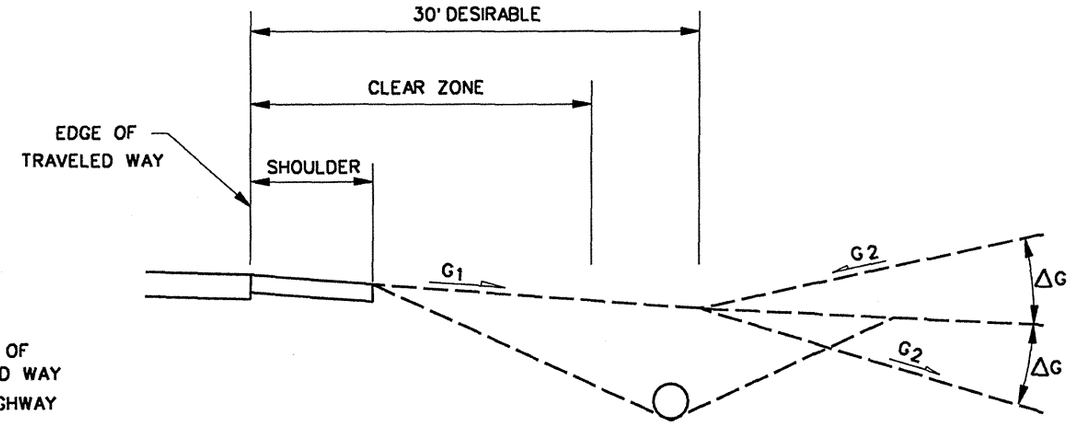
   b. These widths are sufficient for one-way operation. For two-way operation, the minimum width for multi-unit residential driveways should be 30 ft. For commercial/industrial driveways with two-way operation, widths should be provided based on the likelihood that vehicles will be entering and exiting from the driveway simultaneously.

4. Driveway Side Slopes. Section 9-1.0 discusses clear zones for various highway conditions. Note that clear zones are determined based on the direction of travel which is exposed to the drainage pipe end section. This could mean, for example, that the leading end of the pipe may be within the clear zone, and the trailing end may be outside of the clear zone.

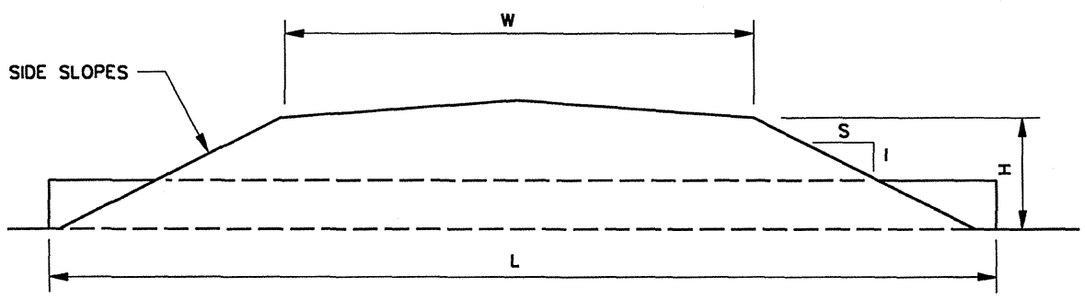
5. Sight Distance. Because of the typical low volumes at driveways, it is not warranted to explore extraordinary measures to improve sight distance at driveways. However, the designer should check for sight obstructions in the vicinity of the driveway entrance (e.g., large trees, hedgerows) which may cause a problem. To perform the check, it is reasonable to assume an eye location of approximately 10 ft from the edge of traveled way.



PLAN VIEW



SECTION B-B
(FOR CUT AND FILL SECTION)



SECTION A-A

Example 6-9.1

Given: W = 16 ft
H = 8 ft
S = 6

Problem: Determine length of pipe.

Solution: L = 16′ + 2 (6)(8′) = 112′

TYPICAL RURAL DRIVEWAYS
Figure 6-9A

6-10.0 INTERCHANGES

6-10.01 Warrants

6-10.01.01 Guidelines

Although an interchange is a high-level compromise for intersection problems, its high cost and environmental impact require that interchanges only be used after careful consideration of its costs and benefits. Because of the great variance in specific site conditions, MDOT has not adopted specific interchange warrants. When determining the need for an interchange or grade separation, the following guidelines should be considered:

1. Design Designation. Once it has been decided to provide a fully access-controlled facility, each intersecting highway must be terminated, rerouted, provided a grade separation or provided an interchange. The importance of the continuity of the crossing road and the feasibility of an alternative route will determine the need for a grade separation or interchange. An interchange should be provided on the basis of the anticipated demand for access to the minor road.

   On facilities with partial control of access, intersections with public roads will be accommodated by an interchange or with an at-grade intersection; grade separations alone are not normally provided. Typically, an interchange will be selected for the higher-volume intersecting roads. Therefore, on a facility with partial control of access, the decision to provide an interchange will be, in general, based on the criteria in the following comments.

2. Functional Classification. Interchanges will be provided at all freeway-to-freeway crossings. On fully access-controlled facilities, interchanges should be provided with all major highways, unless this is determined inappropriate for other reasons. Interchanges to other highways may be provided based on a case-by-case evaluation (e.g., costs, traffic volumes, alternative routes).

3. Congestion. An interchange may be considered where the level of service (LOS) at an at-grade intersection is unacceptable, and the intersection cannot be redesigned at-grade to operate at an acceptable LOS. Although a LOS criteria and traffic volumes are the most tangible of any interchange guideline, the Department has not adopted any specific criteria which, when exceeded, would demand an interchange. However, it is still an important factor. For example, the point at which volumes for an at-grade intersection exceed capacity may warrant an interchange. In addition, other factors, such as costs, right-of-way and environmental concerns, need to be considered.

4. Safety. The crash reduction benefits of an interchange should be considered at an existing at-grade intersection which has a high crash rate.

5. Site Topography. At some sites the topography may be more adaptable to an interchange than an at-grade intersection.

6. Road-User Benefits. Interchanges may significantly reduce the travel time when compared to at-grade intersections but may increase the travel distances. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then an interchange may be warranted.

7. Interchange Spacing. When interchanges are spaced farther apart, freeway operations are improved. Spacing of urban interchanges between interchange crossroads should not be less than 1 mile. Typically, this allows adequate distance for an entering driver to adjust to the freeway environment, to allow for proper weaving maneuvers between entrance and exit ramps, and to allow adequate advance and exit signing. In rural areas, interchanges should not be spaced less than 3 miles apart on the Interstate system or 2 miles on other facilities.

6-10.01.02 New/Revised Access Points on the Interstate System

Each entrance and exit point on the mainline of an Interstate facility, including "locked gate" access (e.g., a utility opening), is defined as an access point (e.g., diamond interchanges have four access points). A revised access is considered to be a change in the interchange configuration even though the number of access points may not change (e.g., replacing a loop with a semi-directional ramp).

The Department's goal is to maintain the highest level of service, safety and mobility on its Interstate system. Among other design features, this is accomplished by controlling access onto the system. In general, new access points on existing fully access-controlled facilities are discouraged. All proposed new or revised access points on the Interstate system will require formal approval from FHWA. See the *Federal Register*, Vol. 55, No. 204, Monday, 10-22-90 and *Federal Register*, Vol. 63, No. 28, Wednesday, 2-11-98. These proposals must fully address the following factors:

1. Traffic Volumes. The proposal must demonstrate that existing interchanges and/or local roads and streets within the corridor cannot satisfactorily accommodate, nor can the existing network be feasibly improved to accommodate, the expected design-year traffic volumes.

2. Alternatives. The proposal must demonstrate that all reasonable alternatives for design options, locations and transportation system management type improvements (e.g., ramp metering, mass transit, HOV facilities) have been evaluated, provided for and/or provision made for future incorporation.

3. Impacts. The proposed new access point should not have a significant adverse impact on the safety and operation of the Interstate facility based on an analysis of current and future traffic (e.g., 20 years in the future). The operational analysis for existing conditions should include:

    a. an analysis of Interstate sections to, and including, at least the first adjacent existing or proposed interchange on either side; and

    b. an analysis of crossroads and other roads/streets to ensure their ability to collect and distribute traffic to and from the proposed interchange.

4. Connections. The proposed new interchange will only be connected to a public road, and it will provide for all traffic movements. Less than "full interchanges" for special purpose access for transit vehicles, for HOV entrances or to park-and-ride lots may be considered on a case-by-case basis.

5. Land Use. The proposal must address the consistency of the interchange with local and regional development plans and transportation system improvements. For possible multiple interchange additions, the proposal must be supported by a comprehensive Interstate network study which should address all proposed and desired access within the context of a long-term plan.

6-10.02 Interchange Types

All interchanges should provide for all movements, even when the anticipated turning volume is low. An omitted maneuver may be a point of confusion to those drivers searching for the exit or entrance. In addition, unanticipated future developments may increase the demand for that maneuver.

Each interchange must be custom-designed to fit the individual site considerations. The final design may be a modification of one of the basic types or may be a combination of two or more basic types. The following are the basic types of interchanges used in Mississippi:

1. Diamond. The diamond is the simplest and most common type of interchange. One-way diagonal ramps are provided in each quadrant with two at-grade intersections provided at the minor road. If these two intersections can be properly designed, the diamond is usually the best choice of interchange where the intersecting road is not access controlled.

2. Full Cloverleafs. Cloverleaf interchanges are used at 4-leg intersections and employ loop ramps to accommodate left-turn movements. Loops may be provided in any number of quadrants. Full cloverleaf interchanges are those with loops in all four quadrants; all others are partial cloverleafs. Collector-distributor (C-D) roads should be considered with cloverleaf designs.

3. Partial Cloverleafs. Partial cloverleaf interchanges are those with loops in one, two or three quadrants. They are appropriate where right-of-way restrictions preclude ramps in one or more quadrants. They are also advantageous where a left-turn movement can be provided onto the major road by a loop without the immediate presence of an entrance loop from the minor road. Collector-distributor (C-D) roads should be considered with partial cloverleaf designs.

4. Directional and Semi-Directional. The following definitions apply to directional and semi-directional interchanges:

   a. Directional Ramp — A ramp that does not deviate greatly from the intended direction of travel.

   b. Semi-Directional Ramp — A ramp that is indirect in alignment, yet more direct than loops.

   c. Fully Directional Interchange — An interchange where all left-turn movements are accommodated by directional ramps.

   d. Semi-Directional Interchange — An interchange where one or more left-turn movements are accommodated by semi-directional ramps, even if the minor left-turn movements are accommodated by loops.

   Directional or semi-directional ramps are used for heavy left-turn movements to reduce travel distance, to increase speed and capacity and to eliminate weaving. These types of connections allow an interchange to operate at a better level of service than is possible with loop ramps.

5. Single Point Urban Interchange (SPUI). The single point urban interchange is a special type of diamond interchange. With this interchange, all legs of the interchange meet at a single point. It can significantly increase the interchange capacity, alleviate the operational problems of having two closely spaced at-grade intersections on the minor road, and overcome the left-turn lane storage problem for drivers wishing to enter the freeway. The SPUI may be considered in highly developed urban areas where right-of-way is restricted. Figure 6-10A presents a schematic of the SPUI.

6-10.03 Freeway/Ramp Junctions

6-10.03.01 Exit Ramps

Type

The Department uses two types of exit ramps — the taper type and the parallel-lane type. The following discusses Department policy on their usage:

1. Taper Type. This is the preferred exit ramp design. It should be used at all exit locations except the following:

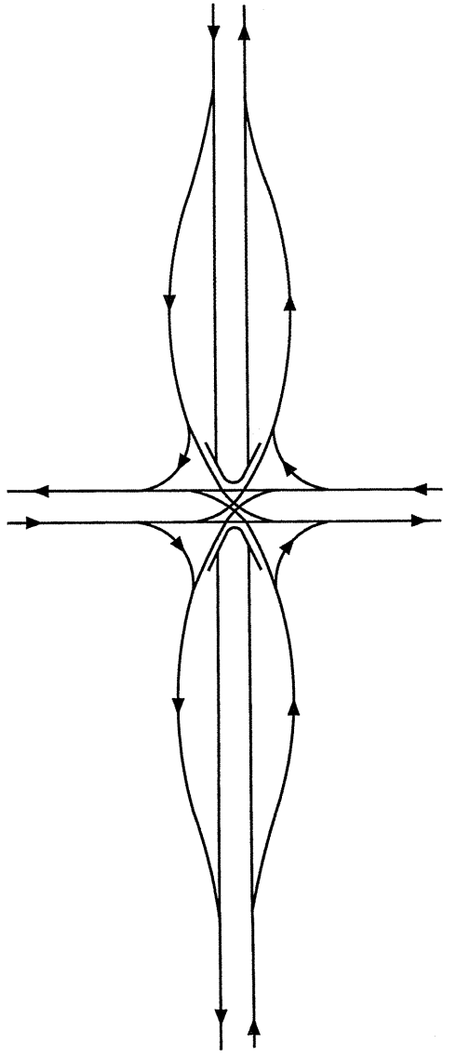
   a. where the available deceleration distance is inadequate (see below),

   b. where all or part of the taper will cross a bridge, or

   c. at any other location where the parallel-lane type is deemed advantageous.

2. Parallel-Lane Type. This type of exit ramp should only be used where judged superior to the taper type as discussed in Comment #1.

The *Mississippi Roadway Design Standard Drawings* illustrates the design details for both the taper and parallel-lane exit ramps.



SCHEMATIC OF SINGLE POINT URBAN INTERCHANGE
Figure 6-10A

Deceleration Distance

Sufficient deceleration distance is needed to safely and comfortably allow an exiting vehicle to leave the freeway mainline. All deceleration should occur within the full width of the exit ramp. The necessary length for deceleration will depend upon the design speed of the mainline and the design speed of the first governing geometric control on the exit ramp. This will usually be a horizontal curve but could be, for example, stopping sight distance on a crest vertical curve. Table 6-10A provides the minimum deceleration distances for various combinations of highway design speeds and ramp design speeds. If the deceleration area is on a grade of 3% or more, the deceleration distance should be adjusted according to the criteria in Table 6-10B.

6-10.03.02 Entrance Ramps

Type

For new and reconstructed interchanges, Department policy is to only use the parallel-lane entrance ramp. The *Mississippi Roadway Design Standard Drawings* illustrates the design details for the Department's entrance ramp.

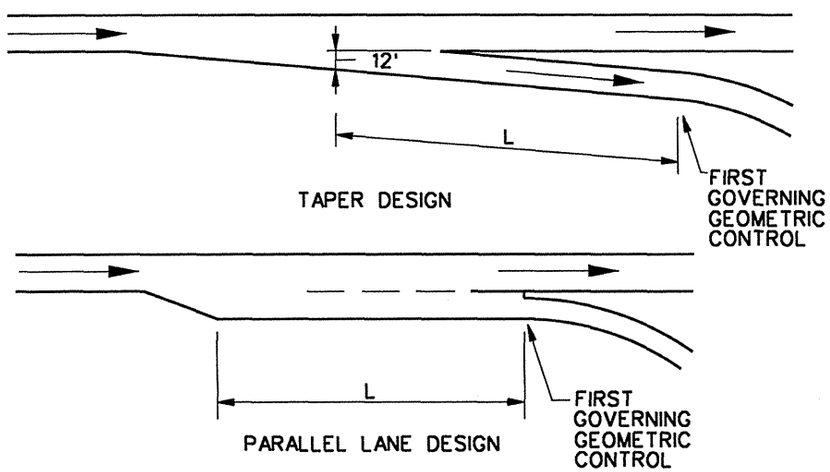
Acceleration Distance

Sufficient acceleration distance will facilitate driver comfort, traffic operations and safety. The necessary length of acceleration will primarily depend upon the design speed of the last (or controlling) horizontal curve on the entrance ramp and the design speed of the mainline. The following will apply:

1. Passenger Cars. Table 6-10C provides the criteria for the minimum length of acceleration. These lengths apply to the full width of the parallel lane; taper lengths are in addition to the table lengths. Where grades of 3 percent or more occur in the acceleration area, adjustments should be made in its length according to Table 6-10D.

Table 6-10A
MINIMUM LENGTH OF DECELERATION

Highway Design Speed, V (mph)	L = Deceleration Length (ft)								
	For Design Speed of Exit Curve, V′ (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	235	185	160	140	-	-	-	-	-
40	315	295	265	235	185	155	-	-	-
50	435	405	385	355	315	285	225	175	-
60	530	500	490	460	430	410	340	300	240
65	570	540	530	490	480	430	380	330	280
70	615	590	570	550	510	490	430	390	340



Notes:

1. *The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the average running speed of the highway mainline to the average running speed of the first governing geometric control.*

2. *These values are for grades less than 3%. See Table 6-10B for steeper upgrades or downgrades.*

Table 6-10B
GRADE ADJUSTMENTS FOR DECELERATION

Direction of Grade	Ratio of Deceleration Length on Grade to Length on Level			
	< 3%	3% ≤ G < 5%	5% ≤ G < 7%	G ≥ 7%
Upgrade	1.0	0.9	0.8	0.7
Downgrade	1.0	1.2	1.35	1.5

Notes:

1. *Table applies to all mainline design speeds.*

2. *The "grade" in the table is the <u>average</u> grade over the distance used for measuring the length of deceleration.*

<u>Example</u>

Given: Mainline Design Speed = 70 mph
First Exit Curve Design Speed = 50 mph
Average Grade = 5% downgrade

Problem: Determine necessary length of deceleration.

Solution: Table 6-10A yields a minimum deceleration distance of 340 ft on the level. According to Table 6-10B, this should be increased by 1.35.

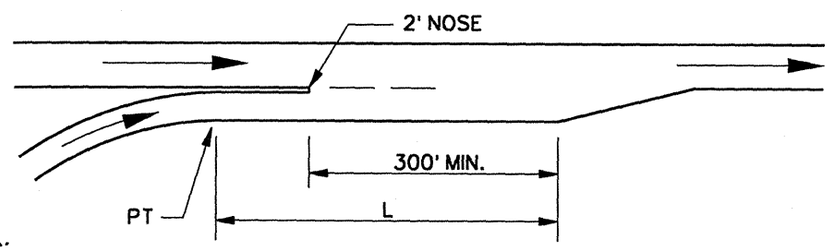
Therefore: L = 340 x 1.35
L = 459 ft

A 460-ft deceleration length should be provided from the full width of the exit ramp to the PC of the first exit curve.

Table 6-10C

MINIMUM ACCELERATION LENGTH (Passenger Cars)

Highway Design Speed (mph)	L = Acceleration Length (ft)								
	For Entrance Curve Design Speed (mph)								
	Stop Condition	15	20	25	30	35	40	45	50
30	190	-	-	-	-	-	-	-	-
40	380	320	250	220	140	-	-	-	-
50	760	700	630	580	500	380	160	-	-
60	1170	1120	1070	1000	910	800	590	400	170
70	1590	1540	1500	1410	1330	1230	1010	830	580



Notes:

1. *The acceleration lengths are calculated from the distance needed for a passenger car to accelerate from the average running speed of the entrance curve to a speed of 10 mph below the average running speed on the mainline. Where V for the mainline ≥ 50 mph, L_{min} = 600 ft.*

2. *These values are for grades less than 3%. See Table 6-10D for steeper upgrades or downgrades.*

3. *Use the value of L or 300 ft beyond the 2-ft nose, whichever is greater. See figure.*

Table 6-10D

GRADE ADJUSTMENTS ON ACCELERATION LANES

Design Speed of Highway (mph)	Ratio of Length on Grade to Length for Design Speed of Entrance Ramp Curve (mph)				
	20	30	40	50	All Speeds
	3% to 4% upgrade				3% to 4% downgrade
40	1.3	1.3	-	-	0.7
50	1.3	1.4	1.4	-	0.65
60	1.4	1.5	1.5	1.6	0.6
70	1.5	1.6	1.7	1.8	0.6
	5% to 6% upgrade				5% to 6% downgrade
40	1.5	1.5	-	-	0.6
50	1.5	1.7	1.9	-	0.55
60	1.7	1.9	2.2	2.5	0.5
70	2.0	2.2	2.6	3.0	0.5

Notes: 1. *No adjustment is needed on grades less than 3%.*

2. *The "grade" in the table is the average grade measured over the distance for which the acceleration length applies.*

Example

Given: Mainline Design Speed = 70 mph
Entrance Ramp Curve Design Speed = 40 mph
Average Grade = 5% upgrade

Problem: Determine necessary length of acceleration.

Solution: Table 6-10C yields an acceleration length of 1010 ft on the level. According to Table 6-10D, this should be increased by a factor of 2.6.

Therefore: L = 1010 x 2.6
L = 2626 ft

A 2625-ft acceleration length should be provided from the PT of the entrance ramp curve to the beginning of the taper.

The values in Table 6-10C provide sufficient distance for vehicular acceleration; they may not safely allow a vehicle to merge into the mainline if traffic volumes are high. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merging area, the acceleration length should be at least 1200 ft in addition to the distances in Table 6-10C.

2. Trucks. Where there are a significant number of trucks to govern the design of the entrance ramp, the truck acceleration distances provided in Table 6-10E should be considered. Typical areas where trucks might govern the ramp design will include weigh stations, truck stops and transport staging terminals. At other freeway/ramp entrances, the truck acceleration distances should be considered where there is substantial entering truck traffic and where:

   a. there is LOS D or worse at the freeway/ramp junction,

   b. there is a significant accident history involving trucks which is attributable to an inadequate acceleration length, and/or

   c. there is undesirable vehicular delay at the junction attributable to an inadequate acceleration length.

   Where upgrades exceed 3%, the truck acceleration distance may also be adjusted for grades (see Table 6-10D). Before providing acceleration lengths for trucks, the designer must consider the impacts of the added length (e.g., additional construction costs, wider structures, right-of-way impacts).

6-10.04 Access Control

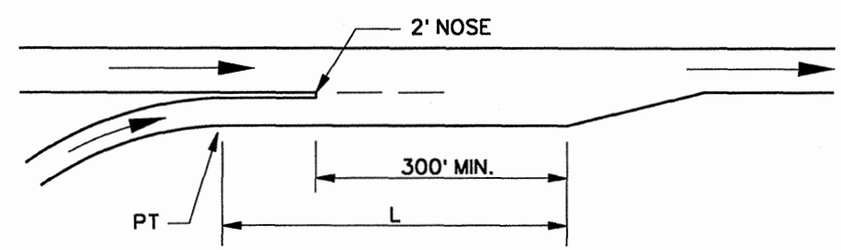
Proper access control must be provided along the crossing road in the vicinity of the ramp/crossing road intersection. This will ensure that the intersection has approximately the same degree of freedom and absence of conflict as the freeway itself. Figure 6-10B illustrates the Department's policy for the location of the no-access line at ramp/crossing road intersections.

Table 6-10E

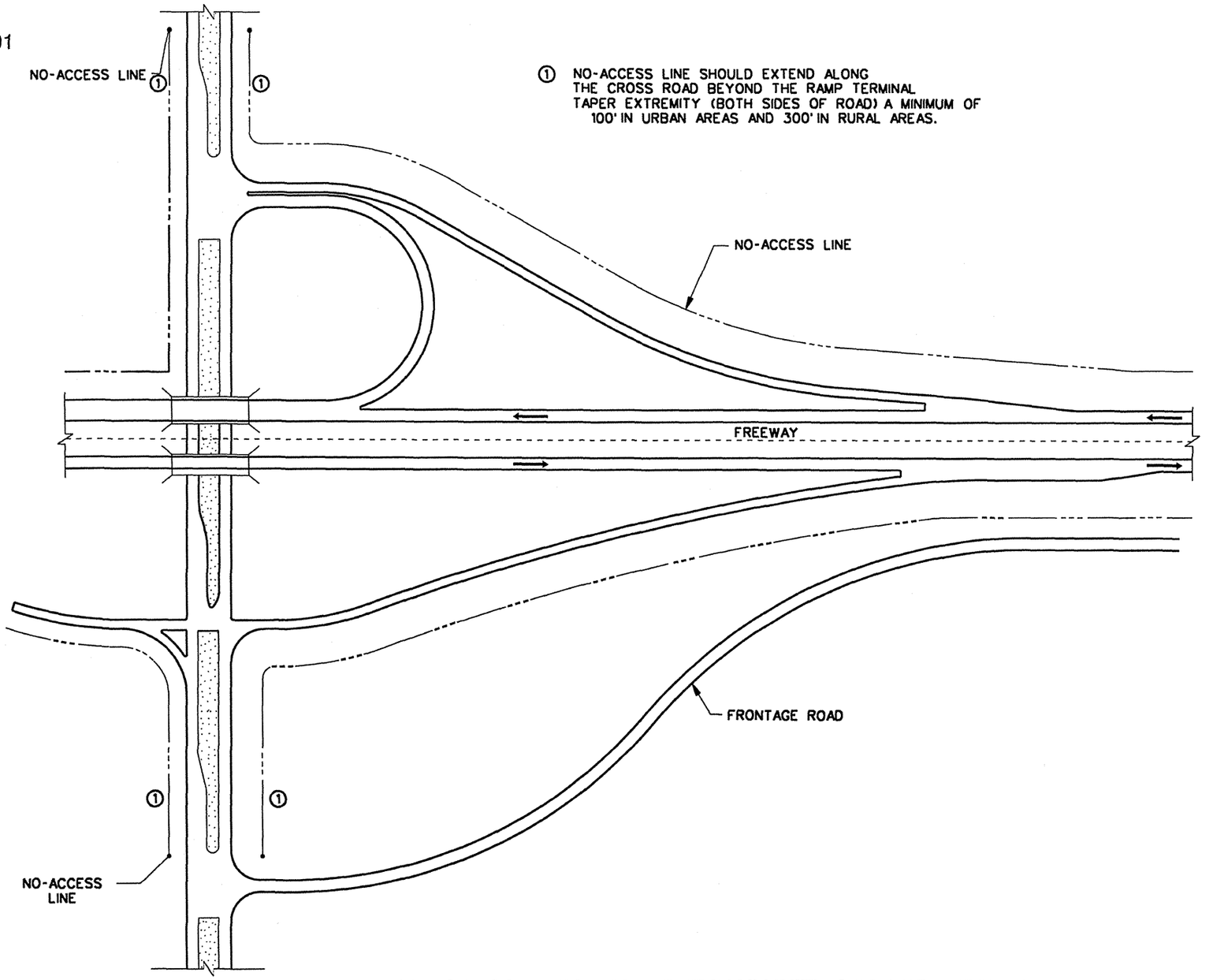
LENGTHS FOR ACCELERATION (300 lb/hp Truck)

Highway Design Speed (mph) (V)	L = Acceleration Length (ft) For Entrance Curve Design Speed (mph) (V')								
	Stop	15	20	25	30	35	40	45	50
30	150*	-	-	-	-	-	-	-	-
40	350*	300*	250*	150*	-*	-	-	-	-
50	900	850	800	700	600	450	-	-	-
60	2200	2150	2100	2000	1900	1800	1000	900	-
70	3700	3700	3700	3600	3400	3300	2700	1950	1100

** Use L from Table 6-10C.*



Note: The acceleration lengths are calculated from the distance needed for a 300 lb/hp truck to accelerate from the average running speed of the entrance curve to reach a speed which is 10 mph below the average running speed on the mainline.



NO-ACCESS LINE TREATMENT IN VICINITY OF RAMP TERMINAL
Figure 6-10B

One situation warrants a special discussion. Many interchanges were initially constructed in Mississippi when the surrounding area was rural in character. Since that time, the area may have become suburban or urban. As indicated in Figure 6-10B, the Department has adopted different criteria for the access control at urban and rural interchanges. However, it is Department policy that the change in area character alone is not a sufficient justification to alter the location of the no-access line.

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CHAPTER 7

DRAINAGE

MISSISSIPPI DESIGN MANUAL

Chapter 7

DRAINAGE

Section		Page
7-1.0	DESIGN RESPONSIBILITIES	7-1
7-1.01	Bridge Design Division	7-2
7-1.02	Roadway Design Division	7-3
7-1.03	State Aid Division	7-3
7-2.0	FLOODPLAINS	7-4
7-2.01	Federal Emergency Management Agency (FEMA)	7-4
7-2.02	Definitions	7-4
7-2.03	Mississippi Regulations	7-5
7-3.0	HYDROLOGY	7-5
7-3.01	Design Storm Frequency	7-6
7-3.02	Culvert Hydraulic Design Data	7-7
7-4.0	DESIGN DISCHARGE	7-7
7-4.01	Rational Method	7-8
7-4.02	Method of USGS 1991 Report "Flood Characteristics of Mississippi Streams" (Rural Discharge)	7-15
7-4.03	Method of USGS 1983 Water-Supply Paper 2207 "Flood Characteristics of Urban Watersheds in the United States"	7-18
7-4.04	Method from USGS 1967 Report on "Urbanization Effects on Floods in Jackson, Mississippi"	7-21
7-4.05	USGS Individual Stream Study Reports	7-23
7-5.0	CULVERT DESIGN PRACTICES	7-23
7-5.01	Minimum Size Culverts	7-23
7-5.02	Culvert Discharge Limits	7-24
7-5.03	Economic Comparisons	7-24
7-5.04	Pipe Culvert Alternatives	7-24
7-5.05	Type and Placement of Pipe	7-25
7-6.0	CULVERT SIZE DETERMINATION	7-25
7-6.01	Inlet and Outlet Control Culvert Flow	7-26
7-6.02	Improved Culvert Entrances	7-26
7-6.03	Size Design for Concrete Pipe Culverts	7-26
7-6.04	Size Design for Box Culverts	7-39
7-7.0	MEDIAN DRAINAGE	7-47
7-7.01	Design Criteria	7-47
7-7.02	Median Drain Inlet Spacing	7-47
7-7.03	Standard Inlets	7-48
7-8.0	STORM SEWER DESIGN	7-49
7-8.01	Determining Runoff Quantities	7-49
7-8.02	Ponding Limits on Roadway	7-50
7-8.03	Inlets	7-51
7-8.04	Inlets on Grade	7-51
7-8.05	Inlets in Sag Vertical Curves or Sump Conditions	7-58
7-8.06	Inlet Spacing	7-63
7-8.07	Storm Sewer Design Policies	7-63
7-8.08	Runoff Discharge Tabulation	7-64
7-8.09	Sizing Storm Sewer Lines	7-65
7-8.10	Subsurface Drainage (Storm Sewer)	7-67
7-8.11	Junction Boxes and Manholes	7-67
7-9.0	SUBSURFACE DRAINAGE	7-68
7-9.01	Application	7-68
7-9.02	Underdrains	7-68
7-10.0	OPEN CHANNELS	7-69
7-10.01	Size and Shape	7-69
7-10.02	Capacity Computations	7-70
7-10.03	Channel Alignment and Gradient	7-71
7-10.04	Channel Lining	7-71
7-10.05	Energy Dissipators	7-72
7-11.0	PHYSICAL STANDARDS	7-74
7-11.01	Alternative Types of Pipe Culverts	7-74
7-11.02	Types of Pipe Culverts (State Aid Division)	7-75
7-11.03	Strength Identification	7-75
7-11.04	Pipe Culvert Strength Requirements	7-75
7-11.05	Special Provision Against Deterioration	7-76
7-11.06	Bedding and Backfill	7-76
7-11.07	Pipe Culvert End Sections	7-76
7-11.08	Multiple Pipes	7-77
7-11.09	Skewed Installations	7-77
7-11.10	Box Culverts	7-78
7-11.11	Culvert Length	7-78
7-11.12	Structure Excavation	7-78
7-12.0	REFERENCES	7-79
	PIPE CULVERT DESIGN FORM	7-80

Chapter 7

DRAINAGE

This chapter is intended only as a generalized and cursory guide of the criteria and procedures for use in highway related drainage design. All drainage design for the Department shall be in strict accordance with AASHTO procedures and specifications. For more extensive and comprehensive information on hydrology/hydraulics/drainage design, there are numerous FHWA, AASHTO, and other approved publications and software available and recommended (see Section 7-12.0).

Drainage design includes both the hydraulic design of highway culverts and the determination of the physical standards for culverts. Designers will determine the runoff discharge, select the size and type of drainage structure, and satisfy the culvert strength requirements for the given location.

Drainage design should achieve the most effective and economical methods by which runoff waters can be passed through and removed from the roadway. Drainage installations should not create hazardous conditions for traffic operations nor should they adversely affect conditions of adjoining properties. The primary objectives of good drainage design are to:

1. provide culvert openings for natural drainage channels;

2. prevent undue accumulation and retention of water upon and adjacent to the roadway; and

3. protect the roadway against storm and subsurface water damage.

For the purpose of converting English units-of-measure to the metric system-of-units, the conversion factors presented in Figure 7-1A should be applied.

Quantity	From U.S. Customary	To Metric (SI)	Multiply By
Length	mile	km	1.609 344
	foot	m	0.304 800
	inch	mm	25.400 000
	yard	m	0.914 400
Area	square yard	m^2	0.836 127
	square foot	m^2	0.092 903
	acre	ha (10 000 m^2)	0.404 687
	square mile	km^2	2.589 998
Volume	cubic yard	m^3	0.764 555
	fluid ounce	mL	29.573 530
	gallon	L	3.785 412
	cubic foot	m^3	0.028 317
Flow Rate	cubic feet/second	m^3/s	0.028 317
Velocity	miles/hour	km/h	1.609 344

CONVERSION FACTORS
Figure 7-1A

7-1.0 DESIGN RESPONSIBILITIES

Responsibilities for drainage design are divided between the Bridge Design Division and the Roadway Design Division. Each Division's responsibility is based principally on the size of the drainage area involved. Design of the channel and erosion protection will be performed by the Division responsible for design of the drainage

structure. The Bridge Design Division is responsible for the structural adequacy of all box and bridge structures. The districts will provide all survey data and field information necessary for the drainage design.

7-1.01 Bridge Design Division

Most major structures (bridges and box bridges) are designed by the Bridge Design Division. Responsibilities include:

1. all locations requiring hydraulic bridges;

2. all locations requiring a "box bridge," classified as exceeding 20 ft in width measured along the roadway (Note: An exception applies when the drainage area < 1000 acres. In this case, the Roadway Design Division may be responsible for design.);

3. all locations with drainage areas in excess of 1000 acres and any subordinate culverts within the same floodplain; and

4. all locations requiring special design box culverts such as non-standard dimensions and skew angles.

Survey and field information required by the Bridge Design Division for hydraulic design includes the following:

1. A typical floodplain profile and the location thereof with reference to the centerline survey. The stream channel should be included on the profile.

2. A profile along the present highway presenting the finished grade.

3. A ground-line profile along the proposed detour.

4. A stream-traverse 500 ft (150 m) upstream and 500 ft (150 m) downstream for typical crossings.

5. Typical channel sections normal to the channel (at least one upstream and one downstream). Elevations and corresponding horizontal dimensions from the traverse baseline should be provided.

6. A stream profile (taken at the deepest part of the stream) extending 500 ft (150 m) upstream and downstream. The survey notes should reference this to the traverse baseline.

7. The water-surface elevation at both ends of the stream profile.

8. Drainage data as follows:

   a. High-Water: Elevations, approximate dates and sources of information for floods of significance. The location of high-water marks should be clearly designated relative to the centerline of the proposed route. Both upstream and downstream high-water marks are desired.

   b. Drainage area at the site.

   c. Comment on drift and bank stability.

9. Data on existing bridges (Note: This information is needed at other highways, county roads and railroads in the vicinity plus a finished grade centerline profile of the facility.):

   a. Profile beneath the bridge.
   b. Bottom elevation of the low stringers.
   c. Type of bridge and span lengths.

10. Critical upstream control elevations (identity and location) where damages, hazards, etc., may begin occurring.

For the hydraulic design of structures on a relocated line, the ten items listed above will be required except Items #2 and #3. Where the new line is not in the vicinity of the old line, the centerline survey profile will often suffice for Item #1. In addition, where twin bridges are proposed, a ground line profile is needed along the centerline of both lanes.

Blue line work prints provided by the Roadway Design Division for use in hydraulic design include the following:

1. layout sheet,

2. roadway typical sections, and

3. plan-profile sheets and cross sections of the location which include the entire width of the floodplain.

7-1.02 Roadway Design Division

Most routine culvert installations will be designed by the individual roadway designer following the procedures and criteria in this *Manual*. The Hydraulic Section of the Division will provide technical assistance when required and will conduct special investigations of all non-routine, unusual conditions (e.g., bridge replacements with culverts and improved entrance structures).

Drainage design responsibilities of the Roadway Design Division include:

1. design of most culverts accommodating drainage areas up to 1000 acres;
2. design of storm sewer systems; and
3. side ditch and respective channel erosion control treatments.

Design of culverts for drainage areas less than 200 acres will require the following minimum field information:

1. drainage area;

2. high-water mark elevations upstream and downstream of the location;

3. descriptive information on existing culverts serving the same drainage area under nearby highways and railroads;

4. sufficient channel flowline profile to establish the natural channel slope, 200-300 ft upstream and downstream of the crossing;

5. typical channel sections upstream and downstream normal to the channel; and

6. critical upstream control elevations (identity and location) where damages, hazards, etc., may begin occurring.

Culvert design for drainage areas greater than 200 acres will require the same applicable field information necessary for Bridge Design Division hydraulic design (see Section 7-1.01).

The roadway designer should review the submitted drainage data on receipt of a new project and should immediately request from the district any omitted essential information.

7-1.03 State Aid Division

Policy and procedure require that the County Engineer will have complete design responsibility for all drainage structures and will obtain the same field data required by the Department as outlined in Section 7-1.0. On complex drainage areas, the County Engineer may request assistance from the U.S. Geological Survey and the State Aid Division. In all cases, review of the County Engineer's design is made by the State Aid Division.

7-2.0 FLOODPLAINS

7-2.01 Federal Emergency Management Agency (FEMA)

The Federal Emergency Management Agency (FEMA) administers the National Flood Insurance Program (NFIP). FEMA has established regulations for modifications to floodways and floodplains. Department coordination with FEMA is required under the following circumstances:

1. a proposed crossing encroaches on a regulatory floodway and would require an amendment to the floodway map;

2. a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated;

3. a local community is expected to enter into the regular program within a reasonable period, and detailed floodplain studies are under way; and

4. a local community is participating in the emergency program, and the base flood elevation in the vicinity of insurable buildings is increased. (Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to base flood elevations as a result of highway construction.)

Details of requirements for meeting the conditions of FEMA and NFIP are contained in a FHWA paper, "Procedures for Coordinating Highway Encroachments on Floodplains with FEMA." Other documents which provide additional guidance on floodplains include Federal Floodplain Executive Order No. 11988 and the Federal-Aid Policy Guide, Title 23-Code of Federal Regulations 650A, "Location and Hydraulic Design of Encroachment on Floodplains."

7-2.02 Definitions

Figure 7-2A provides a schematic for illustrating the FEMA requirements. The following definitions apply to the figure:

1. Regulatory Floodway. The channel of a river or other watercourse and the adjacent land areas which are regulated by Federal, State or local governments. The floodway must be reserved in an open manner (i.e., unconfined or unobstructed either horizontally or vertically) to provide for the discharge of the base flood so the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 ft as established by the FEMA for administering the National Flood Insurance Program).

2. Floodway Fringe. The portion of the base floodplain outside of the regulatory floodway.

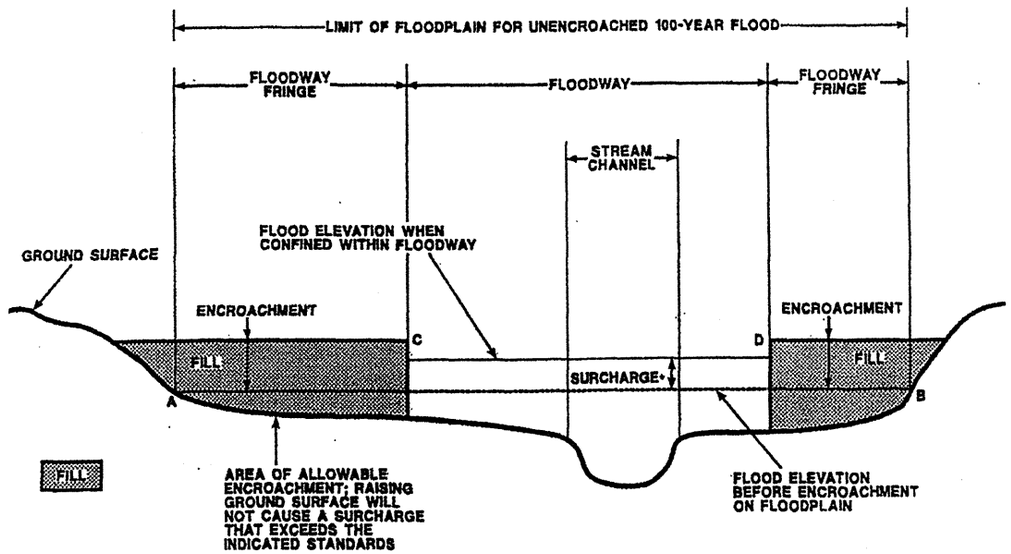
3. 100-year Floodplain. The floodplain limits which have a return frequency of once in 100 years (1% chance floodplain). A floodplain is the low land and relatively flat area adjoining inland and coastal waters. The 100-year floodplain is also the base floodplain.

4. Encroachments. An action within the limits of the base floodplain. Significant encroachment would include a highway or any direct support development that would involve one or more of the following flood-related impacts:

    a. a significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route;

    b. a significant risk; or

    c. a significant adverse impact on natural and beneficial floodplain values.



FLOODWAY SCHEMATIC
Figure 7-2A

5. Surcharge. The cumulative increase in water surface elevation after encroachment on a floodplain.

7-2.03 Mississippi Regulations

The State of Mississippi has established its requirements for floodplains. These are documented in House Bill #8 "Floodplain Management Regulations for the State of Mississippi," adopted August 1, 1979, and amended June 10, 1982. The following summarizes the impacts of the law:

1. Application. The regulations apply to any proposed road or bridge project within a floodplain on a highway under the jurisdiction of the Mississippi Department of Transportation.

2. Authority. The Mississippi Commission of Budget and Accounting has been vested with the overall authority to administer and enforce the regulations. For roads and bridges under the jurisdiction of the Department, this authority has been delegated to the Department.

3. Objective. The basic objective of the regulations is to protect public safety and property by placing restrictions and controls on any development within a floodplain.

4. Standards/Procedures. Articles 4 and 5 of the law delineate specific standards and procedures which must be met before any proposed project within a floodplain can be approved. The designer should refer to the law for these standards and procedures.

7-3.0 HYDROLOGY

Hydrology is the science pertaining to the properties, distribution and behavior of water in nature. Hydrologic analysis is a most important step in the hydraulic design

of a drainage structure where the determination is made of the design discharge that must be accommodated. The design discharge depends largely on topographic and climatic factors, such as the size and characteristics of the drainage basin area. The analysis process considers the following:

1. selecting the design storm frequency,
2. selecting the appropriate method for estimating discharges, and
3. determining the design discharges.

Detailed discussions of the hydrologic analysis process are provided in the AASHTO *Highway Drainage Guidelines* and *Model Drainage Manual.*

7-3.01 Design Storm Frequency

Storm frequency (recurrence interval or return period) is the average interval of time within which a given storm event will be equaled or exceeded once. Storm frequency and flood frequency are used interchangeably in this *Manual.* (The term "storm" has been preferred to "flood" in certain litigation references.)

Economic feasibility dictates establishment of acceptable design storm frequencies rather than structure design for maximum potential runoff. The more common storm frequencies considered in highway design are the 2-, 5-, 10-, 25-, 50- and 100-year recurrence intervals. Deriving the inverse of these respective frequencies indicates the percentage of probability that the storm will occur or be exceeded in a given year — i.e., 1/(2-year) or 0.50 = 50%; 1/(5-year) or 0.20 = 20%; 1/(10-year) or 0.10 = 10%; 1/(25-year) or 0.04 = 4%; 1/(50-year) or 0.02 = 2%; and 1/(100-year) or 0.01 = 1% storm. Based on the highway functional classification and the type of drainage installations, the recommended design storm frequency criteria are provided in Table 7-3A.

The 2.33-year storm is the literal Mean Annual Storm; however, in most hydrological presentations, equations for the 2-year storm have been developed and are

Table 7-3A
RECOMMENDED STORM DESIGN FREQUENCY CRITERIA

Type of Installation	Functional Class		
	Freeway	Arterial	Collector/ Local (1)
Cross Drains	50-year (2)	Up to 50-year (3)	25-year (3) (4)
Median Drains	50-year	Up to 50-year (3)	25-year (3)
Side Drains	50-year (5)	25-year (5)	25-year
Storm Sewers	10-year (5)	10-year (5)	10-year (5)

Notes:

1. *This column will be used for State Aid projects.*
2. *Use at least a 50-year design storm through drainage structures; 100-year storm conveyance either through structures or both through structures and over the highway with appropriate flood risk assessment of hazard and/or nuisance (can require greater storm conveyance through structures).*
3. *Design storm frequency provided is commensurate with flood risk assessment (can be greater storm).*
4. *Check function of structures on a 50-year design storm; use 50-year storm when warranted (e.g., restrictive sags, principal service arteries).*
5. *Side drains unusual on freeways; use a 50-year storm for cross drain channels, and depressed area inlets and drains (can be greater storm for underpasses).*
6. *Spacing of curb and gutter inlets on full-shoulder width highways should contain the 10-year storm runoff (50-year in depressed areas) on the shoulder; one-half of adjacent lane on other controlled-access highways; the width of adjacent lane on other multilane highways; one-third of adjacent lane on 2-lane, 2-way highways (see Section 7-8.0). Unique conditions may warrant conservative deviations from these runoff containment limits.*
7. *Median drain spacing should be designed so that water will not rise above the level of the subgrade for the 10-year storm (see Section 7-7.0).*
8. *Use a 5-minute minimum duration time for median and storm sewer design. Adjust duration time appropriately as design of system progresses.*

considered equivalent to the Mean Annual Storm. Therefore, the 2-year (50%) storm has a 50% probability of occurring in any one-year interval. Often, references are made to the 2% and 1% storms (a 2% and 1% annual probability), respectively, the 50-year and 100-year storms.

Storm design for freeway and arterial projects should be the passage of the design storm (generally the 50-year storm) through the culverts, and conveyance of the base storm (the 100-year storm) either through the culverts or both through the culverts and over the highway without undue traffic interruptions, property damage and possible loss of life. The design storm for collector and local projects should be the 25-year storm unless the culvert is in a restrictive sag vertical curve or unless the roadway is a principal service artery. In these cases, the 50-year storm should be considered. Development within a floodplain may necessitate deviations from the foregoing criteria based on flood hazard assessments.

7-3.02 Culvert Hydraulic Design Data

For each culvert equivalent to a 24-in pipe and larger, the project plans should include:

1. estimated discharge and frequency and estimated headwater elevation for the design storm and the base storm (100-year) if different from the design storm, and the recurrence interval and discharge of the storm that will first overtop the roadway if greater than the base storm (Note: When overtopping storm exceeds the 500-year storm, a statement to this effect with the 500-year discharge and headwater elevation is sufficient.); and

2. when available, the authentic discharge, high-water elevation and date of occurrence of the record storm if greater than the base storm.

Each design discharge should be computed on the basis of land use appropriate for the given location, although no less than the probable 20-year projected land use should be considered for design.

The following list summarizes the design data for each culvert typically included in the project plans.

Q_{Design}	Q_{100}	$Q_{Overtop \leq 500\text{-year}}$	Q_{Record}
HW_{Design}	HW_{100}	$HW_{Overtop}$	HW_{Record}*

* *Date of occurrence authentically gaged.*

Design data should be recorded on the Culvert Hydraulic Design Summary Table illustrated in Figure 7-3A. This table provides a uniform summary of design data and a reference and review source for culvert design evaluation. The table should be included in the Summary of Quantities (Roadway) sheets in the contract plans.

In addition, the designer should record on the plans the design water surface elevations of streams with a skewed approach which controls the roadway grade.

7-4.0 DESIGN DISCHARGE

Design discharge is the estimated rate of flow, usually in cubic feet per second, expected at a culvert location from a particular drainage area as a result of a specific storm frequency event. Several methods are used for determining design discharge. The five methods most often used in Mississippi are listed below including criteria for their use.

1. Rational Method. Used for drainage areas less than the minimums in the 1991 USGS Report as presented in Section 7-4.02 (i.e., East and Coastal < 64 acres; West < 38 acres; Delta < 70 acres) for roadway slopes and for storm sewer design in both urban and rural areas. The Rational Method should be limited to 160 to 200 acres maximum.

2. Method of USGS 1991 Report on "Flood Characteristics of Mississippi Streams." Used for rural drainage areas of 38 acres up to 800 mi^2.

CULVERT HYDRAULIC DESIGN SUMMARY																		
WK. SH. NO.	STATION	D.A.	CULVERT	UP STREAM FLOW LINE ELEV.	DESIGN STORM (Q 50) (50-YEAR STORM)				BASE STORM (Q 100) (100-YEAR STORM)				ROADWAY OVERTOPPING STORM			STORM OF RECORD			REMARKS
					DISCHARGE C.F.S.	*HEADWATER CONTROL W/OUT HW/D	HW(DEPTH)	HW ELEV.	DISCHARGE C.F.S.	* HEADWATER CONTROL W/OUT HW/D	HW(DEPTH)	HW ELEV.	1 YEAR STORM	DISCHARGE C.F.S.	*HW. ELEV.	DATE OCCURRED	DISCHARGE C.F.S.	HIGH WAT. ELEV.	
4	185+40	397	DBL 6'x8'	257.14	670	1.30	7.80'	264.94	784	1.60	9.60'	266.74	100-YR	872	267.70	4/16/79	796	266.90	
5	216+55	24	48"	258.40	145	1.75	7.00'	257.40	170	INLET	8.90'	259.30	200-YR	190	259.50	–	–	–	
6	247+20	225	10'x8'	265.10	674	1.38	8.28'	273.38	672	1.62	9.72'	274.82	100-YR	720	275.80	–	–	–	
7	278+95	59	60"	257.50	193	1.32	6.60'	264.10	226	1.54	7.70'	265.20	>500-YR	>290	267.75	SLAB CONTROL ELEV		266.30	

* HEADWATER ELEVATION VALUES SHOWN ON THESE PLANS ARE THEORETICAL AND MAY VARY FROM ACTUAL CONDITIONS.

Notes:

1. *When in inlet control, provide the numerical value in the HW/D column; when in outlet control, provide a note indicating outlet control in the column.*

2. *When there is no "storm of record," use the column for additional remarks.*

3. *Indicated disclaimer (*) must be provided with the table.*

CULVERT HYDRAULIC DESIGN SUMMARY TABLE
Figure 7-3A

3. Method of USGS 1983 Water Supply Paper 2207 on "Flood Characteristics of Urban Watersheds in the United States." Used in conjunction with the USGS 1991 Report (Method #2).

4. Method shown in USGS 1967 Report on "Urbanized Effects on Floods in Jackson, Mississippi." Used for urban drainage areas greater than 25 acres. This method is applicable to most of the State with adjustments (also see Method #3).

5. USGS Individual Stream Study Reports. Generally used for areas larger than 1000 acres.

Once the design discharge has been determined and culvert design practices considered, the designer may proceed to Section 7-6.0 "Culvert Size Determination."

7-4.01 Rational Method

7-4.01.01 Definition

The formula for determining design discharge by the Rational Method is:

$$Q_{Design} = CIA$$

Where:

Q_{Design} = The peak runoff rate or discharge for the desired design frequency (cubic feet per second — ft^3/s).

C = A dimensionless runoff coefficient representing drainage characteristics of the watershed.

I_{Design} = The average rainfall intensity for a duration equal to the time of concentration and for the recurrence interval chosen for design (inches per hour — in/hr).

A = Drainage area (acres).

7-4.01.02 Procedure

1. Refer to Table 7-4A "Runoff Coefficient Values" and select or derive a proportional value of "C" appropriate for the combination of terrain, slope, land use and soil classification.

2. Refer to Figure 7-4A "Rainfall Regions Map" and select the region number appropriate for the project location.

3. Figures 7-4B through F provide the Intensity-Duration-Frequency (I-D-F) curves. Select the I-D-F figure appropriate for the region. Averaged Regions I and II and II and III are included. To derive the design rainfall intensity (I), the duration time or time of concentration (t_c) is determined. Using either Figure 7-4G or the equations for shallow concentrated flow, determine the average velocity of the flow with respect to the slope of the water course and the type of surface. Generally, either the unpaved (grassed) or paved line values represent the more frequent and nominal conditions (also represented by the equations); however, when significant portions of the surface area are identified as having specific other ground cover influencing the average flow velocity, values for that surface type should be used (e.g., Nearly Bare Ground, or Forest with Heavy Ground Litter). The values are representative for the recommended range of the Rational Method (160 to 200 acres maximum).

   The equations for average velocity of flow:

   Unpaved (n = 0.05) $V = 16.1345(S)^{0.5}$
   Paved (n = 0.025) $V = 20.3282(S)^{0.5}$

   Where:

   V = average velocity, ft/s
   S = water course slope, ft/ft

   Determine the maximum length of travel for the runoff through the drainage area. Dividing by this derived average velocity gives the time of travel (t_c minutes). Enter the I-D-F figure at this t_c value. Intersect the desired frequency curve to read the rainfall intensity (I, in/hr).

   When warranted, for travel lengths no greater than 300 feet (about 2.5 acres maximum), the Kinematic Wave Formulation Equation (see Figure 7-4H, can be used to determine the travel time/time of concentration corresponding to the established I-D-F curve, Intensity (I, in/hr) values. Flow is considered to become shallow concentrated flow after the maximum travel of 300 feet and those respective figures and equations are required.

4. The drainage area (A) in acres may be given with the field survey data or computed from aerial photos or USGS quadrangle maps. The product of CIA will give the design discharge (Q_{Design}) in cubic feet per second.

Example 7.4.1 illustrates the determination of design discharge by the Rational Method.

Example 7-4.1

Given: Location: Warren County, eastern

Drainage Area (A) = 10 acres, rural, flat and impervious soil
Design Frequency = 50-year storm

L = 760 ft

S = 0.005 ft/ft
n = 0.05 (fallow — no residue)
C = 0.40

Problem: Determine the design discharge at the given location by the Rational Method.

Solution: The designer should follow the procedure established in Section 7-4.01.02.

1. From Table 7-4A, the runoff coefficient (C) is selected at 0.40 for rural, flat, impervious soil with a high potential of cultivation.

2. From Figure 7-4A, Warren County is in Region II.

3. Enter Figure 7-4G at a WATERCOURSE SLOPE of 0.50%. Project horizontally to the GRASSED WATERWAY curve. Project vertically and read velocity of about 1.1 ft/s or, using the unpaved equation $V = 16.1345(S)^{0.5}$, $V = 16.1345(0.005)^{0.5} = 1.14$ ft/s. For 760 ft length of travel by 1.14 ft/s yields 666.67 seconds. Multiplying by 1/60 yields 11.11 minutes (t_c). Enter the I-D-F figure at $t_c = 11.11$ minutes. Project vertically to the 50-year curve. Then project horizontally and read the Intensity as 8.4 in/hr.

4. $Q_{Design} = CIA = (0.4)(8.4)(10) = 33.6\ ft^3/s$

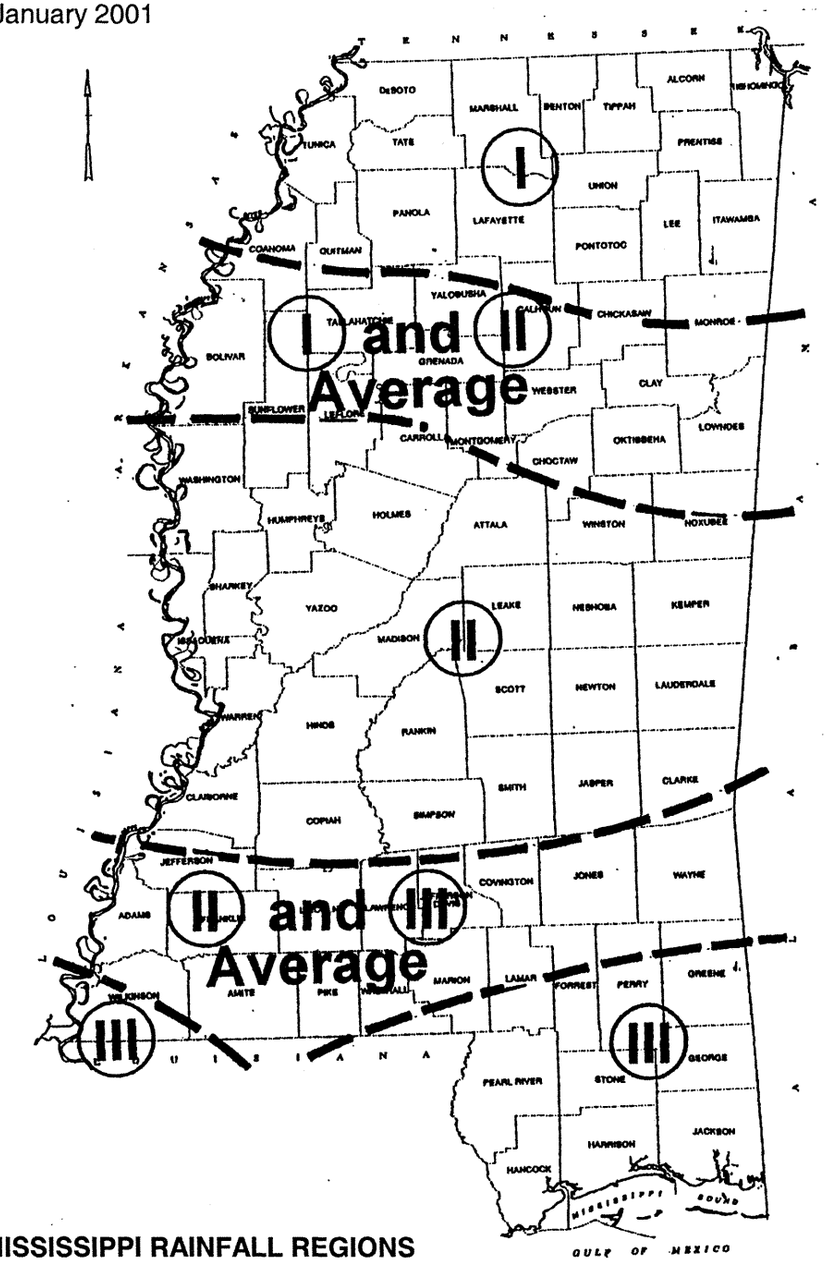
Table 7-4A
RUNOFF COEFFICIENT VALUES (C) FOR THE RATIONAL METHOD
(Drainage Areas — 200 acres maximum, or below minimums in Table 7-4D)*

Slope	Land Use	Soil Classification: Rolling Plains Min.	Rolling Plains Max.	Sand or Sandy Loam Soils (Pervious) Min.	Sand or Sandy Loam Soils (Pervious) Max.	Black or Leossial Soils (Impervious) Min.	Black or Leossial Soils (Impervious) Max.
Flat (0% - 1%)	Cultivated			0.25	0.35	0.30	0.40
	Woodlands			0.15	0.20		
	Pasture			0.20	0.25		
	Paved	0.90		0.90		0.90	
	Residential	0.50	0.60	0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90	0.60	0.90
Rolling (1% - 3.5%)	Cultivated	0.40	0.45	0.45	0.65	0.50	0.70
	Woodlands			0.15	0.20	0.18	0.25
	Pasture	0.25	0.30	0.30	0.40	0.35	0.45
	Paved	0.90		0.90		0.90	
	Residential	0.55	0.60	0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90	0.60	0.90
Hilly (3.5% - 5.5%)	Cultivated			0.60	0.75	0.70	0.85
	Woodlands			0.20	0.25	0.25	0.30
	Pasture			0.35	0.45	0.45	0.55
	Paved	0.90		0.90		0.90	
	Residential	0.60		0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90	0.60	0.90
Mountainous (over 5.5%)	Woodlands					0.70	0.80
	Bare					0.80	0.90
Grassed ROW Slopes		0.70		0.70		0.70	

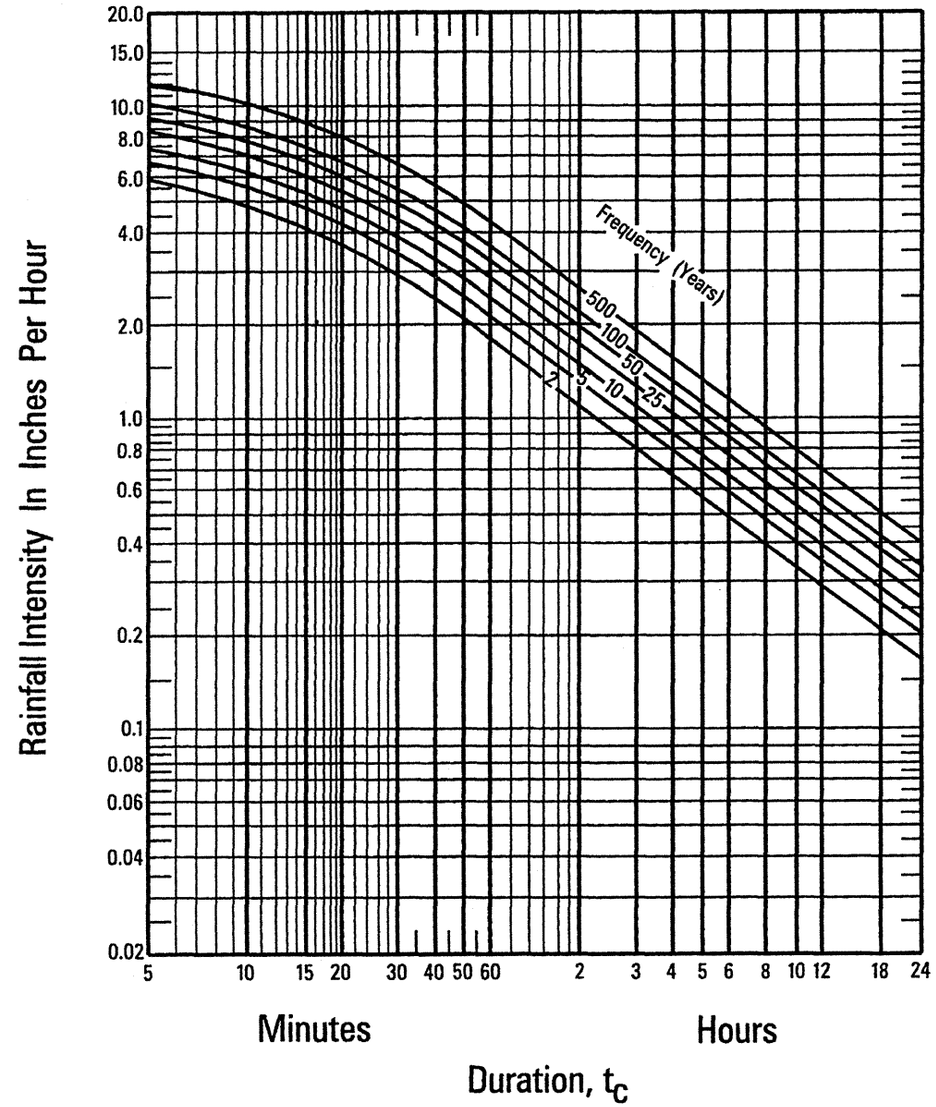
** East and Coastal < 64 acres; West < 38 acres; Delta < 70 acres.*

Note: Use the following "C" values:

Pavement	*C = 0.90*
Paved Shoulder	*C = 0.90*
Non-paved Shoulder	*C = 0.70*
Side Slopes	*C = 0.70*
Urban, Average	*C = 0.66*
Rural, Average	*C = 0.50*

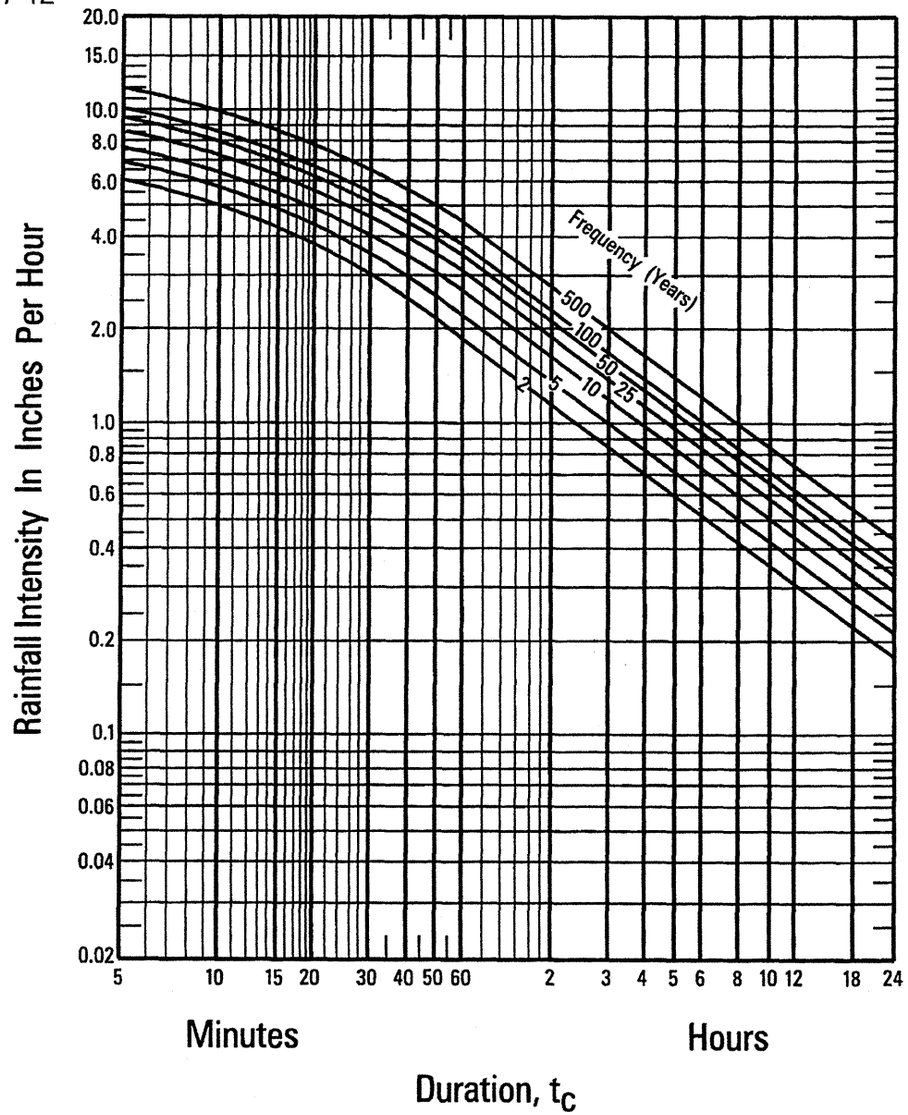


MISSISSIPPI RAINFALL REGIONS
Figure 7-4A

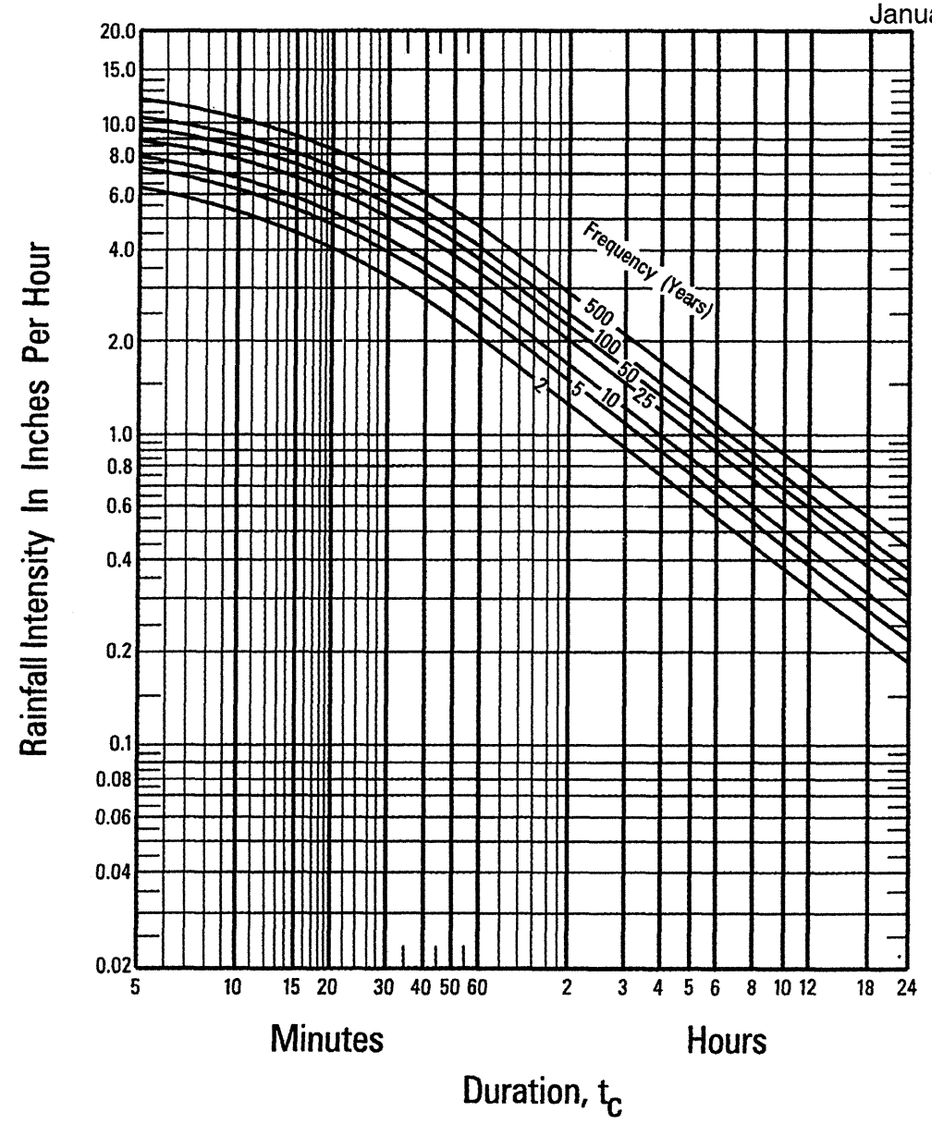


Note: Based on data for Memphis, Tennessee and Tupelo, Mississippi.

RAINFALL INTENSITY - DURATION - FREQUENCY CURVES
Northern Region — Region I
Figure 7-4B

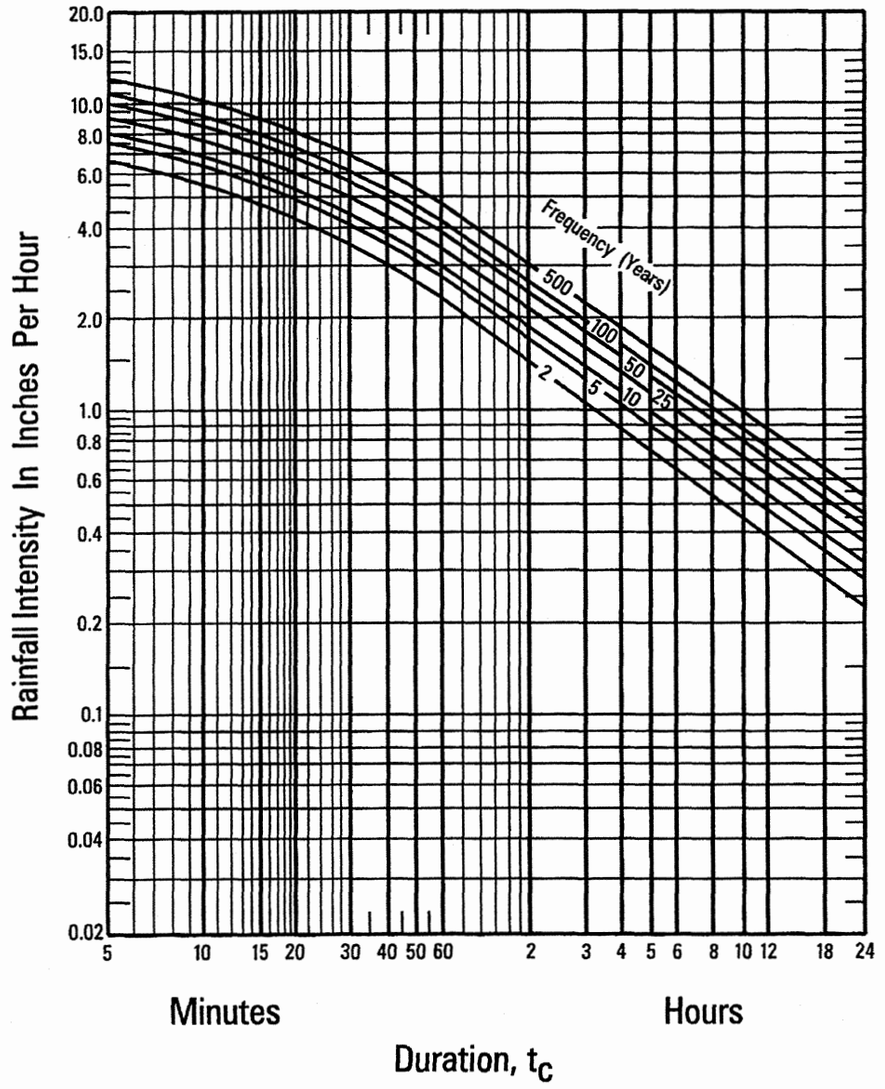


RAINFALL INTENSITY - DURATION - FREQUENCY CURVES
Region I and Region II Average
Figure 7-4C

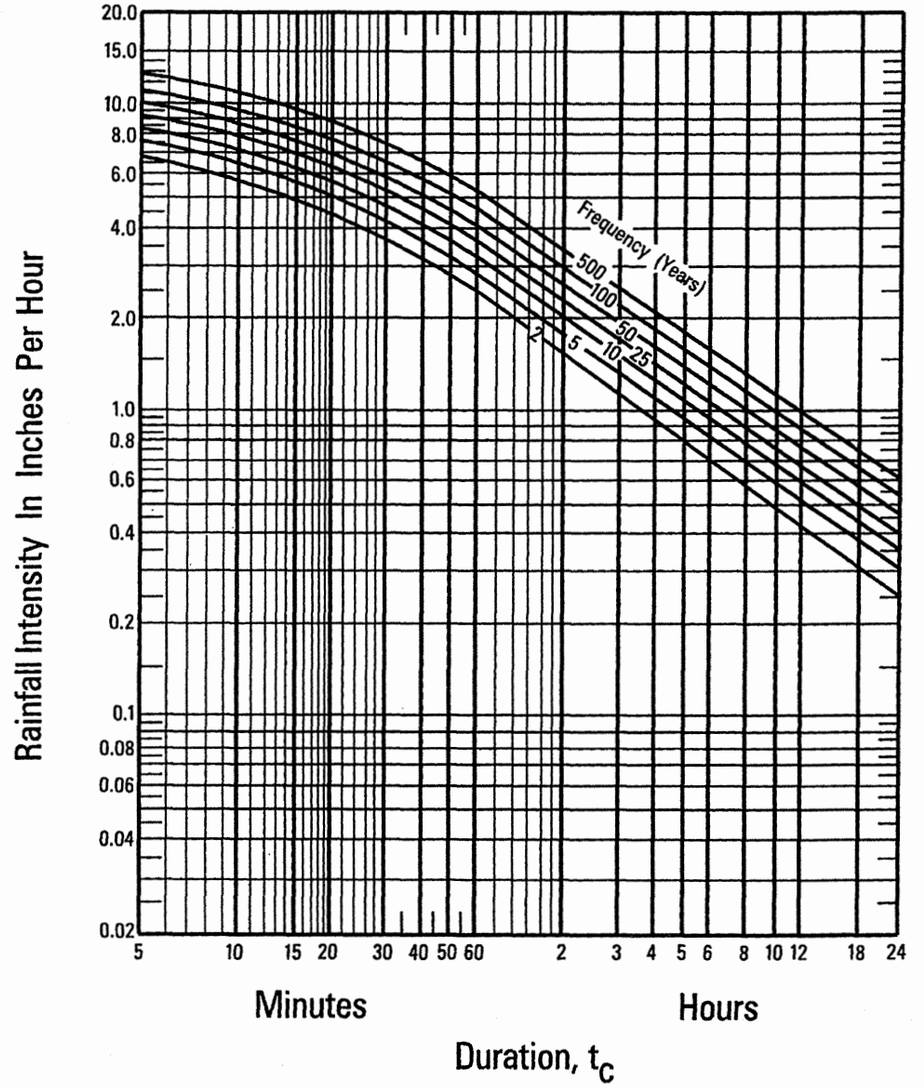


Note: Based on data for Vicksburg, Jackson and Meridan, Mississippi.

RAINFALL INTENSITY - DURATION - FREQUENCY CURVES
Central Region — Region II
Figure 7-4D

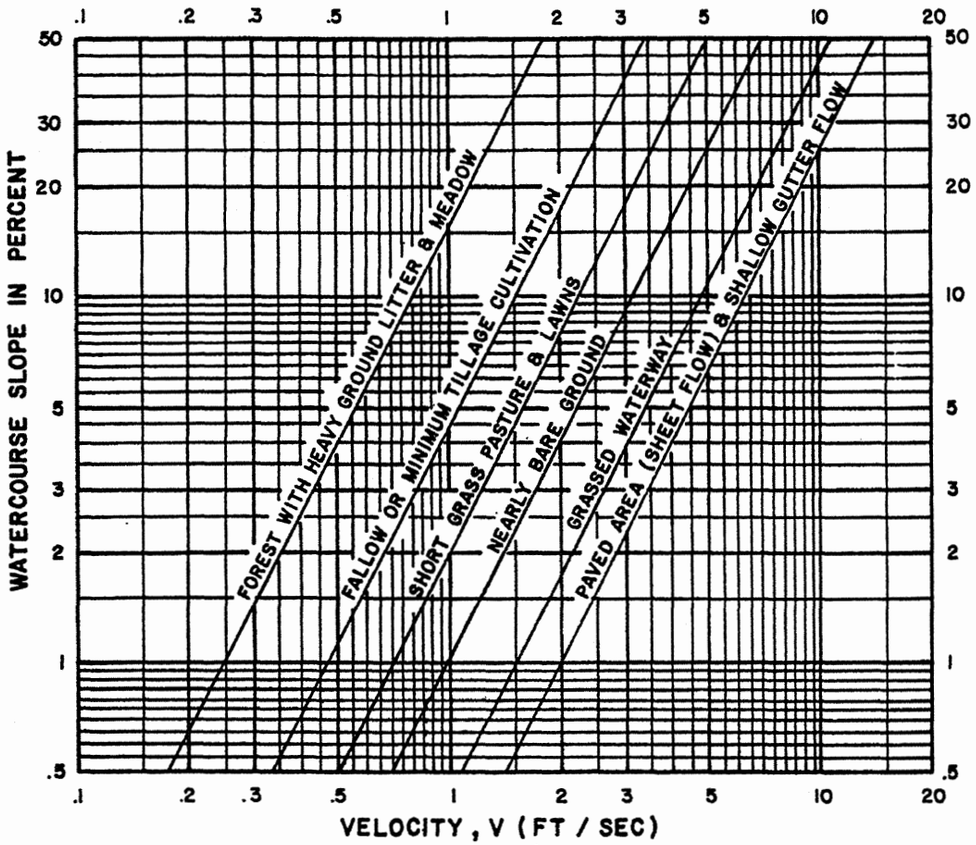


RAINFALL INTENSITY - DURATION - FREQUENCY CURVES
Region II and Region III Average
Figure 7-4E

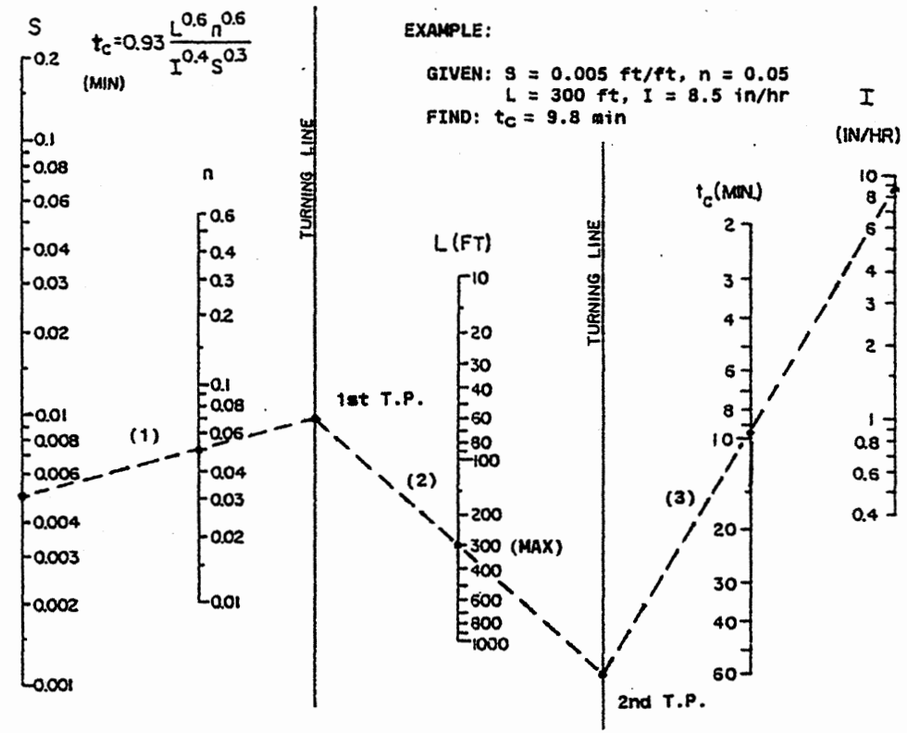


Note: Based on data for Gulfport, Biloxi and Pascagoula, Mississippi.

RAINFALL INTENSITY - DURATION - FREQUENCY CURVES
Southern Region — Region III
Figure 7-4F



AVERAGE VELOCITIES FOR OVERLAND FLOW
Figure 7-4G



Example: Given; slope of drain (s) = 0.005 ft/ft, n = 0.05, length of travel (L) = 300 ft, I = 8.5 in/hr. Find; t_c = 9.8 minutes.

1. *Aline S = 0.005 and n = 0.05 to first turning point.*
2. *Aline first point and L = 300 ft to second turning point.*
3. *(At 300 ft maximum travel, $t_c \leq$ 10 minutes, first trial aline second point with $t_c \leq$ 10 minutes, read I, compare t_c and I to I-D-F design curve values, vary I to match curve values.) Aline second point and I = 8.5 in/hr, read t_c = 9.8 minutes.*

Note: See Table 7-4B for "n" values. Use for sheet flow maximum 300 ft travel only, greater — use shallow concentrated flow.

KINEMATIC WAVE FORMULATION* FOR DETERMINING TIME OF CONCENTRATION (t_c)
Figure 7-4H

Table 7-4B
ROUGHNESS COEFFICIENTS (MANNING'S n) FOR SHEET FLOW

Surface Description	n value[1]
Smooth Surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

Notes:

1. *The n values are a composite of information complied by Engman (1986).*
2. *Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.*
3. *When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.*

7-4.02 Method of USGS 1991 Report "Flood Characteristics of Mississippi Streams" (Rural Discharge)

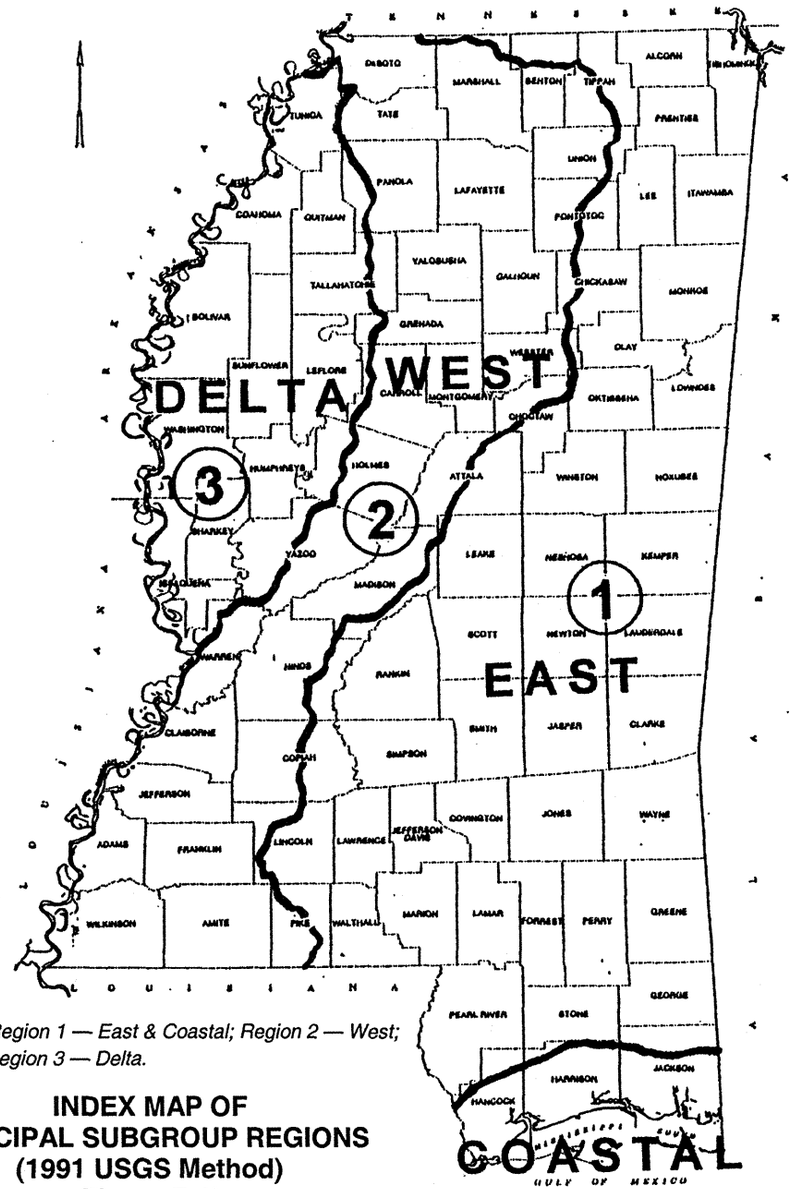
7-4.02.01 Definition

This method of discharge determination is suggested for use on rural, ungaged, natural drainage basins having drainage areas between 0.06 mi^2 (38 acres) and larger up to 800 mi^2 (512,000 acres). The flood characteristics of the State are delineated into three Principal Subgroup Regions with respective discharge equations — (1) East, (2) West and (3) Delta. See Figure 7-4I. Table 7-4C presents limitations of regional flood-frequency estimates and Table 7-4D presents the discharge equations for the three Regions.

7-4.02.02 Procedure

1. Refer to Figure 7-4I and select the appropriate Region for the respective project-site location.
2. Determine the drainage area, in square miles, from quadrangle maps, aerial photographs or field survey data. Compare drainage area values to the acceptable ranges in Table 7-4C for use of the 1991 USGS Report.
3. Determine the length of the main-channel in miles.
4. Determine the drainage channel slope in feet per mile.
5. Select the appropriate recurrence interval discharge equation from Table 7-4D. Apply the drainage area, channel slope and channel length to derive the desired peak discharge, in cubic feet per second.

Example 7-4.2 illustrates the determination of design discharges by the method in the 1991 USGS Report.



Note: Region 1 — East & Coastal; Region 2 — West; Region 3 — Delta.

INDEX MAP OF PRINCIPAL SUBGROUP REGIONS (1991 USGS Method)
Figure 7-4I

Table 7-4C
LIMITATIONS OF REGIONAL FLOOD-FREQUENCY ESTIMATES (1991 USGS Report)

Region	Basin Characteristic	Minimum	Maximum
(1) East & Coastal*	Area (square miles) Channel Slope (feet/mile) Channel Length (miles)	0.10 (64 acres) 1.5 0.4	799 170 123
(2) West	Area (square miles) Channel Slope (feet/mile) Channel Length (miles)	0.06 (38 acres) 2.3 0.3	654 192 70.7
(3) Delta	Area (square miles) Channel Slope (feet/mile) Channel Length (miles)	0.11 (70 acres) 0.4 0.5	1170 10.6 269

** Coastal: Limit to 20 miles inland; use no greater than 0.20% slope.*

Table 7-4D
1991 USGS DISCHARGE EQUATIONS (RURAL DISCHARGE)

Region (3), Delta (≥ 70 acres)**	Region (2), West (≥ 38 acres)**	Region (1), East & Coastal * (≥ 64 acres) **
$RQ_2 = 171\ A^{0.87}\ S^{0.25}\ L^{-0.52}$	$RQ_2 = 66.2\ A^{0.88}\ S^{0.51}\ L^{-0.11}$	$RQ_2 = 296\ A^{0.81}\ S^{0.03}\ L^{-0.36}$
$RQ_5 = 192\ A^{0.93}\ S^{0.37}\ L^{-0.54}$	$RQ_5 = 94.7\ A^{0.93}\ S^{0.51}\ L^{-0.15}$	$RQ_5 = 406\ A^{0.84}\ S^{0.07}\ L^{-0.35}$
$RQ_{10} = 205\ A^{0.96}\ S^{0.42}\ L^{-0.56}$	$RQ_{10} = 122\ A^{0.96}\ S^{0.49}\ L^{-0.19}$	$RQ_{10} = 482\ A^{0.85}\ S^{0.09}\ L^{-0.34}$
$RQ_{25} = 224\ A^{0.99}\ S^{0.48}\ L^{-0.58}$	$RQ_{25} = 164\ A^{0.99}\ S^{0.47}\ L^{-0.24}$	$RQ_{25} = 577\ A^{0.85}\ S^{0.10}\ L^{-0.32}$
$RQ_{50} = 232\ A^{1.00}\ S^{0.52}\ L^{-0.57}$	$RQ_{50} = 197\ A^{1.00}\ S^{0.45}\ L^{-0.26}$	$RQ_{50} = 648\ A^{0.85}\ S^{0.11}\ L^{-0.31}$
$RQ_{100} = 236\ A^{1.00}\ S^{0.57}\ L^{-0.55}$	$RQ_{100} = 230\ A^{1.00}\ S^{0.45}\ L^{-0.25}$	$RQ_{100} = 716\ A^{0.85}\ S^{0.11}\ L^{-0.30}$
$RQ_{200} = 243\ A^{1.00}\ S^{0.60}\ L^{-0.54}$	$RQ_{200} = 262\ A^{1.00}\ S^{0.42}\ L^{-0.25}$	$RQ_{200} = 786\ A^{0.85}\ S^{0.12}\ L^{-0.29}$
$RQ_{500} = 249\ A^{1.00}\ S^{0.64}\ L^{-0.52}$	$RQ_{500} = 305\ A^{1.00}\ S^{0.41}\ L^{-0.25}$	$RQ_{500} = 874\ A^{0.85}\ S^{0.12}\ L^{-0.28}$

Note: Note the negative exponents on length values (L) in equations.

* Coastal*: Limit to 20 miles inland; use no greater than 0.20% slope.*
** *For drainage areas less than these values, use the Rational Method to determine discharges.*

Where:

RQ_t = *The estimated peak discharge, in cubic feet per second, for a recurrence interval of t years.*

A = *The contributing drainage area, in square miles.*

S = *The channel slope, in feet per mile, defined as the difference in elevation between points located at 10% and 85% of the main channel length divided by the channel length between the two points, as determined from topographic maps.*

L = *The main-channel length, in miles, from the point of discharge to the drainage divide as measured in 0.1 mile increments on topographic maps. At a stream junction, the branch draining the largest area is considered the main channel.*

Example 7-4.2 (Rural Discharge)

The following four examples illustrate the use of the 1991 USGS Report to calculate rural discharge.

1.	Arterial project:	Sharkey County, Region (3), Delta
	Drainage area	= 200 acres = 0.3125 mi^2
	Channel slope	= appx. 0.15% = 7.92 ft/mi (use actual slope)
	Channel length	= appx. 3300 ft = 0.625 mi
	Design frequency	= 50-year recurrence
	Design discharge	= 277.99 ft^3/s
2.	Arterial project:	Madison County, Region (2), West
	Drainage area	= 200 acres = 0.3125 mi^2
	Channel slope	= appx. 1.00% = 52.80 ft/mi
	Channel length	= appx. 3300 ft = 0.625 mi
	Design frequency	= 50-year recurrence
	Design discharge	= 414.55 ft^3/s
3.	Arterial project:	Neshoba County, Region (1), East
	Drainage area	= 200 acres = 0.3125 mi^2
	Channel slope	= appx. 1.00% = 52.80 ft/mi
	Channel length	= appx. 3300 ft = 0.625 mi
	Design frequency	= 50-year recurrence
	Design discharge	= 431.48 ft^3/s
4.	Arterial project:	Harrison County, Region (1), East-Coastal*
	Drainage area	= 200 acres = 0.3125 mi^2
	Channel slope	= appx. 0.15% = 7.92 ft/mi
	Channel length	= appx. 3300 ft = 0.625 mi
	Design frequency	= 50-year recurrence
	Design discharge	= 350.21 ft^3/s

** Coastal: Limit to 20 miles inland; use no greater than 0.20% slope.*

7-4.03 Method of USGS 1983 Water-Supply Paper 2207 "Flood Characteristics of Urban Watersheds in the United States"

7-4.03.01 Definition

In combination with the 1991 USGS Report on "Flood Characteristics of Mississippi Streams" for rural natural drainage basins where the discharge is notated as RQ_t, the 1983 USGS Method uses the RQ_t value to derive the estimated design discharge from urbanized watersheds notated as UQ_t.

For an urbanized drainage basin or one having that potential during the useful life of a highway drainage system, the rural discharge, RQ_t, is first determined as described previously in Section 7-4.02. Then, with the appropriate seven-parameter estimating equation, the urban design discharge, UQ_t, is determined. Peak discharges for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year urban storms are related to seven independent variables by linear multiple-regression techniques. The seven equations are as follows (note negative exponents in equations):

1. $UQ_2 = 2.35A^{0.41}SL^{0.17}(RI2+3)^{2.04}(ST+8)^{-0.65}(13-BDF)^{-0.32}IA^{0.15}RQ_2^{0.47}$

2. $UQ_5 = 2.70A^{0.35}SL^{0.16}(RI2+3)^{1.86}(ST+8)^{-0.59}(13-BDF)^{-0.31}IA^{0.11}RQ_5^{0.54}$

3. $UQ_{10} = 2.99A^{0.32}SL^{0.15}(RI2+3)^{1.75}(ST+8)^{-0.57}(13-BDF)^{-0.30}IA^{0.09}RQ_{10}^{0.58}$

4. $UQ_{25} = 2.78A^{0.31}SL^{0.15}(RI2+3)^{1.76}(ST+8)^{-0.55}(13-BDF)^{-0.29}IA^{0.07}RQ_{25}^{0.60}$

5. $UQ_{50} = 2.67A^{0.29}SL^{0.15}(RI2+3)^{1.74}(ST+8)^{-0.53}(13-BDF)^{-0.28}IA^{0.06}RQ_{50}^{0.62}$

6. $UQ_{100} = 2.50A^{0.29}SL^{0.15}(RI2+3)^{1.76}(ST+8)^{-0.52}(13-BDF)^{-0.28}IA^{0.06}RQ_{100}^{0.63}$

7. $UQ_{500} = 2.27A^{0.29}SL^{0.16}(RI2+3)^{1.86}(ST+8)^{-0.54}(13-BDF)^{-0.27}IA^{0.05}RQ_{500}^{0.63}$

Where:

UQ_t = The estimated peak discharge for a recurrence interval of t years (cubic feet per second — ft^3/s).

A = Drainage area (square miles — mi^2).

SL = Channel slope (feet per mile — ft/mi). Note: For initial rural Q (RQ_t), use actual (10% and 85%) slope; for urban Q (UQ_t), use actual slope but not to exceed 70 ft/mi.

RI2 = The basin rainfall intensity for the 2-hour, 2-year occurrence (inches — in). See the Weather Bureau Technical Paper No. 40 (1961) or Miller and others (1973).

ST = The basin storage which includes the percentage of basin occupied by lakes, swamps, etc., resulting in runoff detention. The in-channel storage is not included (range: 0-11%).

BDF = The manmade changes to the drainage system including impervious channels, storm sewers, and curb and gutter. A range of 0-12 should be used where 0 indicates development not prevalent and 12 indicates full development (also, see IA).

IA = The percentage of the drainage basin occupied by other impervious surfaces including houses, buildings, streets and parking lots (range: 3 - 50%).

RQ_t = The peak discharge for an equivalent rural drainage basin in the same hydrologic area as the urban basin and the same recurrence interval of t years (cubic feet per second — ft^3/s).

7-4.03.02 Procedure

1. Refer to the equations for peak discharge for rural conditions, USGS 1991 Report on "Flood Characteristics of Mississippi Streams" (Section 7-4.02) and select the appropriate equation for the recurrence interval (t) desired. Follow procedures in Section 7-4.02 to derive an equivalent rural discharge (RQ_t) in the same hydrologic area as the subject urban basin.

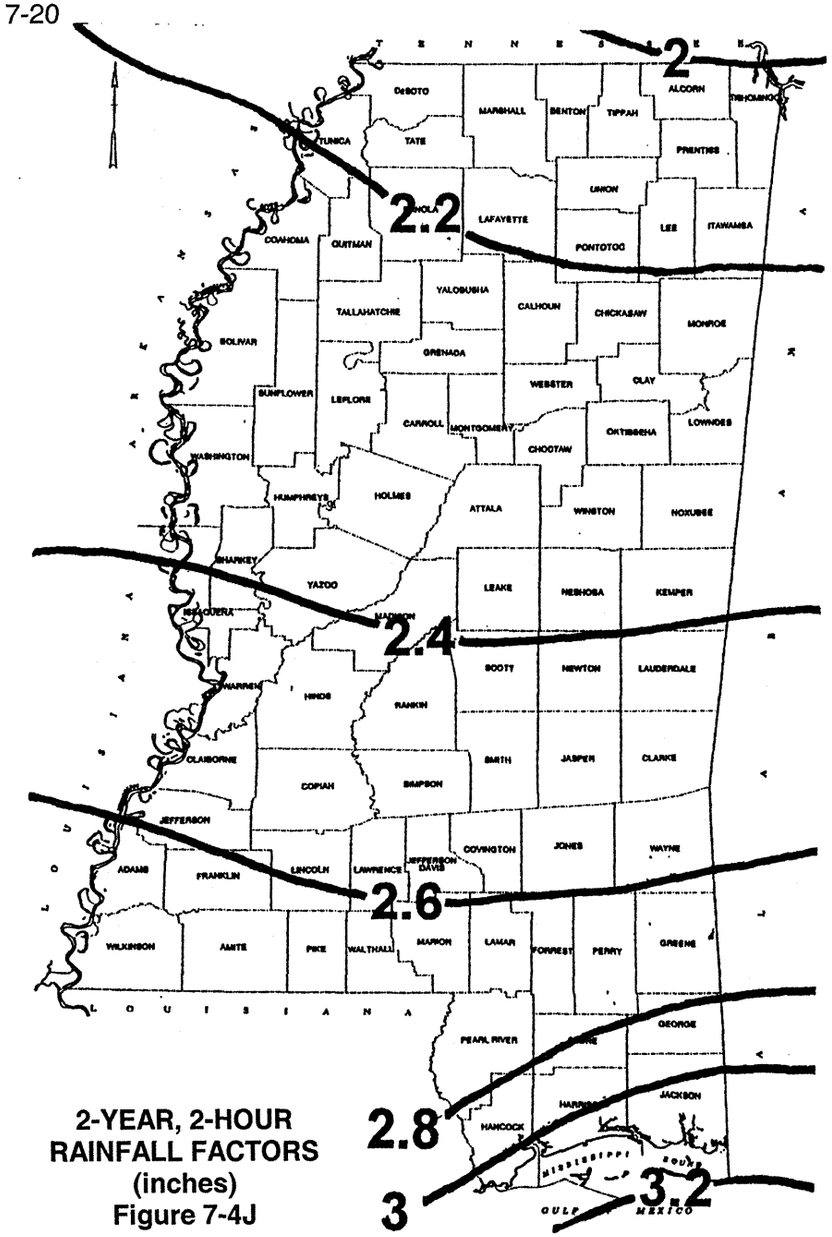
2. The drainage area, in square miles, and the channel slope, in feet per mile, will be established for the RQ_t and should be retained for further use in the UQ_t equation.

3. Refer to Figure 7-4J from the Weather Bureau Technical Paper No. 40 (1961) or Miller and others, NOAA Atlas 2, to determine the rainfall intensity, in inches, for the 2-hour, 2-year occurrence (RI2). The range varies from 2.0 to 3.2 for the subject area in Mississippi, and the value is added to a constant appearing in the equations.

4. Determine the basin storage factor (ST) in percent. Values range between 0.0 and 11.0 for acceptable computations representing the percentage of runoff storage or detection within the total basin in the form of lakes, swamps, etc. In-channel storage is not included. Although a percentage, the value is used as a whole number and is added to a constant appearing in the equations.

5. Determine the basin development factor (BDF) indicating the development of drainage elements within the basin such as storm sewers, channel improvements, impervious channel linings and curb and gutter streets. Values range from 0 (not prevalent) to 12 (full development), and the assigned value is subtracted from a constant in the equations.

6. Determine the percentage of other impervious surfaces (IA) within the drainage basin including houses, buildings, streets and parking lots. This value ranges between 3% and 50% for acceptable computations where the percentage amount is used as a whole number in the equations.



2-YEAR, 2-HOUR RAINFALL FACTORS (inches)
Figure 7-4J

7. Evaluate the appropriate recurrence interval discharge equation for an urban watershed (UQ_t) to derive the desired peak discharge in cubic feet per second.

Example 7-4.3 illustrates the determination of design discharge by the method of USGS 1983 Water-Supply Paper 2207.

Example 7-4.3

Given: Arterial project, Urban: Philadelphia, Neshoba Co., East Region
Drainage area = 200 acres
Channel slope = approximately 1.0%
Channel length = 3300 ft
Design frequency = 50-year recurrence for arterial projects

Problem: Determine the design discharge at the given location by the method of USGS 1983 Water-Supply Paper 2207.

Solution: The designer should follow the procedure established in Section 7-4.03.02.

1. From Example 7-4.2 (same design data), the rural peak discharge (RQ_t) was found to be 431 ft^3/s.

2. From Example 7-4.2, the drainage area was found to be 0.3125 mi^2 and the channel slope is 52.8 ft/mi.

3. From Figure 7-4J, the interpolated 2-hour, 2-year rainfall value is 2.38 in which is added to the constant (3) in the equation:
RI2 + 3 = 2.38 + 3 = 5.38.

4. The runoff storage or detention for this example is 4 acres in wetlands. Therefore, the basin storage factor is 2% (4 acres/200 acres = 0.02 or 2%); but use as the whole number 2.0) added to the constant (8) in the equation: ST + 8 = 2 + 8 = 10.0.

5. It is estimated that drainage development will be attained over 75% of the basin. Therefore, the basin development factor (0.75 x 12) = 9, which is subtracted from the constant (13) in the equation: 13 - BDF = 13 - 9 = 4.

6. It is estimated that other impervious surfaces cover 30% of the basin and the whole number, 30, is used in the equation.

7. The 50-year or UQ_{50} discharge is desired (note negative exponents in equation):

$$UQ_{50} = 2.67A^{0.29}SL^{0.15}(RI2+3)^{1.74}(ST+8)^{-0.53}(13-BDF)^{-0.28}IA^{0.06}RQ_{50}^{0.62}$$

$$UQ_{50} = (2.67)(0.3125^{0.29})(52.8^{0.15})(2.38+3)^{1.74}(2+8)^{-0.53}(13-9)^{-0.28}(30^{0.06})(431^{0.62})$$

UQ_{50} = 681.4 ft^3/s (regional adjustments were made in RQ_t; the designer may proceed to culvert design).

7-4.04 Method from USGS 1967 Report on "Urbanization Effects on Floods in Jackson, Mississippi"

7-4.04.01 Definition

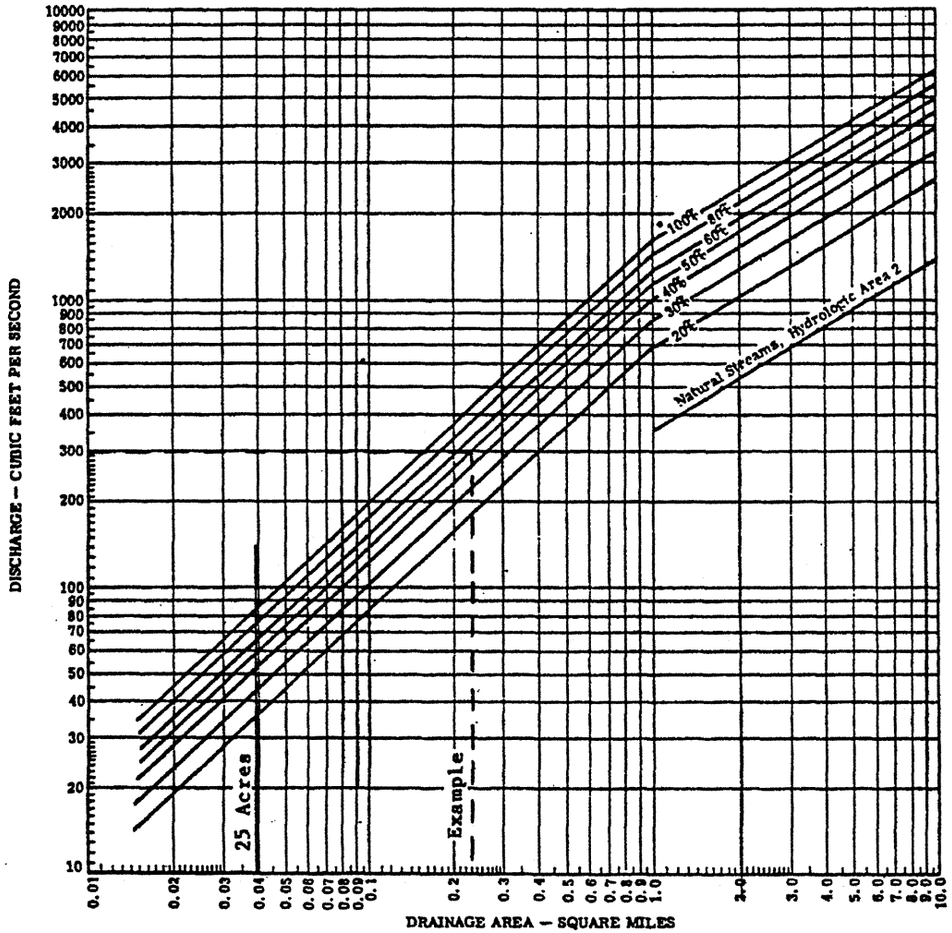
This report, "Urbanization Effects on Floods in Jackson, Mississippi," is a method for determining urban discharge from 25-acre and larger drainage areas. Mean Annual Flood curves have been developed on the basis of certain percentages of the drainage basin having storm sewers and improved channels. The 50% to 60% curves represent average urbanized development. Paved commercial type areas can be represented by the 100% curve. The designer should strive to accurately select the appropriate curve with consideration of probable future use.

Note: Generalizations of drainage characteristic parameters are made in this method. When complete data is available, consider Method #3, USGS 1983 Report on "Flood Characteristics of Urban Watersheds in the United States."

7-4.04.02 Procedure

1. Consider the drainage area, its present development and potential future use. Select the most representative curve in Figure 7-4K based on the percentage of the drainage area with storm sewers and improved channels. Enter the horizontal (abscissa) scale with the drainage area in square miles and vertically intersect the appropriate percentage curve. From this point, project horizontally to the vertical (ordinate) scale and read the mean annual discharge in cubic feet per second.

2. Refer to Figure 7-4L to adjust the discharge value for the desired storm frequency. Enter the horizontal scale of the figure at the desired frequency interval (years), read vertically to the intersection with the curve and project horizontally to read the ratio value.



Notes:

1. *Used for urban drainage areas > 25 acres.*
2. *Curves represent percent of basin with storm sewers and improved channels.*
3. *Unless drainage area is extremely small and all paved (e.g., shopping center), the curves for 50% - 60% will yield a valid representation for urbanized areas.*

MEAN ANNUAL FLOOD CURVES
CITY OF JACKSON VICINITY
(USGS 1967 Report)
Figure 7-4K

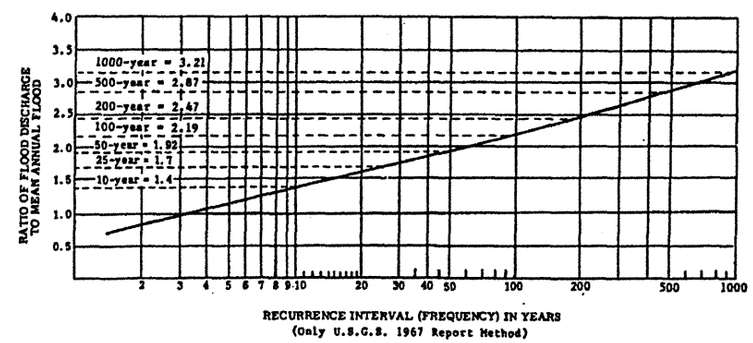
Example 7-4.4

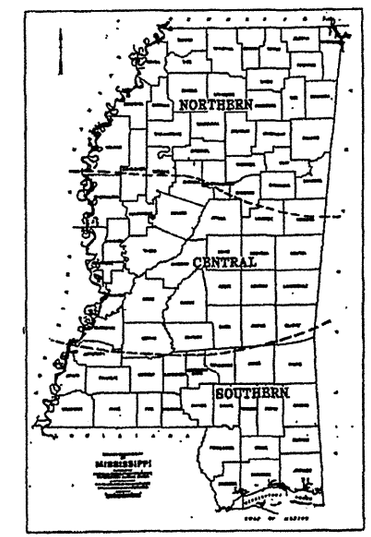
Given: *Harrison County (coastal)*
Drainage area = 147 acres
Development = nominal basin improvements

Problem: *Determine the 50-year discharge (Q_{50}) by the method from the USGS 1967 Report.*

Solution:

1. *Enter Figure 7-4K at drainage area scale in square miles, 147 acres/640 acres/mi^2 = 0.23 mi^2.*
2. *Vertically intersect 50% curve, project horizontally to read discharge (Mean Annual Flood) = 300 ft^3/s.*
3. *Enter Figure 7-4L at 50-year recurrence interval.*
4. *Vertically intersect curve, project horizontally to read ratio value = 1.92.*
5. *(MAF) (ratio value) = (300) (1.92) = 576 ft^3/s.*
6. *Adjust for southern regional location: (576) (1.08) = 622 ft^3/s.*
7. *Adjust for flat coastal slope: (622) (0.77) = 479 ft^3/s. (Q_{50} design discharge).*





Regional Conversion Factors:
Northern ⅓ of State — (Discharge)(0.88) (reduction)
Central ⅓ of State — Direct Chart Values
Southern ⅓ of State — (Discharge)(1.08) (increase)

Note: *For very flat lands of slope less than 0.40% (e.g., delta and coastal lands), use only 77% of adjusted discharge.*

DISCHARGE ADJUSTMENT VALUES FOR
DESIGN STORM FREQUENCY
(USGS 1967 Report)
Figure 7-4L

3. The product of the mean annual flood (from Figure 7-4K) and the ratio value (from Figure 7-4L) yields the design discharge in cubic feet per second.

4. Design discharges for urban areas computed by this method are directly applicable for the central one-third of the State. For other areas of the State, refer to the regional conversion factors provided in Figure 7-4L for a final adjustment of the design discharge. In addition, for very flat lands of slope less than 0.40% (e.g., delta and costal lands), use only 77% of the adjusted discharge.

Examples 7-4.4 and 7-4.5 illustrate the determination of design discharge by the method from the USGS 1967 Report.

Example 7-4.5

Given:	Arterial project:	Jackson, Mississippi; urban
	Drainage area	= 1.16 mi^2
	Development	= 50% storm sewers and improved channels
	Design frequency	= 50-year recurrence for arterial projects

Problem: Determine the design discharge at the given location by the method from the USGS 1967 Report.

Solution: The designer should follow the procedure in Section 7-4.04.02.

1. Enter the horizontal scale of Figure 7-4K at 1.16 mi^2 read upward to the 50% curve, project horizontally left to the vertical scale and read a discharge of 1200 ft^3/s.

2. Enter the horizontal scale of Figure 7-4L at the 50-year frequency, read vertically to the curve, project horizontally to the vertical scale and read the ratio value of 1.92.

3. Q_{Design} = (Mean Annual Flood)(Ratio Value)
   = (1200) (1.92) = 2304 ft^3/s

4. From the regional map in Figure 7-4L, Jackson, Mississippi is in the central one-third of the State. Therefore, no conversion factor is necessary, and Q = 2304 ft^3/s should be used in design.

7-4.05 USGS Individual Stream Study Reports

USGS Individual Stream Study Reports (generally of areas larger than 1000 acres) may be used directly to identify discharge. Even if not used directly, they often indicate runoff discharge conditions for similar areas in the general vicinity of the study report. The Quiver River Report, for example, indicates runoff characteristics typical of the Mississippi Delta, and it is used extensively for delta lands.

7-5.0 CULVERT DESIGN PRACTICES

Previous sections have provided the designer with various methods for estimating the design discharge at culvert locations. The next step in the drainage design process is to design culverts to fit the specific location conditions. The following sections present the general criteria and design practices to be considered in the design process.

7-5.01 Minimum Size Culverts

The Department has established minimum size culverts for particular drainage installations. The minimum dimensions will apply even when the culvert capacity exceeds the design discharge requirements. Table 7-5A provides the minimum culvert size based on the highway classification and the various types of installation.

Table 7-5A
MINIMUM SIZE CULVERTS

Functional Class	Box Culverts	Pipe Culverts				
		Cross Drains	Median Drains[2]	Side Drains	Slope Drains	Stubouts
Freeways	4′x 4′	24″	18″	18″	18″	18″
All other roads	4′ x 4′[1]	24″	18″	18″	18″	18″

Notes:

1. *If approved, smaller existing box culverts may be extended.*
2. *Includes pipes through crossovers.*

7-5.02 Culvert Discharge Limits

Table 7-5B provides recommended limits of discharge for the selection of the type of culvert for use in design.

The design of 18-in pipe culverts and arch pipe equivalents usually will require only verification of the allowable headwater (HW) and documentation in the project file to fulfill the design requirements. Pipe culverts and arch pipe equivalents of 24-in diameter and larger shall have inlet and outlet control analyses and other recorded information (see Section 7-6.0).

From Table 7-5B, the nominal maximum design discharge for a double line of large pipe culverts is approximately 700 ft^3/s. Additional lines of pipe should be investigated when conditions warrant. Allowable headwater elevations usually govern these limitations.

Table 7-5B
RECOMMENDED CULVERT DISCHARGE LIMITS

Design Discharge (ft^3/s)	Type of Culvert
≤ 300	Pipe Culverts
300 - 700	Pipe culverts recommended; however, either multiple pipes or box culverts may be used.
> 700	Multiple pipes or box culverts are permitted.

7-5.03 Economic Comparisons

In some cases, single and multiple lines of pipe culverts can be substituted for box culverts with considerable savings in the cost of material and the time of construction. There will be specific conditions where a box culvert is the more practical structure (e.g., large size, excessive cover, inadequate cover, soil conditions, cattle and equipment passes). When an alternative selection between box culverts and pipe culverts exists, the designer should conduct an economic comparison and record the results. This comparison should consider the initial structure installation costs, the life expectancy and the replacement cost of different types of culverts.

7-5.04 Pipe Culvert Alternatives

Department policy requires Reinforced Concrete Pipe (RCP) for most pipe culvert installations. Under certain conditions, alternative pipe culverts may be permitted or

required. Table 7-5C provides criteria for the required design life and type of pipe culvert based on the functional class and type of culvert installation.

Table 7-5C
TYPE OF PIPE CULVERT INSTALLATION

Functional Classification	Required Design Life			
	Cross [3] Drains	Median Drains	Special Systems, Storm Sewers, Siphons, etc.	Side Drains
Freeways and Arterials	50-year [1]	50-year [1]	50-year [1]	Urban: 50-year [2] Rural: 25-year [2]
Collector/Local	50-year [2]	50-year [2]	50-year [1]	Urban: 50-year [2] Rural: 25-year [2]

Notes:

1. *Reinforced Concrete Pipe (No Alternatives), 50-year minimum.*

2. *Four alternatives exist (soil test permitting) (Federal-aid projects require three alternatives):*

   a. *Reinforced Concrete Pipe,*
   b. *Corrugated (Ferrous) Metal Pipe (with appropriate design life properties),*
   c. *Corrugated Aluminum Pipe (with appropriate design life properties), and*
   d. *Plastic Pipe — Polyethylene and Poly Vinyl Chloride.*

3. *Where alternatives for cross drains exist, consider RCP up to 36-in diameter. When pipe is 36-in diameter or larger, alternatives of either RCP or 3-in x 1-in or 5-in x 1-in Corrugated (Ferrous) Metal Pipe are more comparable.*

4. *The elimination of one or more of the pipe alternatives may be warranted in extraordinary installation site conditions (e.g., unstable support, steep gradient, high embankment, acidity or alkalinity of soil, other corrosive elements, high potential for abrasion or erosion), and these reasons well documented.*

5. *Concrete end sections required on all metal cross drains.*

Where alternatives for cross drains exist, consider RCP up to 36-in diameter. When pipe is 36-in diameter or larger, alternatives of either RCP or 3-in x 1-in or 5-in x 1-in corrugated ferrous metal pipe is more comparable.

7-5.05 Type and Placement of Pipe

Lower headwater values can be achieved with arch pipe. Designers should consider the use of arch pipe where the cover, outfall elevation or gradient is restrictive.

Smaller or lower profile drainage culverts are preferred in the vicinity of residential and farm lands. The culverts should be placed on a flow line similar to the culvert height below the adjacent lands. This will permit effective culvert performance and avoid detrimental ponding on the drainage area.

7-6.0 CULVERT SIZE DETERMINATION

Procedures for determining culvert sizes and for documenting the design are discussed in this section. The Federal Highway Administration publication *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5, September 1985, is a recommended design reference. This publication combines design information previously contained in Hydraulic Engineering Circular (HEC) No. 5 *Hydraulic Charts for the Selection of Highway Culverts*; HEC No. 10 *Capacity Charts for the Hydraulic Design of Highway Culverts;* and HEC No. 13 *Hydraulic Design of Improved Inlets for Culverts* with other more recently developed culvert information. The basic procedure for determining the appropriate culvert size design is provided below. Detailed guidelines and figures are presented in the following sections.

1. For discharges less than 300 ft^3/s refer to the Headwater Depth Chart (Figure 7-6C or 7-6D); otherwise, divide the Q_{50} discharge (cubic feet per second) by 10 ft/s to determine a trial culvert area of opening (pipe or box culvert).

2. Consider the allowable headwater depth (normally the level of the subgrade). Using Headwater Depth Charts provided in this section, select trial culverts with nominal HW/D values of 1.2 to 2.0 for inlet control conditions. Derive headwater (HW) depths for Q_{Design} and Q_{100}.

3. Use the Head for Culvert Charts (Figure 7-6G or 7-6H) and the Critical Depth Charts (Figure 7-6E or 7-6F) to determine outlet control acceptance of trial culverts.

4. When the culvert is in inlet control, consider tapered or improved entrances. Refer to Hydraulic Design Series No. 5 to investigate the most satisfactory culvert size and/or entrance configuration.

7-6.01 Inlet and Outlet Control Culvert Flow

The designer should consider the specific characteristics of inlet and outlet control in culvert size design. These controls influence the upstream water buildup or headwater (HW) necessary for a particular culvert to pass a specific discharge.

Inlet control is the culvert operation when the flow capacity of a culvert is controlled at the entrance by the depth of headwater (HW) and the entrance geometry, including the shape and cross-sectional area of the barrel and type of inlet edge.

Outlet control is the culvert operation when, combined with the inlet control factors, flow through the culvert is controlled by the barrel size, slope, length and roughness, and the outlet tailwater elevation.

The design of all pipe and box culverts equivalent to 24-in diameter and larger should consider these control characteristics. Figure 7-6A illustrates a sample culvert design form where design computations should be recorded for documentation.

7-6.02 Improved Culvert Entrances

For culverts operating in inlet control, improved entrance structures for pipe and box culverts may increase efficiency and/or reduce the required culvert size. Improved entrance structures on existing culverts may increase the culvert capacity sufficiently to accommodate increased discharges resulting from land-use developments. Types of improved inlets for pipe and box culverts are provided in Figure 7-6B. Improvements include beveled entrance edges (without major change from the existing practice) and special design entrance structures. Hydraulic Design Series No. 5 presents details for design. Ordinarily, the design of improved entrance structures will be performed by the Hydraulics Section or hydraulic design specialists.

When the outlet control HW is acceptable and the inlet control HW is greater, entrance improvements should be investigated in Hydraulic Design Series No. 5, as indicated above (see improved inlet/entrance curves by FHWA at end of chapter). However, when the outlet control HW is acceptable but the inlet control HW is less, the culvert operates in outlet control at design discharge, and culvert entrance improvements would be of no advantage. Improved entrance structure computer analysis is available and should be used as routine practice.

7-6.03 Size Design for Concrete Pipe Culverts

The objective of size design for concrete pipe culverts is to select an efficient culvert to convey a given discharge at an allowable headwater (HW) depth. When the limits of design discharge indicate trial of pipe culverts, the inlet control and outlet control headwater elevations are determined. The detailed analysis procedures for headwater calculations are provided in the following sections.

7-6.03.01 Inlet Control — Concrete Pipe Culverts

The following steps should be used for inlet control analysis of pipe culverts. Data should be recorded on the design form illustrated in Figure 7-6A.

1. Establish the culvert entrance flow line and the allowable headwater (HW_i) depth (subgrade level or other controls). Note the permissible culvert height.

2. Refer to Figure 7-6C or D, Headwater Depth for Concrete Pipe or Arch Culverts with Inlet Control. Select the Headwater Depth in Diameters (HW/D) scale based on culvert entrance type. Scale (1) is the turning line for the HW/D scales. Readings on scales (2) and (3) are projected horizontally to scale (1) for the correct turning point back to the Discharge and Diameter scales. Conversely, intersecting points on the turning line are projected horizontally to scales (2) and (3).

3. From the nominal value of 1.5 on the appropriate HW/D scale, project horizontally to scale (1) for the turning point. Align that point with the design discharge on the Discharge (Q) scale. Extend that straight line to intersect the Diameter of Culvert (D) scale and read to the closest pipe diameter required. Record the pipe size and compare to the permissible culvert height.

4. Check the selection by aligning the required pipe diameter on the (D) scale with the design discharge on the (Q) scale. Intersect the HW/D scale (1) turning line, and project horizontally to the proper HW/D scale. Nominal HW/D values range between 1.2 and 2.0, but they can be less than 1.0 and above 6.0 (allowable headwater permitting). Record the HW/D value under the HW_i/D column.

5. To obtain the inlet headwater (HW_i) depth, multiply HW_i/D by the pipe diameter. HW_i depth can be above allowable only when outlet control and an improved culvert entrance are analyzed. Otherwise, try a larger or lower profile culvert. Record the HW_i depth on the design form and proceed to outlet control analysis.

Example 7-6.1 illustrates the use of this procedure to determine the inlet headwater depth for a pipe culvert with a given design discharge.

Example 7-6.1

Given: Design discharge = 78 ft^3/s
6-ft maximum HW depth

Problem: Determine the proper size concrete pipe culvert for the given location (inlet control).

Solution: The designer should follow the procedure for inlet control presented in Section 7-6.03.01. This example is shown on the Culvert Design Form, Figure 7-6A.

1. The allowable headwater (HW_i) depth is given as 6 ft (subgrade level). A permissible culvert height is probably less than 5 ft.

2. Refer to Figure 7-6C. The entrance type is either a groove end with headwall or a flared end section (not shown on chart). Use HW/D scale (2).

3. Project horizontally from 1.5 on the HW/D scale (2) to scale (1). Align that point with the design discharge Q = 78 ft^3/s. Extend that straight line to the D scale intersecting between a 36-in and a 42-in pipe. Try a 36-in pipe size and record this value on the design form.

4. Align the 36-in pipe on D scale with 78 ft^3/s on Q scale to intersect the HW/D scale (1); project horizontally to scale (2); read 1.92. Record the HW/D value in the HW_i/D column.

5. Multiply the HW_i/D value (1.92) by the pipe diameter (D = 3.0 ft), assigning a headwater (HW_i) requirement of 5.76 ft for this pipe to convey the design discharge.

(*Continued*)

CULVERT DESIGN FORM — SHEET 1 OF 1

SQUAD R.W. DEZYNER — DESIGNER A.L. WATERMAN — DATE 7/23/95 — PROJECT 11-0002-06-006-10 — HWY NO. 45 — COUNTY LEE

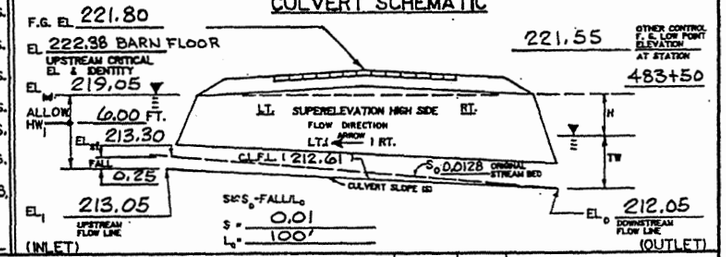
HYDROLOGICAL DATA — METHOD RATIONAL METHOD

- DRAINAGE AREA 32 AC.; 0.0500 SQ. MI.
- DRAIN LENGTH (100 % / 75 %) 1,760 FT.; 0.3333 MI.
- EL. MAX. (100 % / 90 %) 230.9 EL. MIN.; (0 % / 10 %) 213.30
- DRAIN SLOPE 0.01 FT./FT.; 52.80 FT./MI.
- TIME OF CONCENTRATION (t_c) 18.23 MIN.
- DESIGN INTENSITY (I_{DESIGN}) $Q_{DES.}$ 6.1 / Q_{100} 6.7 / Q_{25} 3.6 IN./HR.
- RATIONAL RUNOFF (C) 0.40
- INTENSITY-DURATION REGION #I
- CHANNEL SHAPE TRAPEZOIDAL; 3.5' BOTTOM; 1:1 SIDES

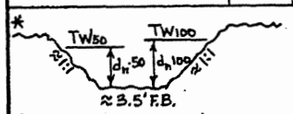
- $Q_{(DESIGN-50)}$ 78 C.F.S.
- $Q_{(BASE-100)}$ 86 C.F.S.
- $Q_{(OVERTOPPING-500)}$ 102 C.F.S.
- $Q_{(BLA.E.-2)}$ 46 C.F.S.
- Q_{25}, I = 5.6 IN/HR, Q = 72 C.F.S.
- Q_{10}, I = 4.8 IN/HR, Q = 61 C.F.S.
- Q_5, I = 4.4 IN/HR, Q = 56 C.F.S.
- CULVERT IN PLACE NONE - INITIAL ALINEMENT

STATION 482+35 — SKEW 15°; LT. FWD. — WORK SHEET NUMBER 5

CULVERT SCHEMATIC



CULVERT DESCRIPTION: SHAPE-ENTRANCE-MATERIAL	SIZE	TOTAL FLOW Q (CFS)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS — INLET CONTROL: HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL: TW* (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	L_oS	HW_{ho} (8)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY (ft. p. s.) (9)	COMMENTS
ROUND-GROOVE-R.C.P.	36"	78	-	1.92	5.76'	0.25'	218.81	2.27'	2.85'	2.93'	2.93'	0.2	3.44'	1.00'	5.37'	218.41	218.81	11.1	Q DES. VERY NEAR AHW.; ACCEPTABLE; CHECK Q100.
" " "	"	86	-	2.17	6.51'	"	219.56	2.38'	2.92'	2.96'	2.96'	"	4.18'	"	6.14'	219.19	219.56	12.2**	Q100 SOME ABOVE AHW.; 1.9' BELOW F.G.; TRY 42" R.C.P.
" " "	42"	78	-	1.27	4.44'	"	217.49	2.27'	2.75'	3.31'	3.31'	"	1.74'	"	3.87'	216.91	217.49	12.2	WELL BELOW AHW.; 36" R.C.P. ALLOWABLE AT LESS COST.
																			WITH HW_i CONTROL, CONSIDERED IMPROVED ENTRANCE
																			STRUCTURE; HOWEVER, ONLY SMALL ADVANTAGE GAINED,
																			THEREFORE, NO ENTRANCE IMPROVEMENTS SET UP.
																			Q50 OUTLET VEL. >10 f.p.s.,
																			TREATMENT REQ'D. SEE
																			NOTE BELOW.



OVERTOPPING:
221.55 LOW PT. AH.
+0.08
221.63 ELEV.
213.05 U.S. FL.
8.58'; ÷ 3.0' = 2.86 (HW/D)
Q ≈ 102 C.F.S.

** Q100 IS TO RIGHT OF 0.01% SLOPE CURVE
∴ PIPE IS FLOWING FULL - SEE (HDS NO. 3)

OUTFALL CHANNEL:
RIPRAP REQUIRED
6' F.B., 2:1 SIDES, 2' FLOW DEPTH
WIDTH = 15' TOTAL
LENGTH = 12'
TONS = 13.5

ENCROACHMENT RISK ASSESSMENT (BASE STORM):

OVERTOPPING STORM X - YR.	DISCHARGE	HW ELEV.	1. HAZARD TO PEOPLE & PROPERTY: TRAFFIC NO / YES	ROADWAY NO / YES	BACKWATER NO / YES	2. OTHER FACTORS, USE & LOCATION: NEED FOR EMERGENCY USE NO / YES	LACKS DETOUR NO / YES	ON REGULATORY FLOODWAY NO / YES	CHECK ✓: ✓ RISK ACCEPTABLE - TRADITIONAL DESIGN USED	RISK SIGNIFICANT - ANALYSIS REQUIRED
500	102 C.F.S.	221.63							✓	

TECHNICAL FOOTNOTES

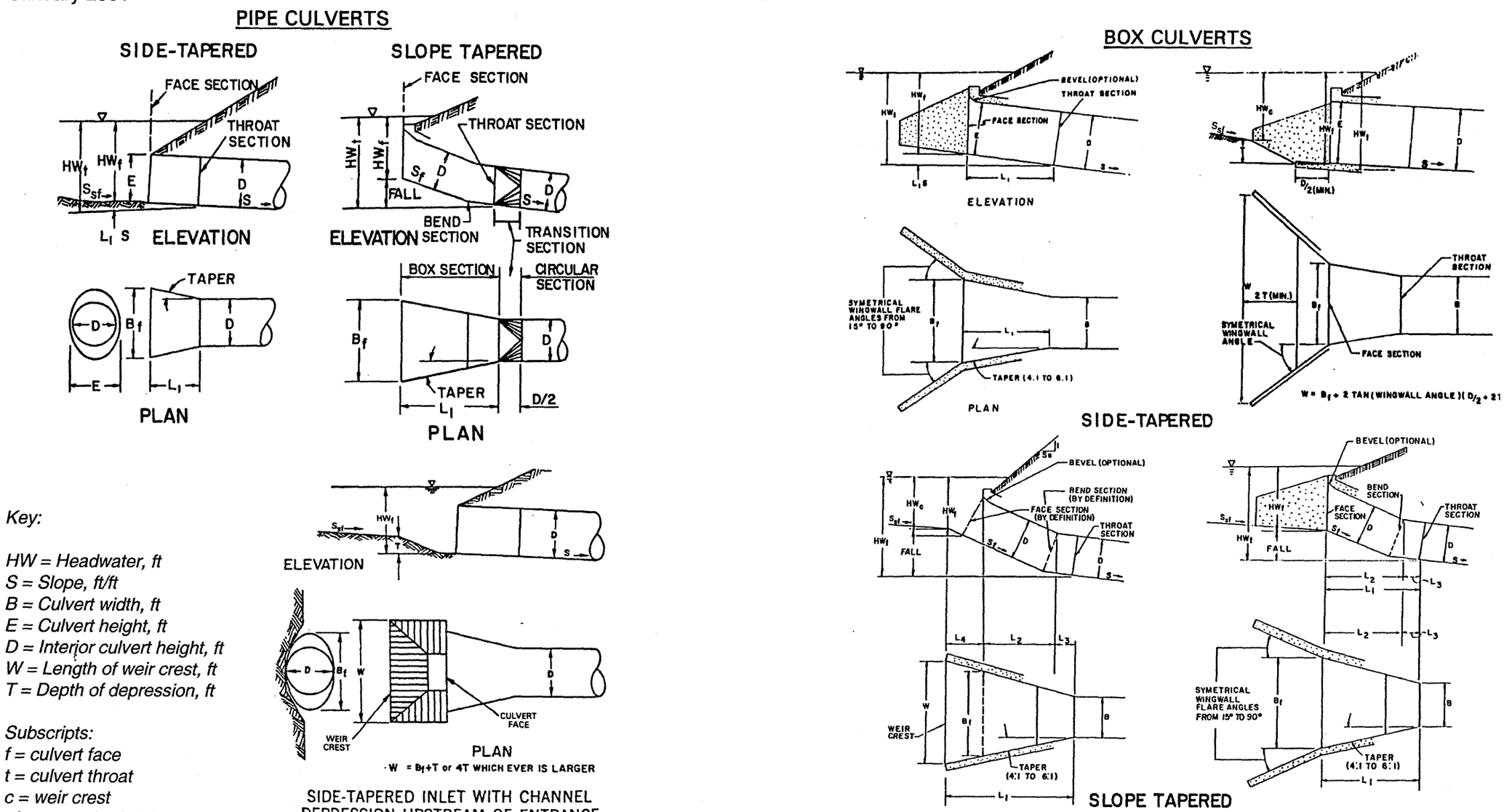
(1) USE Q/NB FOR BOX CULVERTS, (CFS).
(2) HW_i/D = HW/D OR HW_i/D - FROM DESIGN CHARTS.
(3) FALL = HW_i - (EL_{hd} - EL_{sf}); FALL IS ZERO FOR CULVERTS ON NATURAL GRADE
(4) EL_{hi} = HW_i + EL_i (INVERT OF INLET CONTROL SECTION).
(5) TW BASED ON DOWNSTREAM CONTROL OR FLOW DEPTH IN CHANNEL.
(6) h_o = TW OR (d_c + D)/2 (WHICHEVER IS GREATER).
(7) $H = \left[1 + k_e + 29n^2 L/R^{1.33}\right] V^2/2g$.
(8) $HW_{ho} = H + h_o - L_oS$; $EL_{ho} = EL_o + H + h_o$.
(9) V_{hi} = Q/A @ d_h (HDS NO. 3); V_{ho} = Q/A @ h_o.

SUBSCRIPTS
a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

CULVERT BARREL SELECTED
SIZE 36"
SHAPE ROUND
ENTRANCE GROOVE OR F.E.S.
MATERIAL R.C.P. n 0.012

MISSISSIPPI DEPARTMENT OF TRANSPORTATION 3/90 VRB BY CH

CULVERT DESIGN FORM
(Pipe Culvert Example)
Figure 7-6A

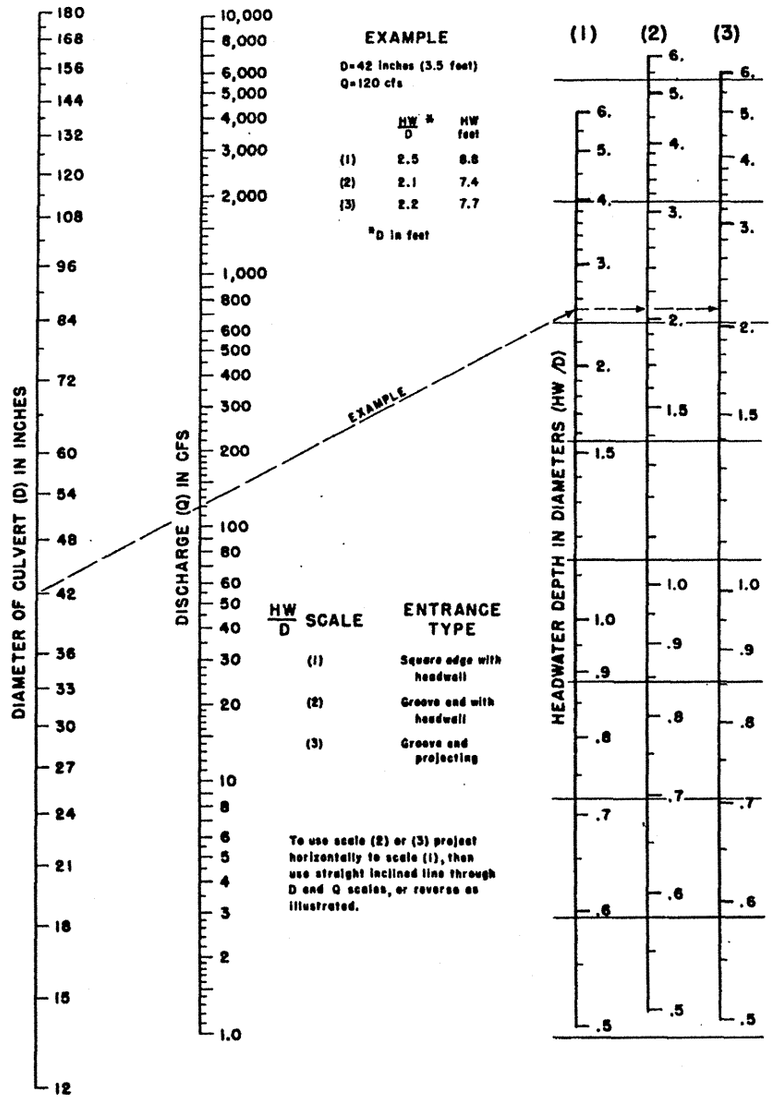


Key:

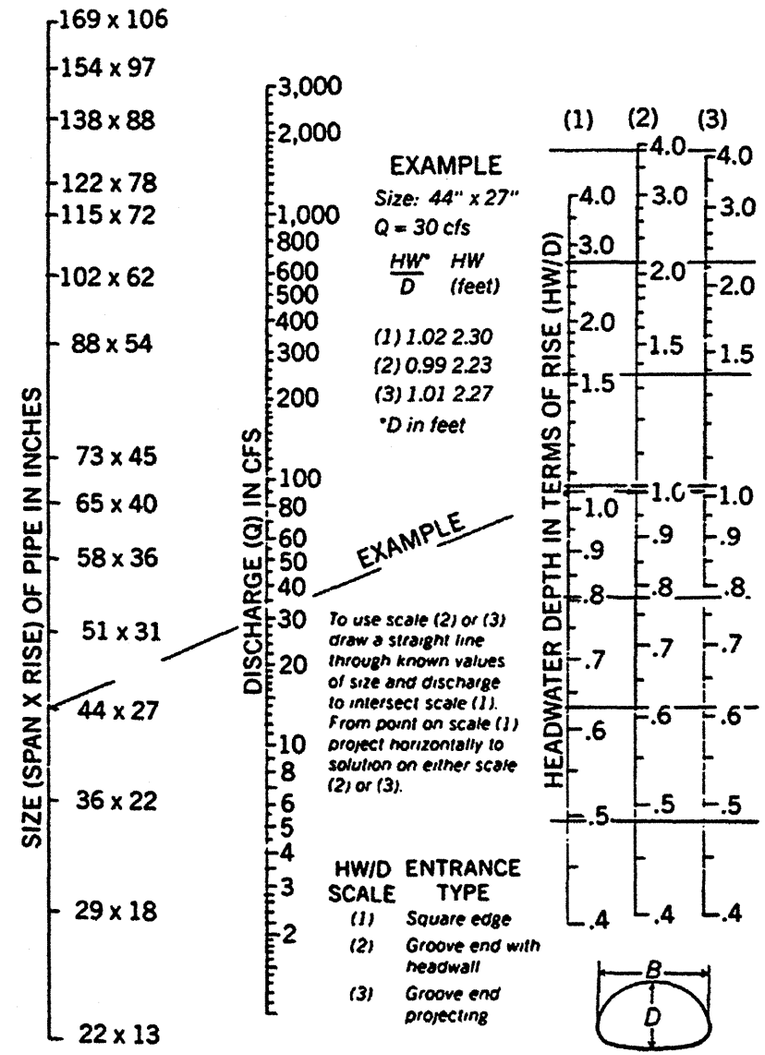
HW = Headwater, ft
S = Slope, ft/ft
B = Culvert width, ft
E = Culvert height, ft
D = Interior culvert height, ft
W = Length of weir crest, ft
T = Depth of depression, ft

Subscripts:
f = culvert face
t = culvert throat
c = weir crest
sf = streambed at face

TYPES OF IMPROVED INLETS FOR PIPE AND BOX CULVERTS
Figure 7-6B



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
Figure 7-6C



HEADWATER DEPTH FOR CONCRETE ARCH CULVERTS WITH INLET CONTROL
Figure 7-6D

(*Example 7-6.1 Continued*)

Note that a HW_i/D value greater than 2.0 would exceed the given maximum 6-ft HW depth limit. Only if the outlet control headwater (HW_{ho}) depth is less and an improved culvert entrance were installed could a HW_i greater than 6 ft be acceptable. Adding the HW_i value (5.76 ft) to the entrance flowline elevation of the pipe gives the headwater elevation at the entrance. This headwater elevation appears allowable and the pipe acceptable. However, to verify the acceptance of this particular pipe, outlet control conditions must be investigated (see Section 7-6.03.02).

7-6.03.02 Outlet Control — Concrete Pipe Culverts

The following steps should be used for outlet control analysis of pipe culverts as selected in Section 7-6.03.01. Data should be recorded on the design form illustrated in Figure 7-6A.

1. Refer to Table 7-6A for entrance loss coefficients. Find the applicable k_e value (for concrete pipe — headwall and groove end, $k_e = 0.2$) and enter the value on the design form.

2. Refer to Figure 7-6E or F, Critical Depth — Circular Pipe or Arch Pipe, to determine the Critical Depth (d_c). Enter the horizontal Discharge (Q) scale with the design discharge and vertically intersect the pipe Diameter/Size curve. At this point, project horizontally to the left to read the d_c value (d_c is never greater than the diameter of pipe). Record the d_c value.

3. Calculate $(d_c + D)/2$, where D is diameter of pipe. Record this value on the design form.

4. Tailwater (TW) is the water depth at the downstream end of the culvert. Backwater downstream from the culvert may cause a detrimental TW. Refer to FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow*, Chapter 3, Rectangular, Trapezoidal and Triangular Channels

Table 7-6A
ENTRANCE LOSS COEFFICIENTS
(Outlet Control — Full or Partly Full Entrance Head Loss)
$H_e = k_e (V^2/2g)$

Type of Structure and Design of Entrance	Coefficient k_e (1)
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-section conforming to fill slope (2)	0.5
Beveled edges, 33.7° or 45° bevel	0.2
Side- or slope-tapered inlet	0.2
Pipe or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-section conforming to fill slope (2)	0.5
Beveled edges, 33.7° or 45° bevel	0.2
Side- or slope-tapered inlet	0.2
Box-Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius or 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edge at crown	0.7
Side- or slope-tapered inlet	0.2

Notes: 1. *Coefficient k_e to apply to velocity head $V^2/2g$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operating full or partly full with control at the outlet.*

2. *"End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design, have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet. Refer to Hydraulic Design Series No. 5 for additional information.*

(Trapezoidal Charts 15-28). With the existing channel bottom width, select the appropriate chart. With the design discharge, Manning roughness coefficient (n) from Table 7-6B (Qn value, if different from the chart) and the existing channel slope, determine the natural depth of flow, which can be the TW on the culvert. However, high water from other downstream sources may cause a higher TW value. For the (h_o) value, use either the highest TW depth or the (d_c + D)/2 value, whichever is greater. Record the TW and h_o values.

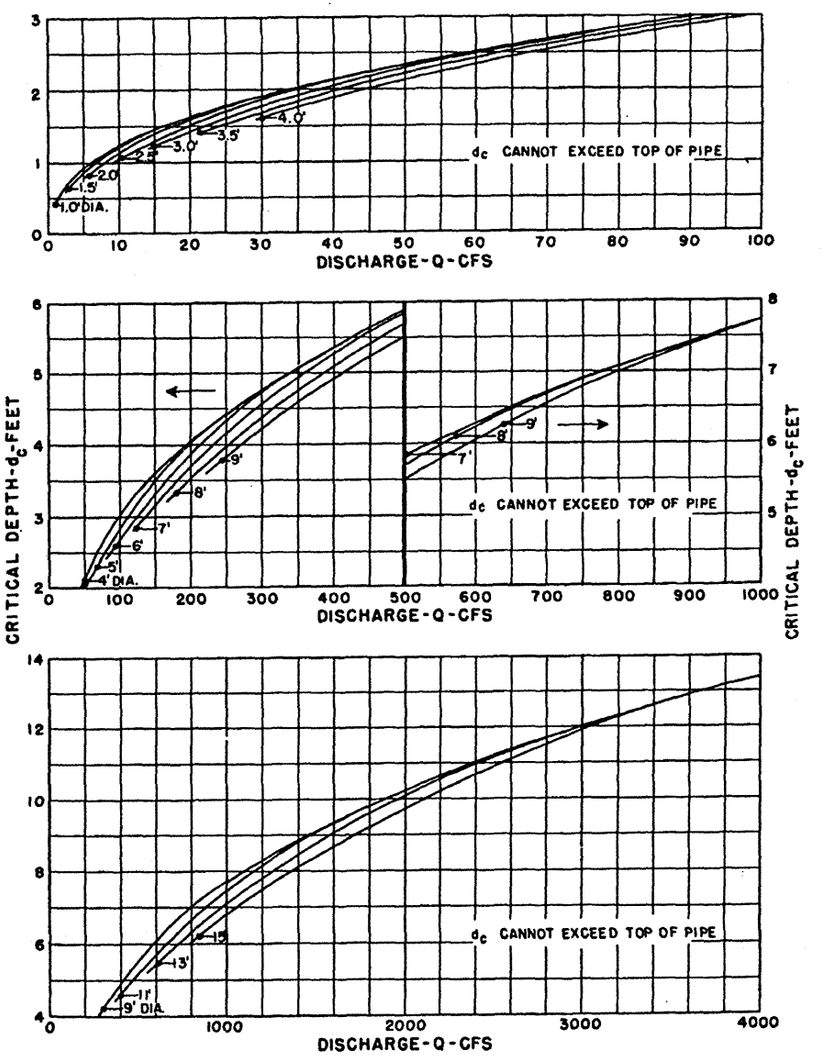
5. Refer to Figure 7-6G or H, Head for Concrete Pipe or Arch Culverts Flowing Full; n = 0.012. Align the diameter of pipe and the length of pipe on the appropriate k_e scale to establish a turning point on the turning line. Through this turning point, align the discharge on the (Q) scale and intersect the Head (H) scale. This (H) value is the necessary differential head to pass the design discharge. Enter the (H) value on the design form.

6. Refer to FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow*, Chapters 5 or 6, Circular-Pipe or Arch-Pipe Channels. Consider the desired or existing channel slope. On the pipe size chart, select the discharge scale of the appropriate (n) value (n = 0.012 for concrete pipe). Vertically intersect the selected Slope curve and project horizontally to the appropriate velocity scale. Determine the acceptability of this slope and velocity. Adjustments to the pipe slope may be necessary. Record the Slope (S) selection and retain the velocity value for later comparison.

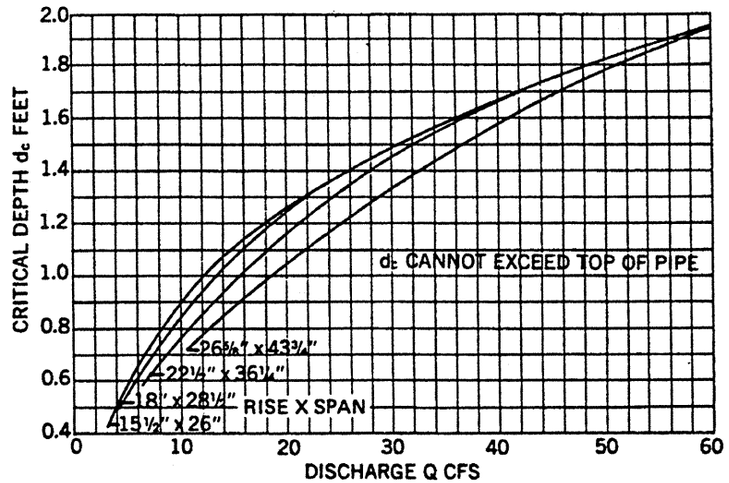
7. Enter the product of the pipe length and pipe slope (L_aS) on the design form.

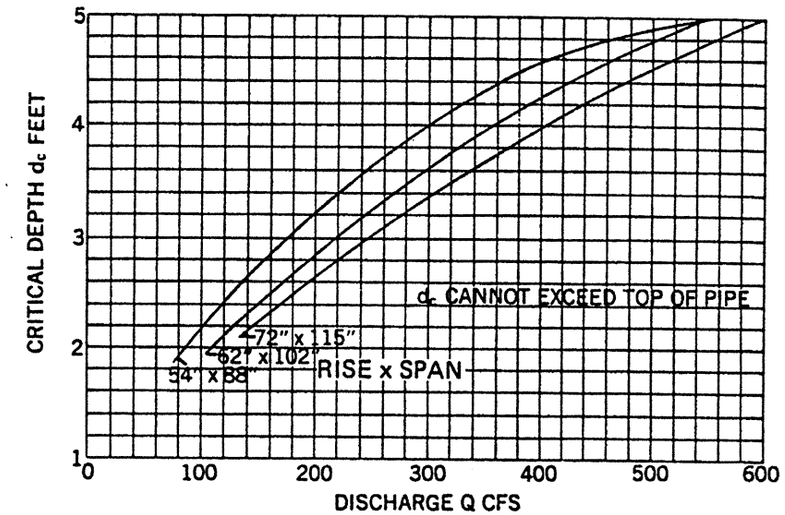
8. For the HW_{ho} depth, evaluate and record $HW_{ho} = H + h_o - L_aS$. The greater HW value, HW_i or HW_{ho}, indicates either inlet or outlet control. If HW_{ho} is allowable and less than HW_i, investigate an improved entrance (Hydraulic Design Series No. 5). Otherwise, try a larger or lower profile culvert.

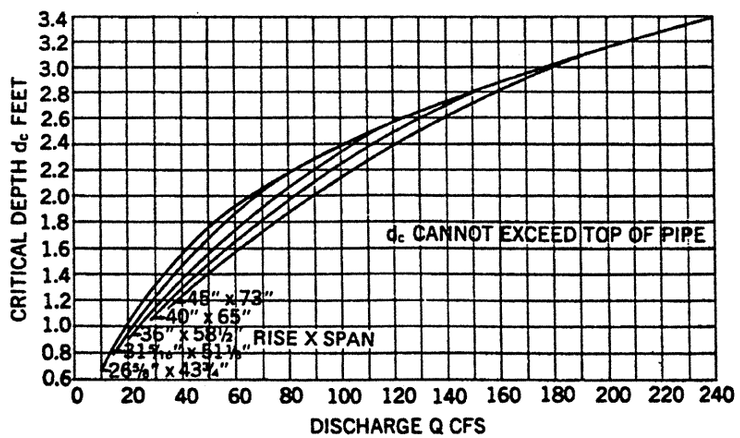
9. Velocity at the outfall end of the culvert is relative to the greater or controlling HW depth. When in inlet control, the area of water prism at normal depth (d_n) is used in the equation V = Q/A, or use the charts of Hydraulic Design Series

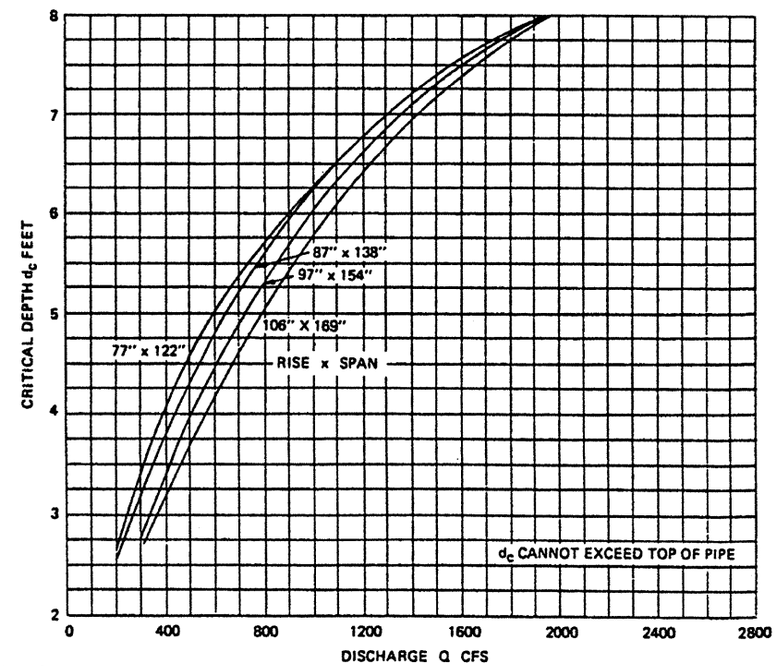


CRITICAL DEPTH
(Circular Pipe)
Figure 7-6E









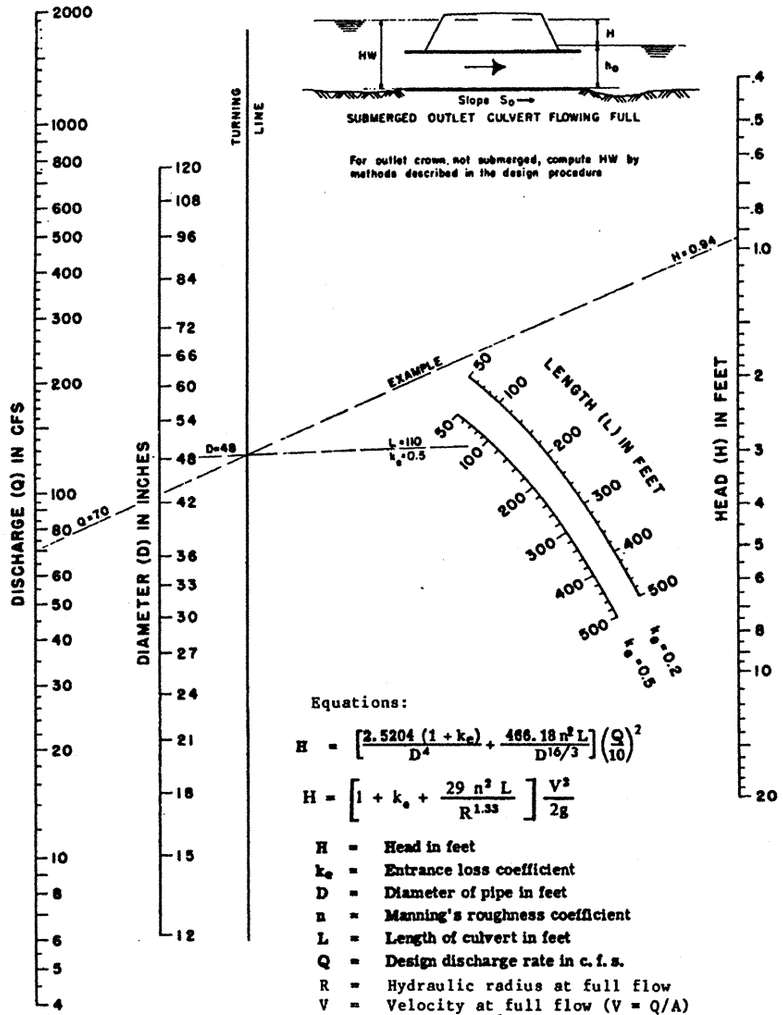
CRITICAL DEPTH — (Arch Pipe)
Figure 7-6F

No. 3, as referenced in #6. When in outlet control, the area of water prism in the equation V = Q/A is either full depth (D) of culvert where tailwater (TW) is greater than D, TW depth where TW is less than D but greater than critical depth (d_c), or d_c where d_c is greater than TW. Outfall channel erosion control treatments should be considered where Q_{50} - velocities exceed 8 ft/s (6 ft/s for Q_{10}) and are essential for soils of less than average resistance to erosion even at lesser velocities. See Table 7-10A "Recommended Permissible Velocities for Unlined Channels."

10. In support of the cross drain culvert size selection (FHWA), determine the roadway overtopping storm. Often with the use of smaller pipe and box culverts, an overtopping storm readily may be greater than the Base Storm (100-year) and estimates for these greater Recurrence Intervals (Frequencies) must be expanded. For the selected culvert, derive the estimated discharge and Recurrence Interval (Frequency) of the storm that will first overtop the roadway. Normally, overtopping is associated with culverts located in the vicinity of a sag vertical curve; otherwise, side drain spill-overs may occur and accumulate in a sag condition. Add 0.08 ft to the low-point elevation in the roadway section where overtopping will begin and subtract the culvert entrance flowline elevation giving the headwater depth, in feet. From the nomographs for the selected culvert, determine and notate the discharge (Q) necessary to generate this headwater depth. With the discharge method used, derive the 2-, 5-, 10-, 25-, 50-, 100- (or greater) and 500-year (if available) storm discharges. On "PROBABILITY X 2 LOG CYCLES" grid paper (see Figure 7-6I), plot these discharges with respect to the respective Recurrence Intervals (Storm Frequencies). The "Probability, in Percent, of Being Exceeded Each Year" values are along the top horizontal scale — the inverse of the respective percent is the Recurrence Interval (Frequency), in years. The "Discharge, in ft^3/s," is the vertical scale. Of these plotted points, a best-fit straight-line "curve" is determined and extended to the upper limits of the grid scales. Using the overtopping discharge from above, enter the Discharge scale projecting horizontally until intersecting the best-fit "curve. Read vertically upward to the Percent scale — the inverse of this

Table 7-6B
MANNING ROUGHNESS COEFFICIENTS, *n*

	Manning's *n* Range
I. Closed conduits:	
A. Concrete pipe	0.011-0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2⅔ by ½-in. corrugation (riveted pipe):	
a. *Plain or fully coated*	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021-0.018
(2) Flow 0.8 depth	0.021-0.016
(3) Flow 0.6 depth	0.019-0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012-0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009-0.011
F. Brick	0.014-0.017
G. Monolithic concrete:	
1. *Wood forms, rough*	0.015-0.17
2. Wood forms, smooth	0.012-0.014
3. Steel forms	0.012-0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017-0.022
2. *Natural floor*	0.019-0.025
I. Laminated treated wood	0.015-0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined (straight alignment):	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.015
4. Flat finish, some gravel on bottom	0.015-0.017
5. Gunite, good section	0.016-0.019
6. Gunite, wavy section	0.018-0.022
B. *Concrete, bottom float finished, sides as indicated:*	
1. Dressed stone in mortar	0.015-0.017
2. Random stone in mortar	0.017-0.020
3. Cement rubble masonry	0.020-0.025
4. Cement rubble masonry, plastered	0.016-0.020
5. Dry rubble (riprap)	0.020-0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017-0.020
2. Random stone in mortar	0.020-0.023
3. Dry rubble (riprap)	0.023-0.033
D. Brick	0.014-0.017
E. Asphalt:	
1. *Smooth*	0.013
2. Rough	0.016
F. Wood, planed; clean	0.011-0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017-0.020
2. *Irregular section*	0.022-0.027
III. Open channels, excavated (straight alignment, natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016-0.018
2. Clean, after weathering	0.018-0.020
3. With short grass, few weeds	0.022-0.027
4. In gravelly soil, uniform section, clean	0.022-0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022-0.025
2. Grass, some weeds	0.025-0.030
3. Dense weeds or aquatic plants in deep channels	0.030-0.035
4. Sides clean, gravel bottom	0.025-0.030
5. Sides clean, cobble bottom	0.030-0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028-0.033
2. Light brush on banks	0.035-0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035-0.040
b. Jagged and irregular	0.040-0.045
E. Channels not maintained, weeds and brush uncut:	
1. *Dense weeds, high as flow depth*	0.08-0.12
2. Clean bottom, brush on sides	0.05-0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07-0.11
4. Dense brush, high stage	0.10-0.14
IV. Highway channels and swales with maintained vegetation (values shown are for velocities of 2 and 6 fps):	
A. Depth of flow up to 0.7 foot:	
1. Bermuda grass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07-0.045
b. Length 4-6 inches	0.09-0.05
2. Good stand, any grass:	
a. *Length about 12 inches*	0.18-0.09
b. Length about 24 inches	0.30-0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14-0.08
b. Length about 24 inches	0.25-0.13
B. Depth of flow 0.7-1.5 feet:	
1. Bermuda grass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05-0.035
b. Length 4 to 6 inches	0.06-0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12-0.07
b. Length about 24 inches	0.20-0.10
3. *Fair stand, any grass:*	
a. Length about 12 inches	0.10-0.06
b. Length about 24 inches	0.17-0.09
V. Street and expressway gutters:	
A. *Concrete gutter, troweled finish*	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. *Smooth*	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of *n* by	0.002
VI. Natural stream channels:	
A. Minor streams (surface width at flood stage less than 100 ft.):	
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	0.035-0.05
c. Some weeds, light brush on banks	0.035-0.05
d. Some weeds, heavy brush on banks	0.05-0.07
e. Some weeds, dense willows on banks	0.06-0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01-0.02
2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e about	0.01-0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.05
2. Cultivated areas:	
a. No crop	0.03-0.04
b. Mature row crops	0.035-0.045
c. Mature field crops	0.04-0.05
3. *Heavy weeds, scattered brush*	0.05-0.07
4. Light brush and trees:	
a. Winter	0.05-0.06
b. Summer	0.06-0.08
5. Medium to dense brush:	
a. Winter	0.07-0.11
b. Summer	0.10-0.16
6. Dense willows, summer, not bent over by current	0.15-0.20
7. Cleared land with tree stumps, 100-150 per acre:	
a. No sprouts	0.04-0.05
b. *With heavy growth of sprouts*	0.06-0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10-0.12
b. Flood depth reaches branches	0.12-0.16
C. *Major streams (surface width at flood stage more than 100 ft.):* Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of *n* may be somewhat reduced. Follow recommendation in publication cited if possible. The value of *n* for *larger streams of most regular section*, with no boulders or brush, may be in the range of	0.028-0.033



Equations:

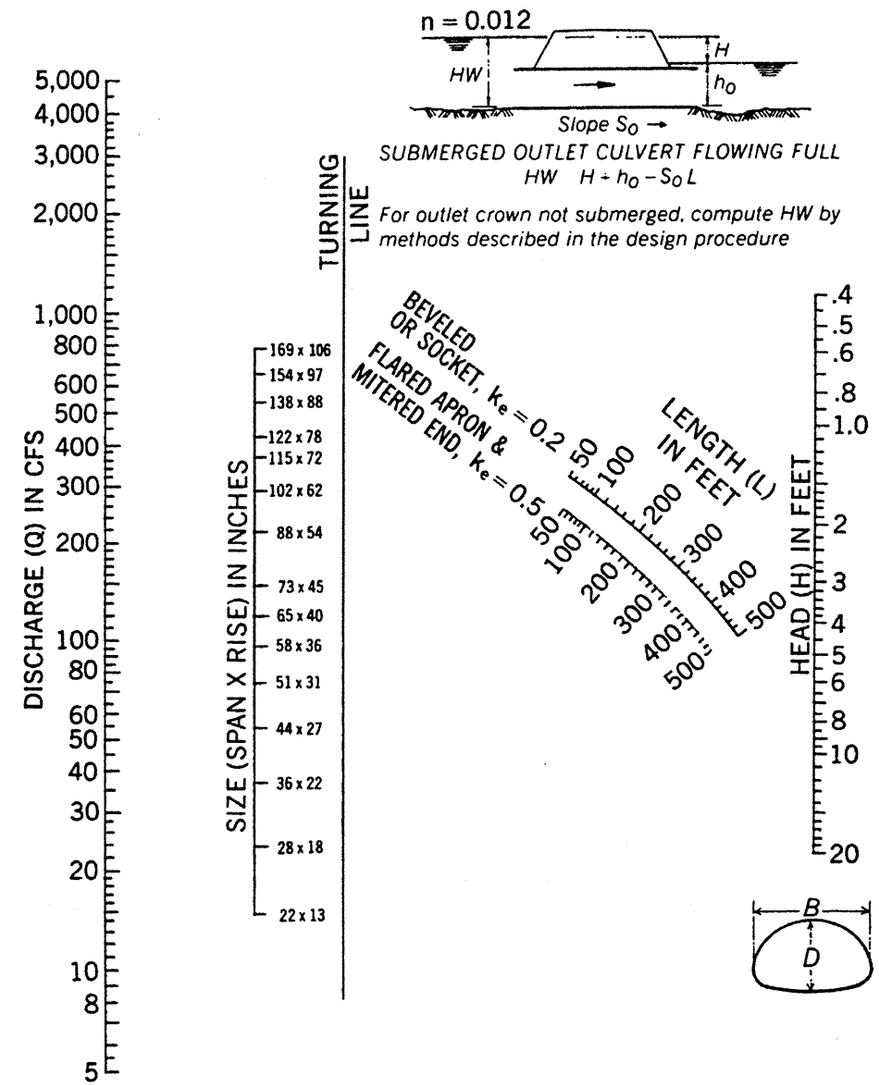
$$H = \left[\frac{2.5204\,(1+k_e)}{D^4} + \frac{466.18\,n^2 L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$$

$$H = \left[1 + k_e + \frac{29\,n^2 L}{R^{1.33}}\right]\frac{V^2}{2g}$$

H = Head in feet
k_e = Entrance loss coefficient
D = Diameter of pipe in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Design discharge rate in c. f. s.
R = Hydraulic radius at full flow
V = Velocity at full flow (V = Q/A)
g = 32.2 ft/sec^2

HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL (n = 0.012)
Figure 7-6G

BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR CONCRETE ARCH CULVERTS FLOWING FULL
Figure 7-6H

Percent value is the Recurrence Interval (Frequency). Record the Recurrence Interval (Storm Frequency), x-year, and the discharge (Q) on the design form. Storm frequencies up to the 10,000-year can be estimated from the probability grid figure; however, when the overtopping storm is greater than the 500-year storm, a statement to this effect is sufficient and no further investigation is necessary with these derived values appropriately recorded.

11. Having determined the overtopping storm, perform a Risk Assessment (FHWA — see HEC No. 17), which may indicate that a more in-depth Risk Analysis is required. More typically associated to bridge crossings, the concepts are applicable to other flood plain encroachments (smaller cross drain structures). Risks to the highway facility and surrounding property must be considered. Supplying the data for the Encroachment Risk Assessment block on the Culvert Design Form serves as a guide for the assessment where the appropriate blank is checked. Generally, for pipe and box culverts within the drainage area limits of this responsibility (see Section 7-1.02), the Risk Assessment will be sufficient; however, there may be instances where a Risk Analysis is necessary (see HEC No. 17).

Example 7-6.2 illustrates the use of the above procedure to determine the outlet control headwater depth for a pipe culvert with a given design discharge.

Example 7-6.2

Given: Same data from Example 7-6.1.

Problem: Determine the proper size concrete pipe culvert for the given location (outlet control).

Solution: The designer should follow the procedure for outlet control presented in Section 7-6.03.02. This example is shown on the Culvert Design Form, Figure 7-6A.

1. Table 7-6A provides the entrance loss coefficient (k_e) of 0.2. Enter the value on the design form.

2. Figure 7-6E provides the Critical Depth, d_c, of 2.85 ft. Record the d_c value.

3. $(d_c + D)/2 = (2.85 \text{ ft} + 3.0 \text{ ft})/2 = 2.93$ ft. Enter this value on the design form.

4. This is a very shallow natural channel. Therefore, the tailwater (TW) will be less than d_c, and the h_o value will be $(d_c + D)/2 = 2.93$ ft. Record the h_o value.

5. Refer to Figure 7-6G. Assume a pipe length of 100 ft. Align the pipe diameter (36 in) and the pipe length (100 ft) on the $k_e = 0.2$ scale to establish the turning point. At that point, align the design discharge and intersect the Head (H) scale. Read 3.44 ft, and enter the value on the form.

6. Refer to FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow*, Chapter 5, Circular Pipe Channels. The proposed pipe slope is 0.01 ft/ft. For a 36-in pipe, enter the discharge scale of n = 0.012 at 78 ft^3/s. Vertically intersect the 0.01 Slope curve (note from normal depth curves, d_n = 2.95 ft); project horizontally to the n = 0.012 Velocity scale; and read a velocity of 11.1 ft/s at normal depth. Record the Slope (S) on the form and retain the velocity for later comparison.

7. Enter the product of the pipe length and pipe slope (L_aS), $L_aS = (100 \text{ ft})(0.01) = 1.00$ ft, on the design form.

8. Evaluate the equation $HW_{ho} = H + h_o - L_aS$. $HW_{ho} = 3.44 \text{ ft} + 2.93 \text{ ft} - 1.00 \text{ ft} = 5.37$ ft. Enter this value on the design form.

9. HW_i (5.76 ft, Example 7-6.1 is greater than HW_{ho} (5.37 ft). The culvert operates in inlet control and the outfall velocity is 11.1 ft/s (derived in #6 with respect to the normal depth of flow, d_n = 2.95 ft). Similarly, evaluate the equation V = Q/A (A is the area of water prism from Table 7-6C). V = 78/7.03 = 11.09 ft/s. Had the culvert been in outlet control, one of either D, TW < D,

Table 7-6C
AREA, WETTED PERIMETER AND HYDRAULIC RADIUS OF PARTIALLY FILLED CIRCULAR PIPE

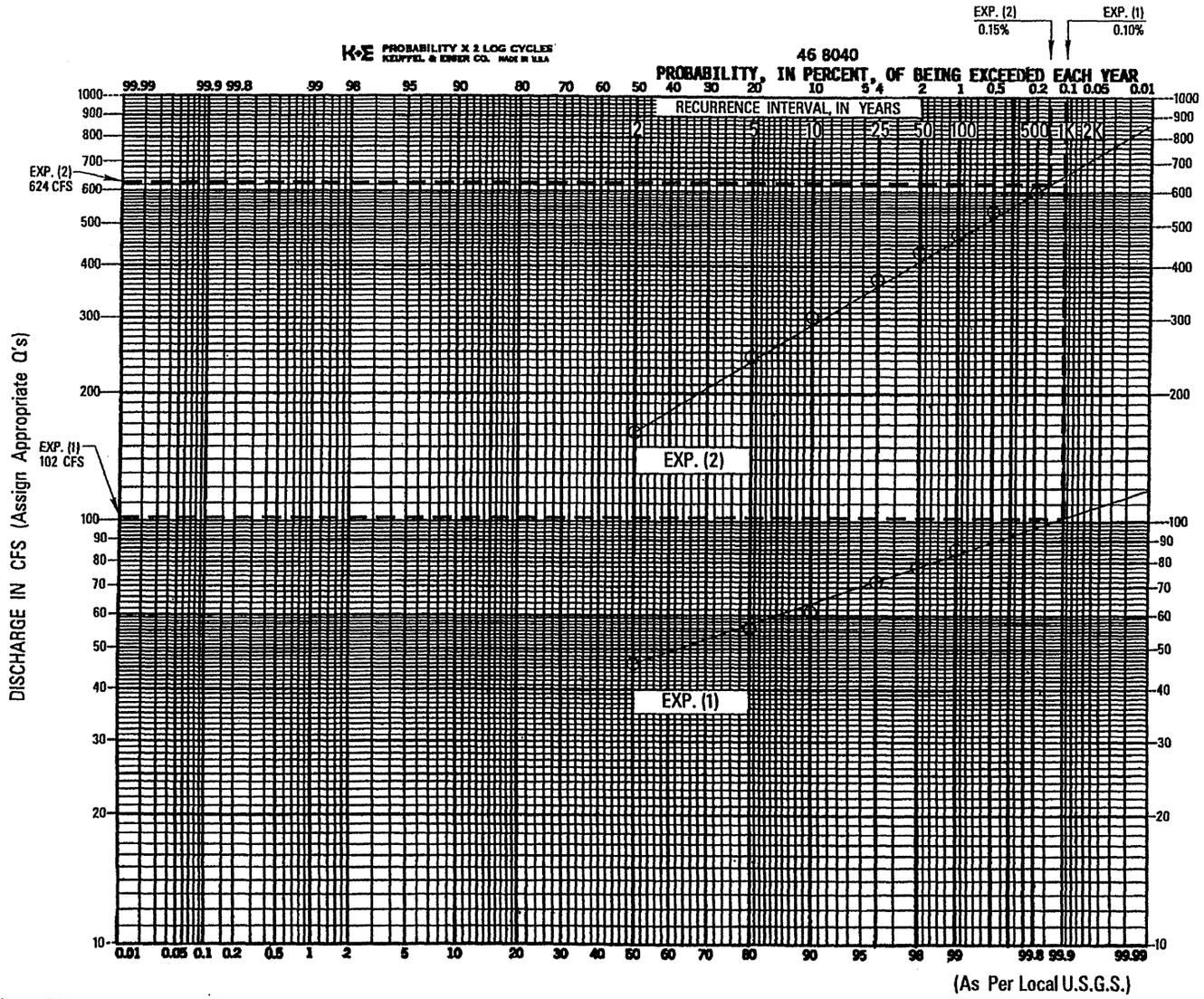
$\frac{d}{D}$	$\frac{area}{D^2}$	$\frac{wet.\ per.}{D}$	$\frac{hyd.\ rad.}{D}$	$\frac{d}{D}$	$\frac{area}{D^2}$	$\frac{wet.\ per.}{D}$	$\frac{hyd.\ rad.}{D}$
0.01	0.0013	0.2003	0.0066	0.51	0.4027	1.5908	0.2531
0.02	0.0037	0.2838	0.0132	0.52	0.4127	1.6108	0.2561
0.03	0.0069	0.3482	0.0197	0.53	0.4227	1.6308	0.2591
0.04	0.0105	0.4027	0.0262	0.54	0.4327	1.6509	0.2620
0.05	0.0147	0.4510	0.0326	0.55	0.4426	1.6710	0.2649
0.06	0.0192	0.4949	0.0389	0.56	0.4526	1.6911	0.2676
0.07	0.0242	0.5355	0.0451	0.57	0.4625	1.7113	0.2703
0.08	0.0294	0.5735	0.0513	0.58	0.4723	1.7315	0.2728
0.09	0.0350	0.6094	0.0574	0.59	0.4822	1.7518	0.2753
0.10	0.0409	0.6435	0.0635	0.60	0.4920	1.7722	0.2776
0.11	0.0470	0.6761	0.0695	0.61	0.5018	1.7926	0.2799
0.12	0.0534	0.7075	0.0754	0.62	0.5115	1.8132	0.2821
0.13	0.0600	0.7377	0.0813	0.63	0.5212	1.8338	0.2842
0.14	0.0668	0.7670	0.0871	0.64	0.5308	1.8546	0.2862
0.15	0.0739	0.7954	0.0929	0.65	0.5404	1.8755	0.2881
0.16	0.0811	0.8230	0.0986	0.66	0.5499	1.8965	0.2899
0.17	0.0885	0.8500	0.1042	0.67	0.5594	1.9177	0.2917
0.18	0.0961	0.8763	0.1097	0.68	0.5687	1.9391	0.2933
0.19	0.1039	0.9020	0.1152	0.69	0.5780	1.9606	0.2948
0.20	0.1118	0.9273	0.1206	0.70	0.5872	1.9823	0.2962
0.21	0.1199	0.9521	0.1259	0.71	0.5964	2.0042	0.2975
0.22	0.1281	0.9764	0.1312	0.72	0.6054	2.0264	0.2987
0.23	0.1365	1.0003	0.1364	0.73	0.6143	2.0488	0.2998
0.24	0.1449	1.0239	0.1416	0.74	0.6231	2.0714	0.3008
0.25	0.1535	1.0472	0.1466	0.75	0.6318	2.0944	0.3017
0.26	0.1623	1.0701	0.1516	0.76	0.6404	2.1176	0.3025
0.27	0.1711	1.0928	0.1566	0.77	0.6489	2.1412	0.3032
0.28	0.1800	1.1152	0.1614	0.78	0.6573	2.1652	0.3037
0.29	0.1890	1.1373	0.1662	0.79	0.6655	2.1895	0.3040
0.30	0.1982	1.1593	0.1709	0.80	0.6736	2.2143	0.3042
0.31	0.2074	1.1810	0.1755	0.81	0.6815	2.2395	0.3044
0.32	0.2167	1.2025	0.1801	0.82	0.6893	2.2653	0.3043
0.33	0.2260	1.2239	0.1848	0.83	0.6969	2.2916	0.3041
0.34	0.2355	1.2451	0.1891	0.84	0.7043	2.3186	0.3038
0.35	0.2450	1.2661	0.1935	0.85	0.7115	2.3462	0.3033
0.36	0.2546	1.2870	0.1978	0.86	0.7186	2.3746	0.3026
0.37	0.2642	1.3078	0.2020	0.87	0.7254	2.4038	0.3017
0.38	0.2739	1.3284	0.2061	0.88	0.7320	2.4341	0.3008
0.39	0.2836	1.3490	0.2102	0.89	0.7384	2.4655	0.2996
0.40	0.2934	1.3694	0.2142	0.90	0.7445	2.4981	0.2980
0.41	0.3032	1.3898	0.2181	0.91	0.7504	2.5322	0.2963
0.42	0.3130	1.4101	0.2220	0.92	0.7560	2.5681	0.2944
0.43	0.3229	1.4303	0.2257	0.93	0.7612	2.6061	0.2922
0.44	0.3328	1.4505	0.2294	0.94	0.7662	2.6467	0.2896
0.45	0.3428	1.4706	0.2331	0.95	0.7707	2.6906	0.2864
0.46	0.3527	1.4907	0.2366	0.96	0.7749	2.7389	0.2830
0.47	0.3627	1.5108	0.2400	0.97	0.7785	2.7934	0.2787
0.48	0.3727	1.5308	0.2434	0.98	0.7816	2.8578	0.2735
0.49	0.3827	1.5508	0.2467	0.99	0.7841	2.9412	0.2665
0.50	0.3927	1.5708	0.2500	1.00	0.7854	3.1416	0.2500

or d_c > TW depth should be used to derive the area for the equation V = Q/A. By Manning's equation, the velocity of Q_{50} in the natural channel is found to be 6.7 ft/s (acceptable for average soils). To transition from culvert outfall velocity into the natural channel, erosion control treatment is necessary. Riprap is suggested where an approximate length equal to 4D will suffice in this example (see FHWA Hydraulic Engineering Circular No. 15 *Design of Stable Channels with Flexible Linings*).

10. It is determined that a headwater depth of 8.58 ft begins overtopping the roadway. From Figure 7-6C, HW/D = 8.58 ft/3.0 ft = 2.86, and the discharge (Q) is 102 ft^3/s. The respective discharges, 2-, 5-, 10-, 25-, 50-, and 100-year are 41, 52, 60, 69, 78, and 86 ft^3/s. On "PROBABILITY X 2 LOG CYCLE" grid paper, see Figure 7-6I, plot these discharges to the respective recurrence intervals (storm frequencies). Determine the best-fit straight-line "curve," extending upper limit. On the Discharge scale at the overtopping discharge (102 ft^3/s), project horizontally to "curve" line. Read upward to Percent scale of approximately 0.20% — the inverse is the Recurrence Interval (Frequency) which is 1/0.002 = 500-year storm. Record the Recurrence Interval (Storm Frequency) and the overtopping discharge (Q) on the design form.

11. For the last step in this pipe culvert design, perform an Encroachment Risk Assessment. Mark the guide blocks at the lower portion of the design form. Using the Base Storm (100-year) for the analysis of category 1, Hazard to People and Property (no critical elevations existing below the 100-year headwater), the respective three responses are "NO" (mark through "YES" in each); and for category 2, Other Factors, Use & Location, the respective three responses are "NO" (mark through "YES" in each). For the Base Storm, the associated risk at this site appears to be acceptable — check the Risk Acceptable block on the design form. Had the risk not been acceptable, a Risk Analysis would be necessary (refer to HEC No. 17).

With HW_i greater than HW_{ho}, the designer should consider an improved entrance which could result in a culvert size decrease. Entrance improvements should be investigated in Hydraulic Design Series No. 5.



Note: Blank grid at end of chapter.

OVERTOPPING STORM RECURRENCE INTERVALS (Frequency)
Figure 7-6I

At culvert sites, the Recurrence Intervals (Frequency) of overtopping storm discharges (see Section 7-6.03 and 7-6.04.02, Step 10) can be estimated on "PROBABILITY X 2 LOG CYCLE" grid paper with "Probability, in Percent, of Being Exceeded Each Year" values along the top scale. The inverse of the respective percent is the Recurrence Interval (Frequency), in years. Plot the respective storm Discharge to Recurrence Interval points for 2-, 5-, 10-, 25-, 50- and 100-year (or greater) storms. Draw a best-fit straight-line "curve" for these points, extending upper limit. On the Discharge scale at overtopping discharge, project horizontally to "curve" line. Read upward to the Percent scale — the inverse is the Recurrence Interval (Frequency), e.g., 0.20% is 1/0.002 = 500-year storm.

Example (1):

Problem: *Determine the approximate Recurrence Interval (Frequency) of the overtopping storm for a given 32-acre drainage site utilizing a 36-inch RCP (example data, see Figure 7-6A).*

Solution: *For this culvert site, overtopping Q = 102 ft^3/s (see Section 7-6.03.02, Step 10). Deriving the 2-, 5-, 10-, 25-, 50-, and 100-year storm discharges, in cubic feet per second, are respectively, 46, 56, 61, 72, 78, and 86 ft^3/s. On the "PROBABILITY X 2 LOG CYCLE" grid paper, plot the Discharge to Recurrence Interval (Frequency) points. Draw the best-fit straight-line "curve" for these points. On the Discharge scale at the overtopping Q = 102 ft^3/s, project horizontally to the "curve" line. Project upward to read the Percent at approximately = 0.20%. The inverse of the Recurrence Interval (Frequency) is 1/0.002 = 500-year storm.*

Example (2):

Problem: *Determine the approximate Recurrence Interval (Frequency) of the overtopping storm for a given 251-acre drainage site utilizing a double cell (6′ x 4′) box culvert (example data, see Figure 7-6J).*

Solution: *For this culvert site, overtopping Q = 624 ft^3/s (see Section 7-6.04.02, Step 10). Deriving the 2-, 5-, 25-, 50-, 100-, 200-, and 500-year storm discharges, in cubic feet per second, are respectively, 162, 244, 304, 374, 432, 477, 539, and 598 ft^3/s. On the "PROBABILITY X 2 LOG CYCLE" grid paper, plot the Discharge to Recurrence Interval (Frequency) points. Draw the best-fit straight-line "curve" for these points. On the Discharge scale at the overtopping Q = 624 ft^3/s, project horizontally to the "curve" line. Project upward to read the Percent at approximately = 0.15%. The inverse or the Recurrence Interval (Frequency) is 1/0.0015 = 667-year storm.*

7-6.04 Size Design for Box Culverts

7-6.04.01 Inlet Control — Box Culverts

The following steps should be used for inlet control analysis of box culverts. Data should be recorded on the design form illustrated in Figure 7-6J. Refer to Figure 7-6K for all regular box culverts and Figures 7-6L through N for box culverts with beveled edges which can provide lower HW values.

1. On Figure 7-6K, select the proper HW/D scale based on the type of wingwall flare. Scale (1) is the turning line.

2. For the area of opening in a trial box culvert, divide the design discharge (ft^3/s) by 10 ft/s. The profile sheet in combination with other control factors will indicate a permissible height of box. For the box width (B), divide the area of opening by the permissible standard box height. Record the trail size on the design form.

3. To determine the Q/B value on the Ratio-of-Discharge-to-Width scale of Figure 7-6K, divide the design discharge (Q) by the trial standard box width. Record the Q/B value.

4. Align the height of the box on the (D) scale and the derived (Q/B) scale value. Intersect the HW/D scale (1) turning line and project horizontally to the appropriate scale. Record the HW/D value in the HW_i/D column. Nominal HW/D values are between 1.2 and 2.0 but can range from less than 1.0 to above 8.0 (allowable headwater permitting).

5. For the HW_i depth, multiply HW_i/D by the height of the box (D). HW_i can be above allowable only when outlet control and an improved culvert entrance are analyzed. Otherwise, try a larger or lower profile culvert. Record the HW_i depth on the design form.

Example 7-6.3 illustrates the use of the above procedure to determine the inlet headwater depth for a box culvert with a given design discharge.

Example 7-6.3

Given: Design discharge = 432 ft^3/s
7-ft allowable headwater at the culvert entrance (subgrade level).

Problem: Determine the proper size box culvert for the given location (inlet control).

Solution: The designer should follow the procedure for inlet control presented in Section 7-6.04.01. This example is shown on the Culvert Design form, Figure 7-6J.

1. The wingwall flare will be either 90° or 15°; therefore, select the (HW/D) scale (2) on Figure 7-6K (check beveled inlet edges, Figures 7-6L through N).

2. To obtain the approximate area required for the box opening, divide the design discharge (432 ft^3/s) by 10 ft/s which yields 43.2 ft^2. The profile sheet indicates only a low-profile (height) box is permissible. Assume a box height of 4 ft. 43.2/4 = 10.8 ft box width. A double cell 6 ft x 4 ft box is the first standard size at least cost which will meet the area requirement. Enter this trial size box culvert on the design form.

3. Divide the design discharge (Q) by the box total opening width (B) to obtain the (Q/B) scale value, 432/12 = 36.0.

4. Align the height of box, 4 ft, on the (D) scale with 36.0 on the (Q/B) scale. Intersect the (HW/D) scale (1), the turning line, projecting horizontally to the (HW/D) scale (2) for the appropriate HW/D value of 1.59. This appears acceptable. Record the HW_i/D value on the design form.

(*Continued*)

CULVERT DESIGN FORM — SHEET 1 OF 1

SQUAD R.W. DEZYNER — DESIGNER A.L. WATERMAN — DATE 4/13/87 — PROJECT 79-0068-01-005-10 — COUNTY CLAY

HYDROLOGICAL DATA

METHOD U.S.G.S. 1976 "F.F.M.S."

DRAINAGE AREA 359 AC, 0.5609 SQ. MI.

DRAIN LENGTH 4,421 FT, 0.8374 MI.

EL. MAX. 228.28 EL. MIN. 210.60

DRAIN SLOPE 0.004 FT/FT, 21.12 FT/MI.

TIME OF CONCENTRATION (t_c) N.A. MIN.

DESIGN INTENSITY (I_{DESIGN}) N.A IN/HR.

RATIONAL RUNOFF (C) N.A.

INTENSITY-DURATION REGION N.A.

CHANNEL SHAPE TRAPEZOIDAL; 18.0' BOTTOM; 2:1 SIDES

Q_{DESIGN} (50) 432 C.F.S.

Q_{BASE} (100) 532 C.F.S.

$Q_{(OVER-TOPPING)}$ (521) 624 C.F.S.

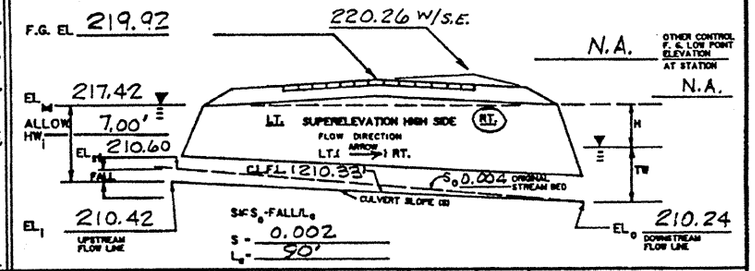
$Q_{(M.A.F.)}$ (2) 225 C.F.S.

CULVERT IN PLACE NONE - INITIAL ALIGNMENT

STATION 527+60 SKEW 30°; RT. FWD. — WORK SHEET NUMBER 7

F.G. EL 219.92 — 220.26 W/S.E.

OTHER CONTROL F.G. LOW POINT ELEVATION AT STATION: N.A. / N.A.



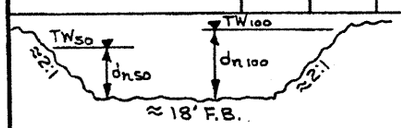
$S = S_o - FALL/L_a$

S = 0.002

L_a = 90'

HEADWATER CALCULATIONS

CULVERT DESCRIPTION: SHAPE-ENTRANCE-MATERIAL	SIZE	TOTAL FLOW Q (CFS)	FLOW PER BARREL Q/N (1)	INLET CONTROL: HW_i/D (2)	HW_i	FALL (3)	EL_M (4)	OUTLET CONTROL: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	L_aS	HW_{ho} (8)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY (ft. p. s.) (9)	COMMENTS
RECT. - SQUARE - R.C.	DBL 6'x4'	432	36.0	1.57	6.28'	0.18'	216.70	3.17'	3.40'	3.70'	3.70'	0.5	2.32'	0.18'	5.84'	216.26	216.70	**9.1	DOES APPROACHES AHW; BUT ACCEPTABLE; TRY Q_{100}.
" " "	"	532	44.3	2.00	8.00'	"	218.42	3.55'	3.90'	3.95'	3.95'	0.5	3.52'	"	7.29'	217.71	218.42	**11.1	Q_{100} SOME ABOVE AHW; 1.5' BELOW F.G.; TRY BEVEL EDGE.
" BEVELED "	"	432	36.0	1.33	5.32'	"	215.74	3.17'	3.40'	3.70'	3.70'	0.2	1.94'	"	5.46'	215.88	215.88	9.1	LESS HW W/BEVEL, INTO OUTLET CONTROL — USE.
" " "	"	532	44.3	1.60	6.40'	"	216.82	3.55'	3.90'	3.95'	3.95'	0.2	2.95'	"	6.72'	217.14	217.14	11.1	BEVELED EDGE, BUT NO OTHER ENTR. IMPROVE'MT.
																			Q_{50} OUTLET VEL < 10 f.p.s., NO ADDITIONAL TREATMENT
																			REQ'D FOR AVERAGE RESISTANCE TO EROSION SOILS.



NATURAL CHANNEL DOWNSTREAM

** HDS No. 3 INDICATES NEARLY FULL FLOW AT Q_{50}, d = 3.95' USED IN OUTLET VELOCITY. AT Q_{100} CULVERT HAS FULL FLOW, d = 4.00' USED IN OUTLET VELOCITY.

TECHNICAL FOOTNOTES

(1) USE Q/NB FOR BOX CULVERTS, (CFS).

(2) $HW_i/D = HW/D$ OR HW_f/D FROM DESIGN CHARTS.

(3) $FALL = HW_i - (EL_M - EL_{ef})$; FALL IS ZERO FOR CULVERTS ON NATURAL GRADE

(4) $EL_M = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION).

(5) TW BASED ON DOWNSTREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ OR $(d_c + D)/2$ (WHICHEVER IS GREATER).

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] v^2/2g$.

(8) $HW_{ho} = H + h_o - L_aS$; $EL_{ho} = EL_o + H + h_o$.

(9) $V_M = Q/A$ @ d_n (HDS NO. 3); $V_{ho} = Q/A$ @ h_o.

SUBSCRIPTS

a. APPROXIMATE
f. CULVERT FACE
M. DESIGN HEADWATER
M. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
ef. STREAMBED AT CULVERT FACE
tw. TAILWATER

CULVERT BARREL SELECTED

SIZE DBL. CELL 6'x4' BOX

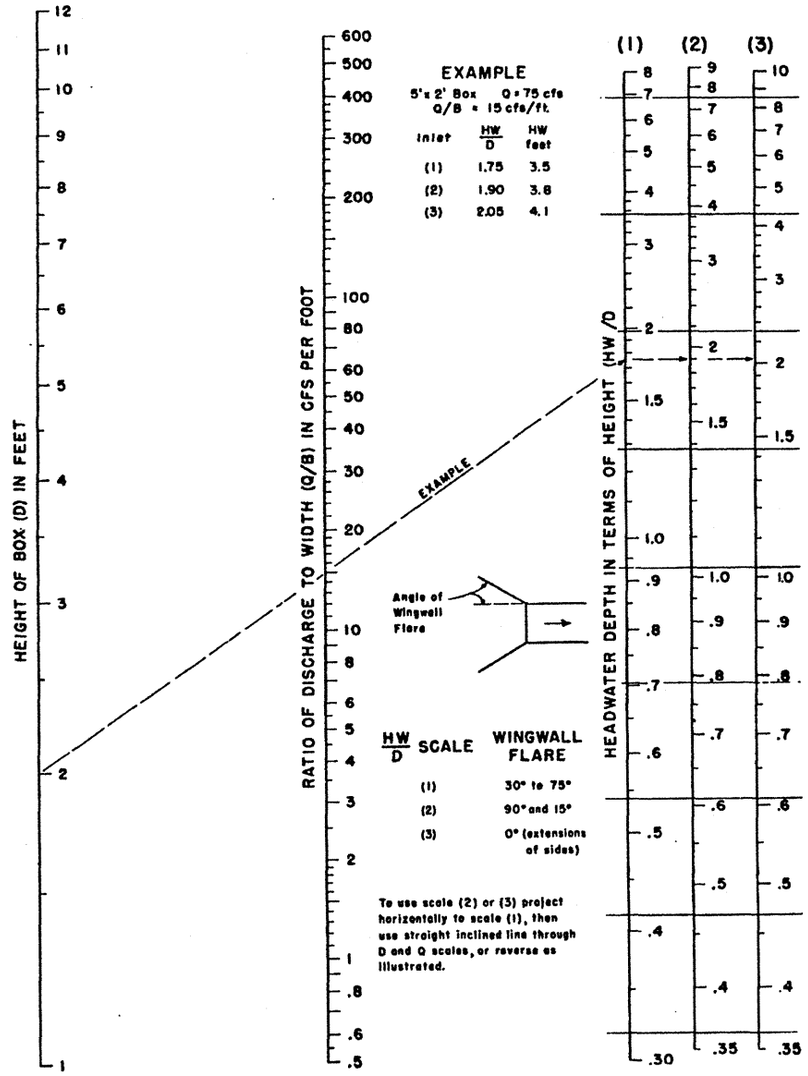
SHAPE RECTANGULAR

ENTRANCE TOP BEV. MIN. #(2)

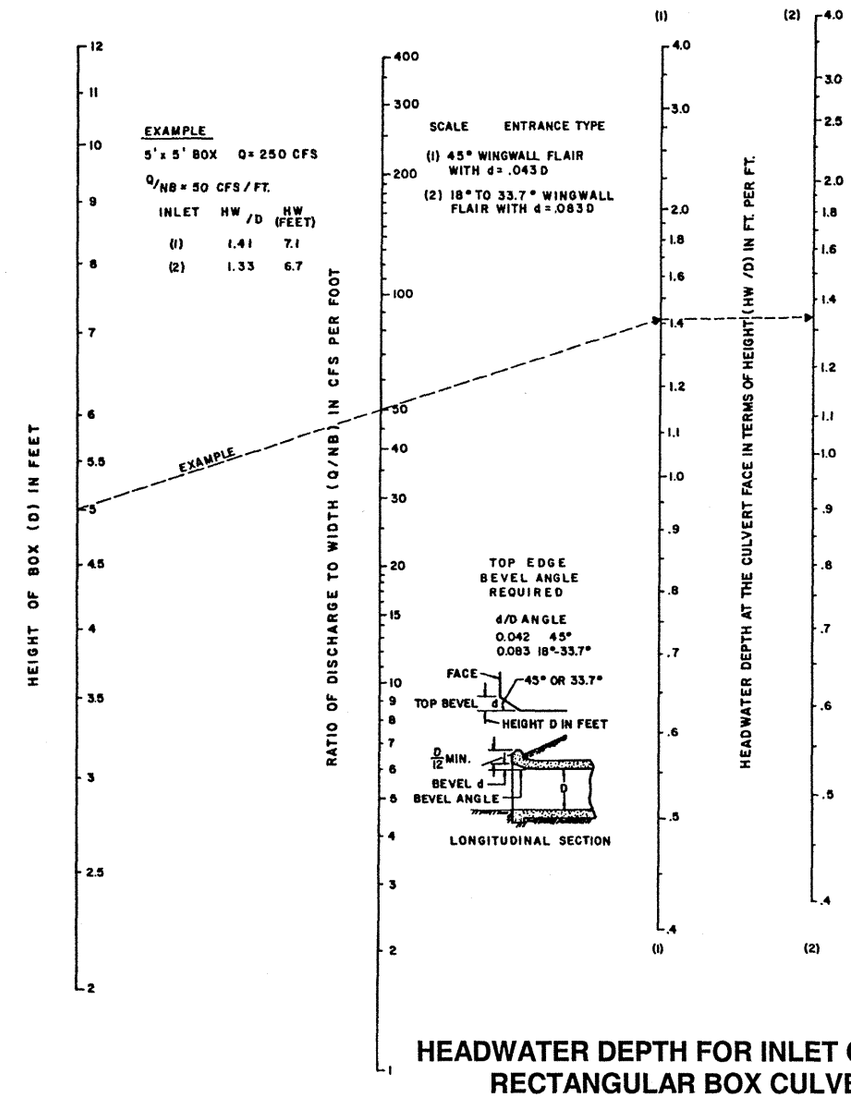
MATERIAL R.C. n 0.013

MISSISSIPPI STATE HIGHWAY DEPARTMENT 3/87 PRB BY JBM

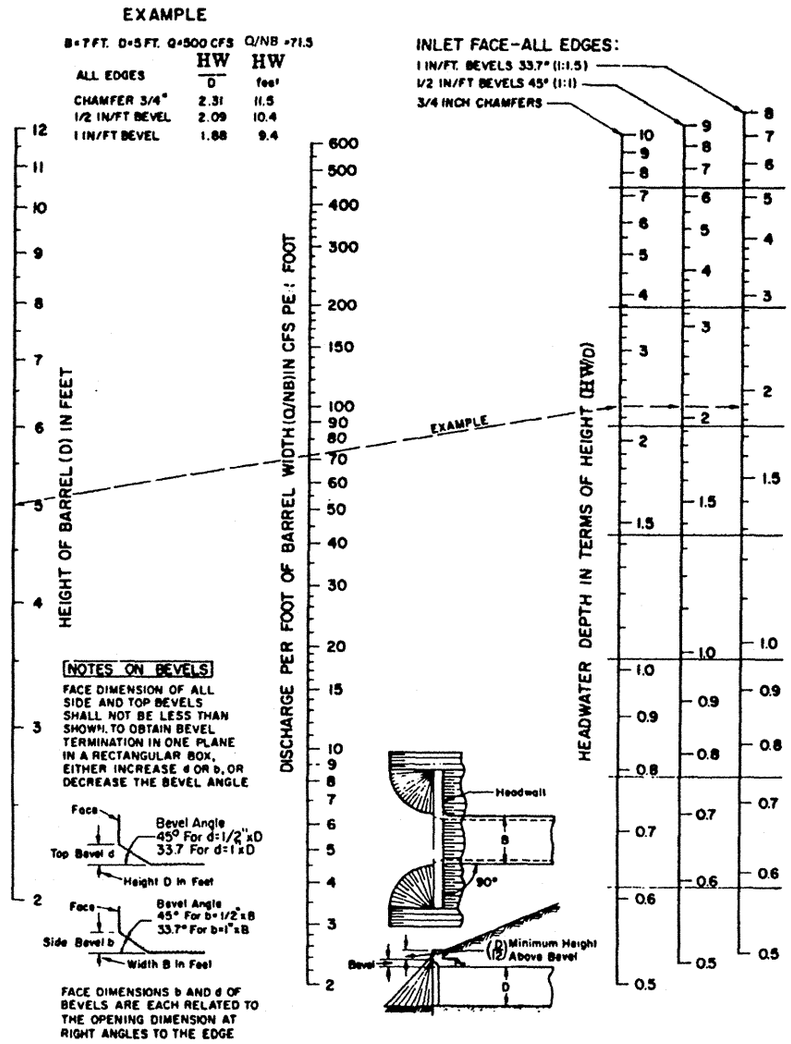
CULVERT DESIGN FORM
(Box Culvert Example)
Figure 7-6J



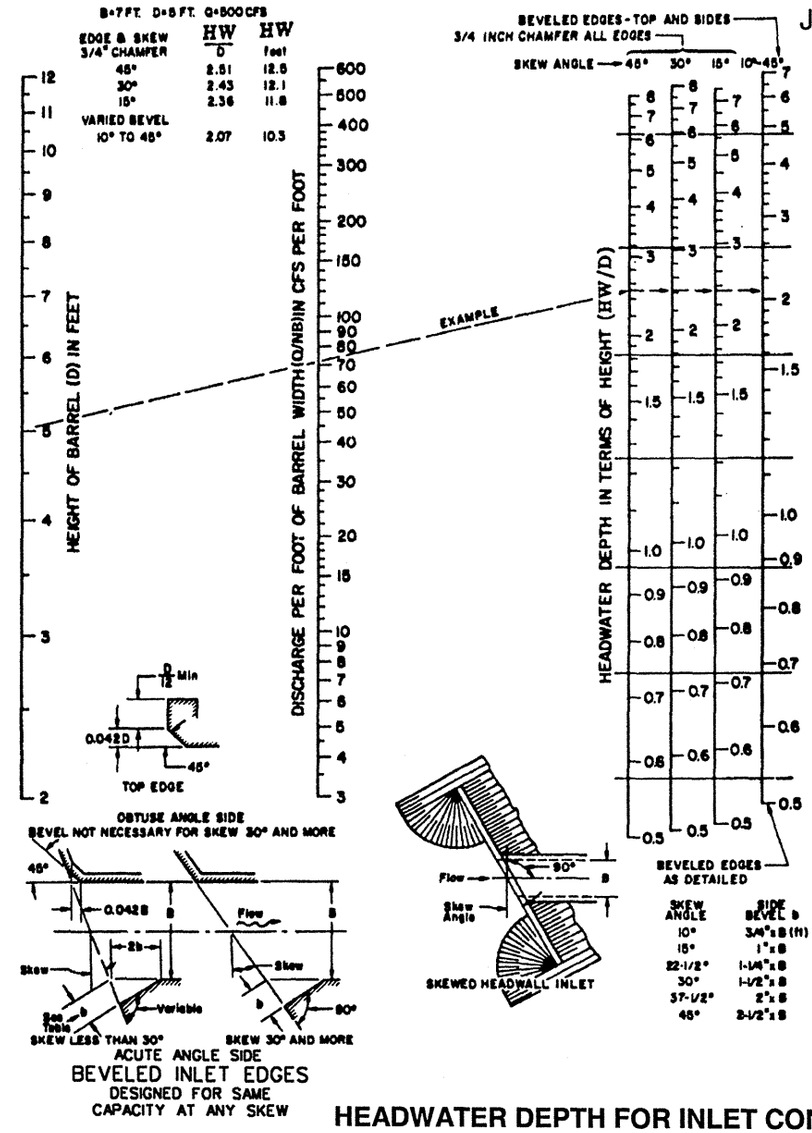
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL
Figure 7-6K



HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS FLARED WINGWALLS 18° TO 33.7° & 4.5° WITH BEVELED EDGE AT TOP OF INLET
Figure 7-6L



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES
Figure 7-6M



HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES
Figure 7-6N

(*Example 7-6.3 Continued*)

5. Multiply the HW_i/D value (1.59) by the height of the box (4 ft) to obtain HW_i, or headwater on the culvert of 6.36 ft. Enter the value on the design form.

   This headwater value added to the upstream flow line elevation of the box yields the headwater elevation at the box entrance. In this example, the HW_i of 6.36 ft on the 7-ft allowable is near critical but probably acceptable. The HW_i may be reduced with the beveled inlet edge. Proceed to outlet control analysis.

7-6.04.02 Outlet Control — Box Culverts

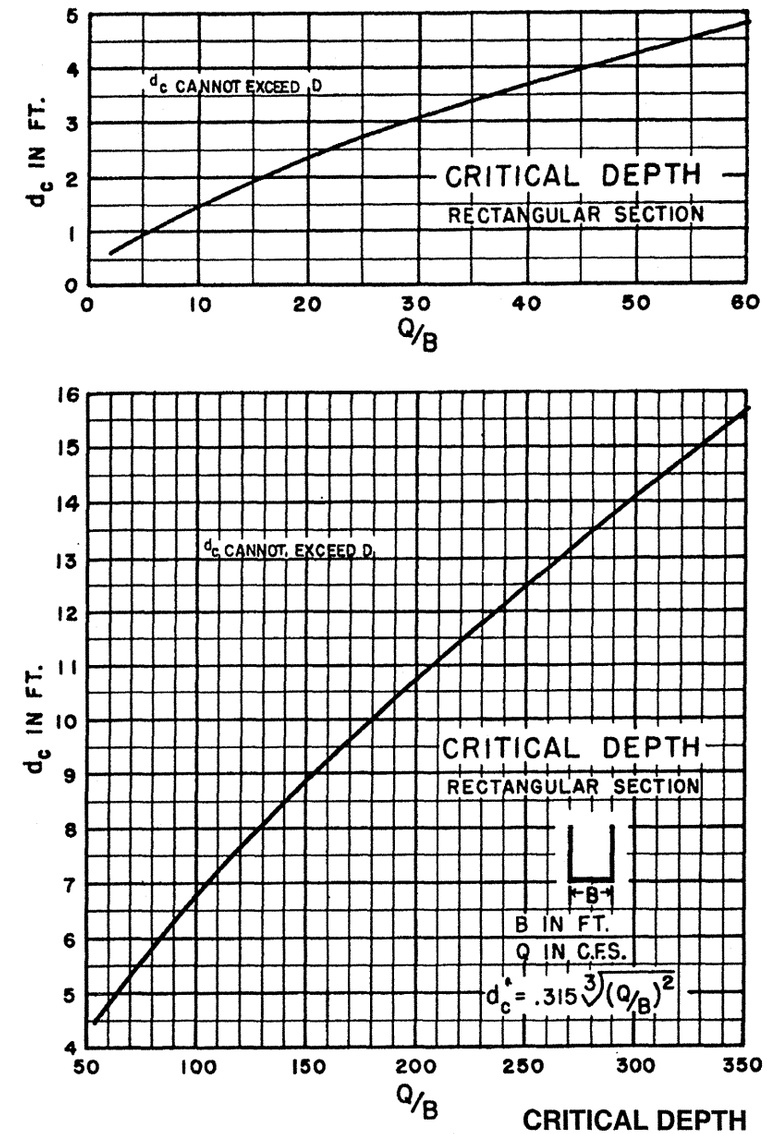
The following steps should be used for outlet control analysis of box culverts as selected in Section 7-6.04.01. Data should be recorded on the design form illustrated in Figure 7-6J.

1. Refer to Table 7-6A for the entrance loss coefficients to find the applicable k_e value for concrete box culverts. For wingwalls 10° to 25° to barrel square-edged at crown, k_e = 0.5. Enter the value on the design form. Notice that rounding or beveling of entrance edges yields less entrance loss values.

2. Refer to Figure 7-6O, Critical Depth — Box Culvert. Use the Q/B value to enter the horizontal scale and vertically intersect the heavy curve line. At this point, project horizontally to the d_c scale. Record d_c on the design form.

3. Calculate $(d_c + D)/2$ (D is height of box), and enter the value on the design form.

4. Tailwater (TW) is the water depth at the downstream end of the culvert. Backwater downstream from the culvert may cause detrimental TW. Refer to FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow*, Chapter 3, Rectangular, Trapezoidal, and Triangular Channels (Trapezoidal Charts 15-28). With the existing channel bottom width, select the appropriate



CRITICAL DEPTH
RECTANGULAR SECTION
Figure 7-6O

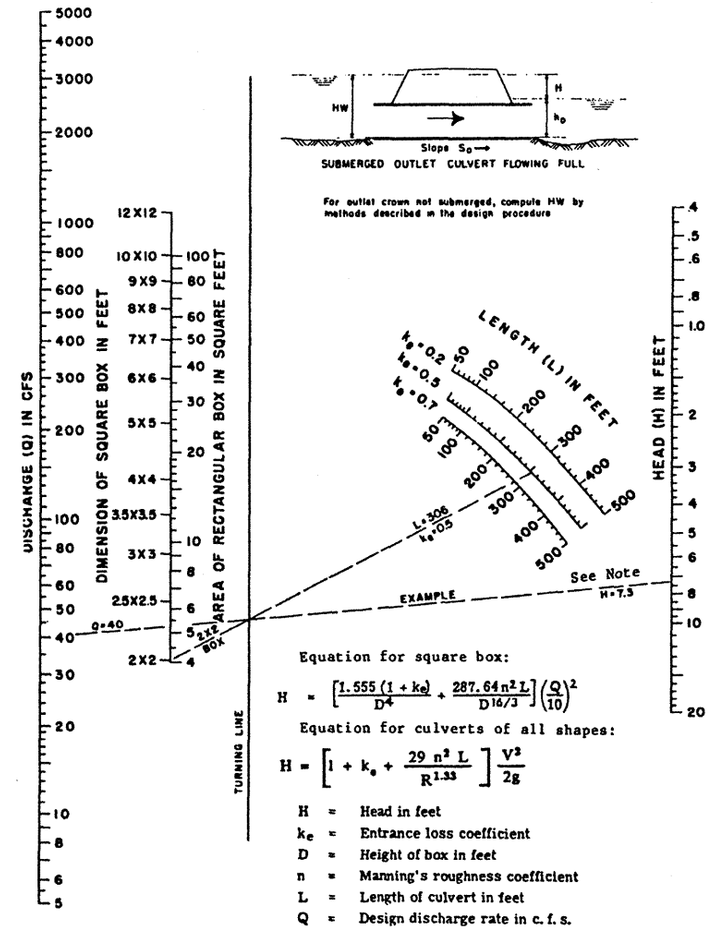
chart. With the design discharge, Manning roughness coefficient (n) from Table 7-6B (Qn value, if different from the chart) and the existing channel slope, determine the natural depth of flow, which can be the TW on the culvert. However, high water from other downstream sources may cause a higher TW value. For the (h_o) value, use either the highest TW depth or the (d_c + D)/2 value, whichever is greater. Record the TW and h_o values.

5. From the cross sections, determine the approximate length of the culvert. Refer to Figure 7-6P, Head for Concrete Box Culverts Flowing Full; n = 0.012. Align area of box and length of box on the appropriate k_e scale to establish a turning point on the Turning Line. Through this turning point, align the design discharge on the (Q) scale and intersect the Head (H) scale. This (H) value is the necessary differential head to pass the design discharge. Other values of (n) can be applied by use of the applicable equation for (H) as provided on Figure 7-6P. Very long box culverts reasonably can have several feet of acceptable differential head. Record the (H) value on the design form.

6. Refer to FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow*, Chapter 3, Rectangular Channels to establish a functional culvert slope. Enter the discharge times n, (Qn) scale (n = 0.013 for concrete box culverts). Vertically intersect the NORMAL DEPTH OF FLOW-FEET curve at maximum box height and interpolate culvert slope on the SLOPE — FEET PER FOOT curves. If appropriate, record this design slope (S) on the form. Given conditions can necessitate altering box slope and size. Project horizontally from the intersection to the Vn scale value. Dividing by n gives the normal velocity, or the velocity at normal depth (d_n), when the culvert is operating in inlet control. Retain the velocity value for later comparison.

7. Enter the product of the box length and box slope (L_aS) on the design form.

8. For the HW_{ho} depth, evaluate and record $HW_{ho} = H + h_o - L_aS$. The greater HW value, HW_i or HW_{ho}, indicates either inlet or outlet control. If HW_{ho} is allowable and less than HW_i, investigate an improved entrance (Hydraulic Design Series No. 5); otherwise, try larger or lower profile culverts.



Equation for square box:

$$H = \left[\frac{1.555\,(1 + k_e)}{D^4} + \frac{287.64\,n^2 L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$$

Equation for culverts of all shapes:

$$H = \left[1 + k_e + \frac{29\,n^2 L}{R^{1.33}}\right]\frac{V^2}{2g}$$

H = Head in feet
k_e = Entrance loss coefficient
D = Height of box in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Design discharge rate in c.f.s.
R = Hydraulic radius at full flow
V = Velocity at full flow (V = Q/A)
g = 32.2 ft/sec^2

Note: Use of n = 0.013 in equations yields H = 8.22 ft.

*More realistic "n" for concrete box culverts may be n = 0.013 where use of the equations will give more representative "H" values.

HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012 *
Figure 7-6P

9. Velocity at the outfall end of the culvert is relative to the greater or controlling HW depth. When in inlet control, the area of water prism at normal depth (d_n) is used in the equation V = Q/A, or use the charts of Hydraulic Design Series No. 3, as referenced in #6. When in outlet control, the area of water prism in the equation V = Q/A is either full depth (D) of culvert where tailwater (TW) is greater than D, TW depth where TW is less than D but greater than critical depth (d_c), or d_c where d_c is greater than TW. Outfall channel erosion control treatments should be considered where Q_{50} — velocities exceed 8 ft/s (6 ft/s for Q_{10}) and are essential for soils of less than average resistance to erosion even at lesser velocities. See Table 7-10A "Recommended Permissible Velocities for Unlined Channels."

10. In support of the cross drain culvert size selection (FHWA), determine the roadway overtopping storm. Often with the use of smaller pipe and box culverts, an overtopping storm readily may be greater than the Base Storm (100-year) and estimates for these greater Recurrence Intervals (Frequencies) must be expanded. For the selected culvert, derive the estimated discharge and Recurrence Interval (Frequency) of the storm that will first overtop the roadway. Normally, overtopping is associated with culverts located in the vicinity of a sag vertical curve; otherwise, side drain spill-overs may occur and accumulate in a sag condition. Add 0.08 ft to the low-point elevation in the roadway section where overtopping will begin and subtract the culvert entrance flow line elevation giving the headwater depth, in feet. From the nomographs for the selected culvert, determine and notate the discharge (Q) necessary to generate this headwater depth. With the discharge method used, derive the 2-, 5-, 10-, 25-, 50-, 100- (or greater) and 500-year (if available) storm discharges. On "PROBABILITY X 2 LOG CYCLES" grid paper (see Figure 7-6I), plot these discharges with respect to the respective Recurrence Intervals (Storm Frequencies). The "Probability, in Percent, of Being Exceeded Each Year" values are along the top horizontal scale — the inverse of the respective Percent is the Recurrence Interval (Frequency), in years. The Discharge, in ft^3/s, is the vertical scale. Of these plotted points, a best-fit straight-line "curve" is determined and extended to the upper limits of the grid scales. Using the overtopping discharge from above, enter the discharge scale projecting horizontally until intersecting the best-fit "curve." Read vertically upward to the Percent scale — the inverse of this Percent value is the Recurrence Interval (Frequency). Record the Recurrence Interval (Storm Frequency), x-year, and the discharge (Q) on the design form. Storm frequencies up to the 10,000-year can be estimated from the probability grid figure; however, when the overtopping storm is greater than the 500-year storm, a statement to this affect is sufficient and no further investigation is necessary with these derived values appropriately recorded.

11. Having determined the overtopping storm, perform a Risk Assessment (FHWA — see HEC No. 17), which may indicate that a more in-depth Risk Analysis is required. More typically associated to bridge crossings, the concepts are applicable to other flood plain encroachments (smaller cross drain structures). Risks to the highway facility and surrounding property must be considered. Supplying the data for the Encroachment Risk Assessment block on the Culvert Design Form serves as a guide for the assessment where the appropriate blank is checked. Generally, for pipe and box culverts within the drainage area limits of this responsibility (see Section 7-1.02), the Risk Assessment will be sufficient; however, there may be instances where a Risk Analysis is necessary (see HEC No. 17).

Example 7-6.4 illustrates the use of the above procedure to determine the outlet control headwater depth for a box culvert installation.

Example 7-6.4

Given: Same data from Example 7-6.3.

Problem: Determine the proper size box culvert with square-edge at crown for the given location (outlet control). (Double cell 6 ft x 4 ft box culvert is the initial trial size of box culvert.)

Solution: The designer should follow the procedure for outlet control presented in Section 7-6.04.02. This example is shown on the Culvert Design Form, Figure 7-6J.

1. Table 7-6A provides the entrance loss coefficient (k_e) of 0.5. Enter the value on the design form.

2. Enter Figure 7-6O, Critical Depth, with Q/B = 432/12 = 36.0. Vertically intersect the heavy curve line and horizontally project to the d_c scale, reading a value of 3.43 ft. Record d_c on the design form.

3. Calculate $(d_c + D)/2$ = (3.43 ft + 4 ft)/2 = 3.72 ft and record it.

4. Let the (TW) value be insignificant; therefore, use the $(d_c + D)/2$ value of 3.72 ft for the h_o value. Record the h_o value.

5. Refer to Figure 7-6P. Notice the nomograph is developed for n = 0.012. For box culverts, an (n) value of 0.013 should be used in the appropriate equation for (H) selected from the Figure. Since this example is not a square box, $H = [1 + k_e + (29n^2L)/R^{1.33}]\ V^2/2g$, with values of (R) and (V) at full flow conditions. The box opening area (A): (2)(6 ft)(4 ft) = 48 ft^2; wetted perimeter (WP): 2[(2)(6 ft) + (2)(4 ft)] = 40 ft; hydraulic radius (R) = A/WP = 48/40 = 1.20; and velocity (V) = Q/A = 432/48 = 9 ft/s. Therefore, $H = [1 + 0.5 + (29)(0.013^2)(100)/1.20^{1.33}]\ 9^2/(2)(32.2)$ = 2.37 ft. Record the differential head (H) necessary to pass the design discharge. (Had rounding or beveling of the entrance edges or sides and/or slope-tapering been applied, the k_e value would be 0.2 and H = 1.99 ft.)

6. Refer to FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow*, Chapter 3, Rectangular Channels. Select chart having a total width of 12 ft; enter the Qn scale [Qn = 432 (0.013)] = 5.62; vertically intersect the NORMAL DEPTH OF FLOW-FEET curve at maximum 4-ft box height and note the intersection (d_n = 3.95 ft) with 0.002 on the SLOPE — FEET PER FOOT curve (this slope can be functional in an existing channel slope of 0.004 ft/ft, matching the downstream end while developing a fall at the upstream end). Project horizontally from the intersection to the Vn scale and read 0.118; divide by n, 0.118/0.013 = 9.08 ft/s velocity at normal depth (d_n) if the culvert is operating in inlet control. Retain the velocity for later comparison and record the design slope (S) on the form. Outfall channel treatment may be required at greater than 8 ft/s velocity.

7. Record the product of the box length and box slope (L_aS), L_aS = (100 ft) (0.002) = 0.20 ft.

8. Evaluate the equation $HW_{ho} = H + h_o - L_aS$. HW_{ho} = 2.37 ft + 3.72 ft - 0.20 ft = 5.89 ft. Record this value on the design form.

9. For outfall velocity, compare HW_i = 6.36 ft and HW_{ho} = 5.89 ft. With HW_i greater, the culvert operates in inlet control and the outfall velocity is 9.08 ft/s (derived in #6 with respect to the normal depth of flow, d_n = 3.95 ft). Similarly in the expression V = Q/A, 432/(12)(3.95) = 9.11 ft/s. Had the culvert been in outlet control, one of either D, TW < D, or d_c > TW depth on the culvert should be used to derive the wetted area for the equation V = Q/A. For a soil of average resistance to erosion, this example indicates a Q_{50} - velocity greater than 8 ft/s which requires erosion control treatment in the outfall channel (minimum riprap length = 4 * height of box). The design form example uses an 14-ft flat bottom channel with 2:1 sides on a slope of 0.004 ft/ft which is shown in HDS No. 3 to have a Q_{50} - velocity of 5.75 ft/s (acceptable for average soils).

10. It is determined that a headwater depth of 9.92 ft begins overtopping the roadway. From Figure 7-6K, HW/D = 9.92 ft/4.0 ft = 2.48, extending to 4.0 ft on the (D) scale intersects the Discharge-To-Width Ratio (Q/B) at 52.0 and the discharge (Q) = (52)(B) = (52)(6 ft)(2)= 624 ft^3/s. The respective discharges, 2-, 5-, 10-, 25-, 50- ,100-, 200- and 500-year are 162, 244, 304, 374, 432, 477, 539 and 598 ft^3/s. On "PROBABILITY X 2 LOG CYCLE" grid paper, Figure 7-6I, plot these Discharges to the respective Recurrence Intervals (Storm Frequencies). Determine the best-fit straight-line "curve," extending upper limit. On the Discharge scale at the overtopping discharge (624 ft^3/s), project horizontally to "curve" line. Read upward to Percent scale of approximately 0.15% — the inverse is the Recurrence Interval (Frequency) which is 1/0.0015 = 667-year storm. Record the Recurrence Interval (Storm Frequency) and the Overtopping Discharge (Q) on the design form.

11. For the last step in this box culvert design, perform an encroachment Risk Assessment. Mark the guide blocks at the lower portion of the design form. Using the Base Storm (100-year) for the analysis of category 1, Hazard to People and Property (no critical elevations existing below the 100-year headwater), the respective three responses are "NO" (mark through "YES" in each). For category 2, Other Factors, Use and Location, the respective three responses are "NO" (mark through "YES" in each). For the Base Storm, the associated Risk at this site appears to be acceptable — check the Risk Acceptable block on the design form. Had the Risk not been Acceptable, a Risk Analysis would be necessary (refer to HEC No. 17).

    With HW_i greater than HW_{ho}, the designer should investigate an improved entrance which could result in decreasing the size of culvert. Refer to Hydraulic Design Series No. 5. Improved entrance structures should be considered, especially when a substantial reduction in cost may be realized by reducing the size of long culverts and by lowering a potentially detrimental headwater (see improved entrance curves by FHWA at end of Chapter).

7-7.0 MEDIAN DRAINAGE

For medians between widely separate roadways (≥ 125 ft ℄ to ℄), adequate drainage is provided by normal cross drains under the roadway. Where narrower medians exist (< 125 ℄ to ℄), it is often necessary to provide intermittent drop inlets ideally located coincident with the cross drains; however, separate median drain outlet culverts should be installed where necessary.

7-7.01 Design Criteria

Drainage design for depressed medians and interchange areas usually is based on the Rational Method because of the relatively small areas involved. Those portions of the roadway surface sloped toward the median should be included in the computed drainage area. Median inlets should be placed at the low point of sag vertical curves. The reach in either direction is critical for sag inlets and special care should be exercised in design (see Table 7-7A). For long continuous runs, intermittent inlets will be necessary.

7-7.02 Median Drain Inlet Spacing

For divided roadway sections separated 88 ft or less (centerline to centerline), median drains should be spaced to maintain either no greater than a 1-ft depth of flow or no rise above the level of the subgrade for the 10-year storm. Although the median ditch may have a greater capacity on steeper grades, the designer should apply a more conservative drain spacing distance of 700 to 800 ft nominal and 1000 to 1200 ft maximum on grades of 1.0 percent and greater. The designer should also examine the possibility of riprap or paved ditches where steep grades exist.

When zero percent grades are necessary, median ditch grades should be "warped" from back and ahead toward the drain. The maximum recommended drain spacing is 300 ft in either direction at a 0.2% minimum gradient.

Table 7-7A provides recommended inlet spacing criteria for sag vertical curves and various side slopes on divided highways with medians 88 ft or less (centerline to centerline).

Divided facilities with medians in excess of 88 ft will usually maintain individual ditch sections.

Table 7-7A
RECOMMENDED MEDIAN DRAIN INLET SPACING
(Sag Vertical Curve)
88 ft, ℄ to ℄

Design Criteria	Side Slopes			
	Both 6:1	6:1 and 4:1	Both 4:1	Both 3:1
Total Reach Back and Ahead (ft)	600	500	400	300
Pipe Size (in)	18	18	18	18
Q (ft^3/s)	8.6	7.1	5.7	4.3

Note: Sag vertical median drains are more critically limited in reach. The 600 ft for the 6:1 side slopes is the total reach capacity for the drain in two directions (300 ft back and 300 ft ahead).

7-7.03 Standard Inlets

The most current safety measures should be incorporated in all median inlet design. The designer should strive for safe traversability and avoid protruding edges on all median inlets and cross drains. The following types of median inlets are available in the *Roadway Design Standard Drawings*:

1. Median Inlets for Pipe and Box Culverts [Standard Drawing MI-1 and MI-3 (Type I)] — Drop Inlets. These designs and modifications are used when either larger inlets are necessary or when the median flow line to the cross drain invert depth is nominally less than 4 ft 2 in. Skewed cross drains are adapted with specific inlet dimensioning. Inlet MI-1B is modified to connect into large pipe cross drains to eliminate unnecessarily large inlets and junction boxes.

2. Median Inlets with Stack Pipe [Standard Drawing MI-2 and MI-3 (Type II)] — Drop Inlets. These designs and modifications should be used for minimal inlets where the median flow line to the cross drain invert depth is more than 4 ft 2 in. Inlet MI-2A is modified to connect into large pipe cross drains.

3. Low Profile Median Inlet (Standard Drawing MI-4) — Drop Inlet with Grate on Foreslope. This design and modification should be used when outfall elevations are critical and abbreviated inlet heights are necessary. Standard Drawing PA-1 details a paved apron for median inlets and should be designated at all median inlet locations.

7-8.0 STORM SEWER DESIGN

Within the influence of urban areas, storm runoff waters collected in the roadway normally will be controlled with curb and gutter sections and will run off through storm sewer structures. These drainage systems should limit the water ponding along the gutters and behind the curbs to amounts that will not interfere with traffic or damage property. This is accomplished by placing either curb-opening or gutter inlets, or a combination of these, or drop inlets at specific intervals and points.

Storm sewer discharges are usually determined by the Rational Method using a 10-year storm and 5-minute minimum duration, except in sag vertical curves where the 50-year storm design is required. Clogging potential encourages the use of combination curb-opening inlets (more self-cleaning) and gutter inlets (more functionally efficient).

7-8.01 Determining Runoff Quantities

The designer should use the following guidelines in the development of a storm sewer system.

1. A set of working prints, 1″ = 100′ scale, is convenient to use in determining the initial overall drainage patterns of the storm sewer system.

2. With the aid of field information, quadrangle maps and aerial photographs, outline on the working prints the drainage area limits that will be collected along the roadway and determine the cumulative amounts of runoff discharge. Table 7-8A provides the recommended runoff coefficients (C) for the Rational Method.

3. For long, uniform sections, it is helpful to compute a constant specific discharge per unit of length. This may be done by measuring the drainage area for a typical section (1000 ft for example), computing the total discharge for that section, and dividing by 10 to determine the average discharge per 100-ft station.

   For example, to calculate the discharge for one-half a roadway for a 1000-ft distance, add the widths of half the median, (0.5)(20 ft) = (10 ft), two lanes, (2)(12 ft) = (24 ft), an auxiliary lane, (12 ft), the curb and gutter section, (2.75 ft), and any typical behind-the-curb distance of over-the-curb flow, (5 ft). Thus, 10 ft + 24 ft + 12 ft + 2.75 ft + 5 ft = 53.75 ft.

   Multiply the width by the distance, (53.75 ft)(1000 ft) = (53,750 ft^2), and divide by square feet per acre, (53,750/43,560 = 1.23 acres). Using the Rational Method:

   Q_{Design} = CIA = (0.9)(7.6)(1.23)
   = 8.41 ft^3/s per 1000 ft, or 0.841 ft^3/s per station (8.41/10) of uniform roadway section.

4. Some sections of roadway will indicate significant runoff discharge flowing over the curb. Where practical, adequate measures to intercept this flow along with other flow behind the curb should be provided. These discharges also should be included in the tabulations for design purposes.

Table 7-8A
COEFFICIENT OF RUNOFF (C)
(Rational Method)

Surface	"C"
Paved roadway surfaces, shopping centers, etc.	0.9
Roadway slopes, grassed commercial areas.	0.7
Grassed urban areas (average).	0.66
Rural areas (average).	0.5

5. It is desirable to locate the obvious inlet locations on the 1″ = 100′ scale prints. Generally, regardless of the amount of discharge to be accommodated, curb inlets should be located:

   a. upstream of street intersections (cross streets and crosswalks),

   b. in sag vertical curves,

   c. at the point on the outside of a curve prior to the beginning of superelevated sections, and

   d. at bridge ends and approaches.

Intermediate curb inlets should be spaced according to the design criteria.

7-8.02 Ponding Limits on Roadway

Water flow in the gutters should be confined to a width and depth that will neither obstruct nor cause hazard to traffic. The ponding width will have a corresponding depth and the quantity of water carried depends on the gutter gradient, gutter roughness coefficient, roadway cross slope and inlet spacing. Table 7-8B provides the recommended limits of roadway ponding based on the highway classification.

The designer should use Figure 7-8A for gutter capacity design. The figure is self-explanatory, with instructions and examples. The ponding width or spread (T) is established from the recommended criteria in Table 7-8B and the design discharge (Q) is determined by the Rational Method. With given restraints on ponding, a determination can be made of the allowable accumulated runoff discharge that the gutter can accommodate. This, in turn, will indicate the required spacing of inlets. Example 7-8.1 illustrates the use of Figure 7-8A to determine curb and gutter capacity.

Table 7-8B
RECOMMENDED ROADWAY PONDING LIMITS

Highway Classification	Ponding Limit
Freeways and multilane highways with full shoulder width	Width of Shoulder
Controlled access highways with less than full shoulder width	One-half of adjacent lane
Other multilane highways	Width of adjacent lane
Two-lane, two-way highways	One-third of adjacent lane

Example 7-8.1

Given: Longitudinal gradient (S) = 0.04
Cross slope gradient (S_x) = 0.03
Ponding limit (T) = 6 ft
n = 0.016

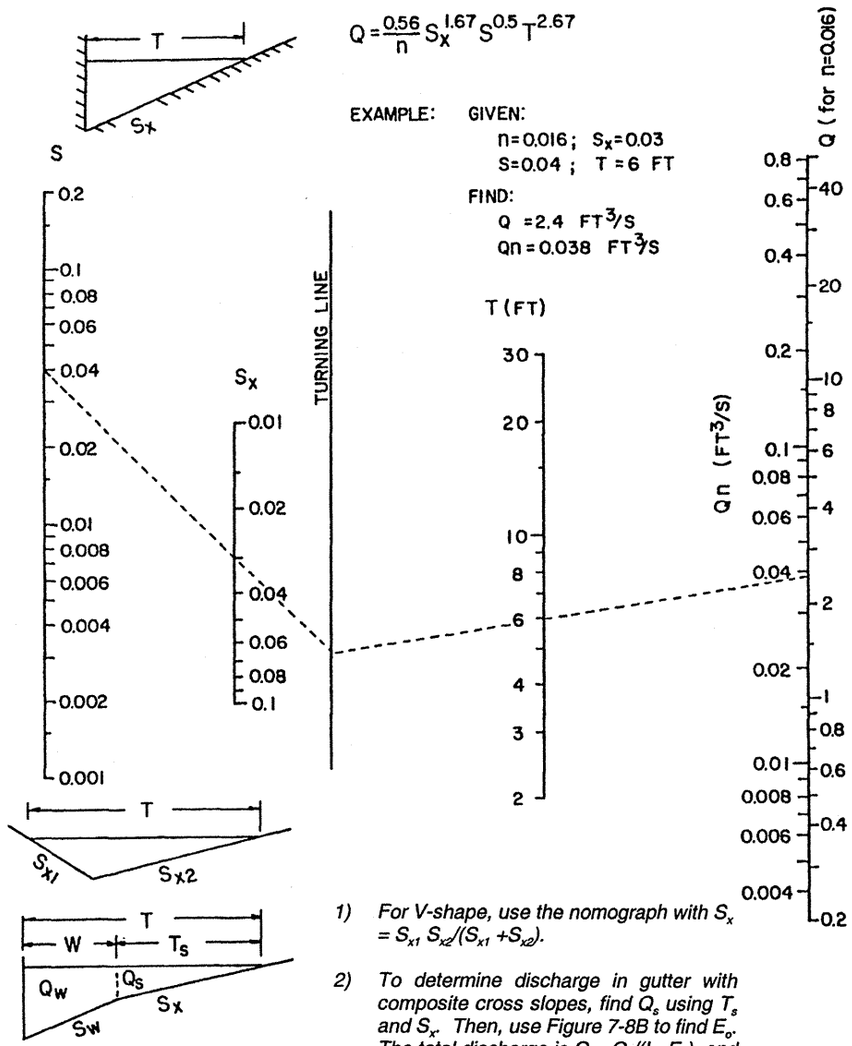
Problem: Determine the gutter flow capacity for the given location.

Solution: From Figure 7-8A:

1. Given the longitudinal gutter gradient, S = 0.04, connect a straight line through the cross slope gradient, S_x = 0.03, to intersect the turning line.

2. Connect a straight line from the turning line point through the allowable ponding width (spread), T = 6 ft, projecting to the discharge, Q, and the (Qn) (Manning n value) scale. Read Q = 2.4 ft^3/s and Qn = 0.038 ft^3/s.

   For an allowable spread of 6.0 ft on the respective longitudinal and cross slope gradients, it is shown that the capacity of this curb and gutter section equals 2.4 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 Figure 7-8B 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
Figure 7-8A

7-8.03 Inlets

Generally, gutter inlets with the grate openings parallel to the water flow are more efficient than curb-opening inlets. When gutter inlets are subject to bicycle and pedestrian traffic, the grate openings must ensure safe traversability. The resulting narrow longitudinal openings are susceptible to clogging. These gutter inlets in combination with a curb opening sufficiently sized for reception of debris can be very effective. The procedures for determining inlet capacity and spacing of inlets are presented in the following sections.

7-8.04 Inlets on Grade

7-8.04.01 Gutter Grate Inlets (On Grade)

With the grate bars aligned parallel to the water flow, all the gutter flow passing over the grate inlet (frontal flow) will be intercepted when the grate is free of debris, the inlet is sufficiently long, and the gutter flow velocity is moderate. In addition, there will be side flow interception where the cross slope is an included factor for these grate conditions. The following procedure should be used to determine the interception of gutter grates on grade.

1. Refer to Figure 7-8A to compute the flow in triangular gutter sections as illustrated in the figure.

2. Figure 7-8B is provided for use with Figure 7-8A to find the flow in a width of gutter (W), less than the total spread (T). E_o represents the ratio of frontal flow to total gutter flow.

3. Figure 7-8D provides the ratio of frontal flow intercepted to total frontal flow (R_f). At flow velocities (V) greater than splash-over velocities (V_o), grate efficiency intercepting frontal flow is diminished.

4. Figure 7-8F provides the ratio of side flow intercepted to total side flow (R_s).

5. The efficiency of (E) of a grate is expressed as follows:

$$E = R_f E_o + R_s (1 - E_o)$$

6. The interception capacity of a grate inlet is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q [R_f E_o + R_s (1 - E_o)]$$

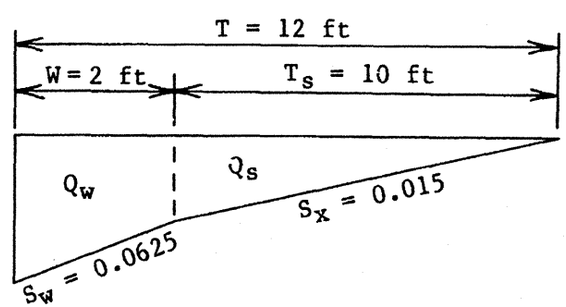
Grate inlets on grade should provide 25% allowance for the possibility of becoming clogged (i.e., the interception capacity should be reduced by 25%, with the inlet spacing set accordingly).

Example 7-8.2 illustrates the use of this procedure to determine the interception capacity of a gutter grate on grade.

Example 7-8.2

Given:

n	= 0.016	L	= 2.38 ft
S	= 0.015	Q	= 3.53 ft³/s
S_x	= 0.015	T	= 12 ft
S_w	= 0.0625		



COMPOSITE GUTTER SECTION

Problem: Determine the interception capacity (Q_i) of the gutter grate inlet on grade.

Solution: Following the procedure in Section 7-8.04.01:

1. From Figure 7-8A, with composite slopes (given $S = 0.015$ and $S_x = 0.015$), connect a straight line intersecting the turning line; connect a line from the turning point through the allowable spread, T_s, ($T_s = T - W = 12$ ft $- 2$ ft $= 10$ ft); project to discharge, Q, $Q = Q_s$; and read $Q_s = 1.80$ ft³/s.

2. From Figure 7-8B, enter the horizontal scale, $W/T = 2$ ft/12 ft = 0.167; read upward to $S_w/S_x = 0.0625/0.015 = 4.167$; and project horizontally to $E_o = 0.49$. Total flow in the gutter section (Q): $Q = Q_s/(1 - E_o) = 1.80/(1 - 0.49) = 3.53$ ft³/s. Flow in 2-ft width (W): $Q_w = Q - Q_s = 3.53 - 1.80 = 1.73$ ft³/s.

   Note: Figure 7-8C, flow in composite gutter sections, is a direct solution of flow in composite sections; however, extreme care in usage is necessary to obtain accurate results.

3. From Figure 7-8D (given L = 2.38 ft), read vertically to the P - 1-⅛ grate curve; project horizontally to the left side of the figure; and read $V_o = 7.0$ ft/s; project horizontally to the right side of the figure intersecting the gutter velocity, V, which equals the total gutter flow divided by the (A) area of flow.

   A of $Q_s = (0.15)(10)/2 = 0.75$ ft²
   A of $Q_w = 2(0.275 + 0.15)/2 = 0.425$ ft²
   $A_{total} = 1.175$ ft²
   $V = Q/A_{total} = 3.53/1.175 = 3.00$ ft/s

   Projecting to the right side of the figure, an R_f value of 1.0 is intercepted before reaching V = 3.00 ft/s; use $R_f = 1.0$.

Note: Figure 7-8E, velocity in triangular gutter sections, is a direct solution to the flow velocity, not to be used in composite sections.

4. From Figure 7-8F (given $S_x = 0.015$), move horizontally to L = 2.38 ft; read upward to V = 3.00 ft/s; project horizontally to the left of the figure; and read $R_s = 0.10$.

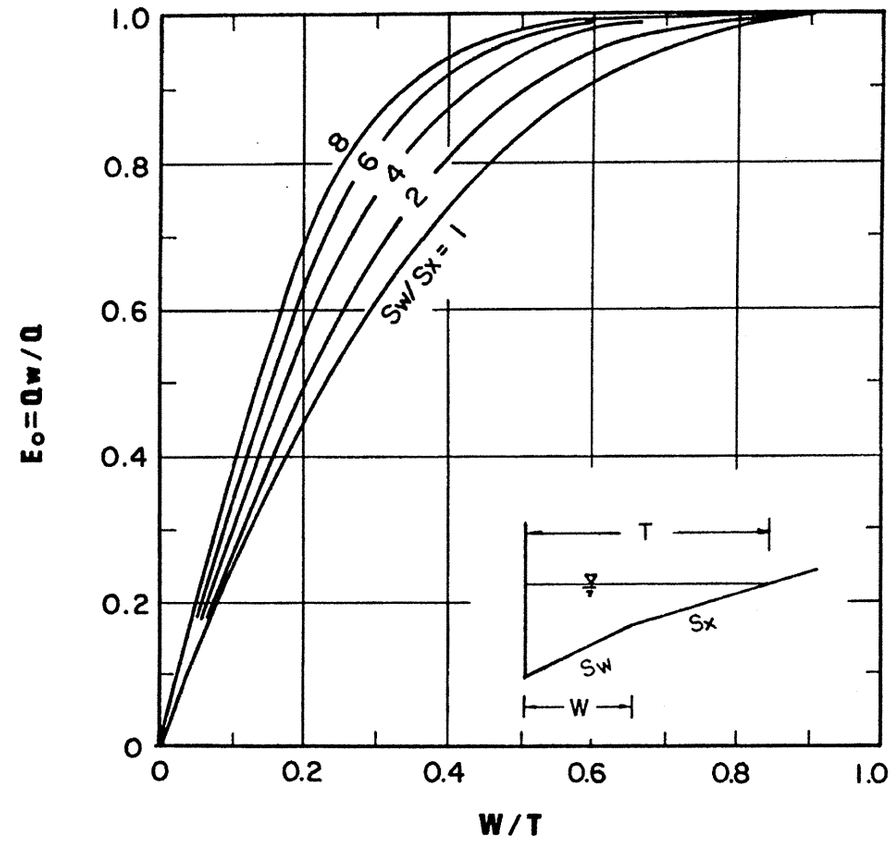
5. The grate efficiency (E):

$$E = R_f E_o + R_s (1 - E_o)$$
$$= 1.0 (0.49) + 0.10 (1 - 0.49) = 0.54.$$

6. The interception capacity of the grate inlet (Q_i):

$$Q_i = EQ = 0.54 (3.53) = 1.91 \text{ ft}^3/\text{s}$$

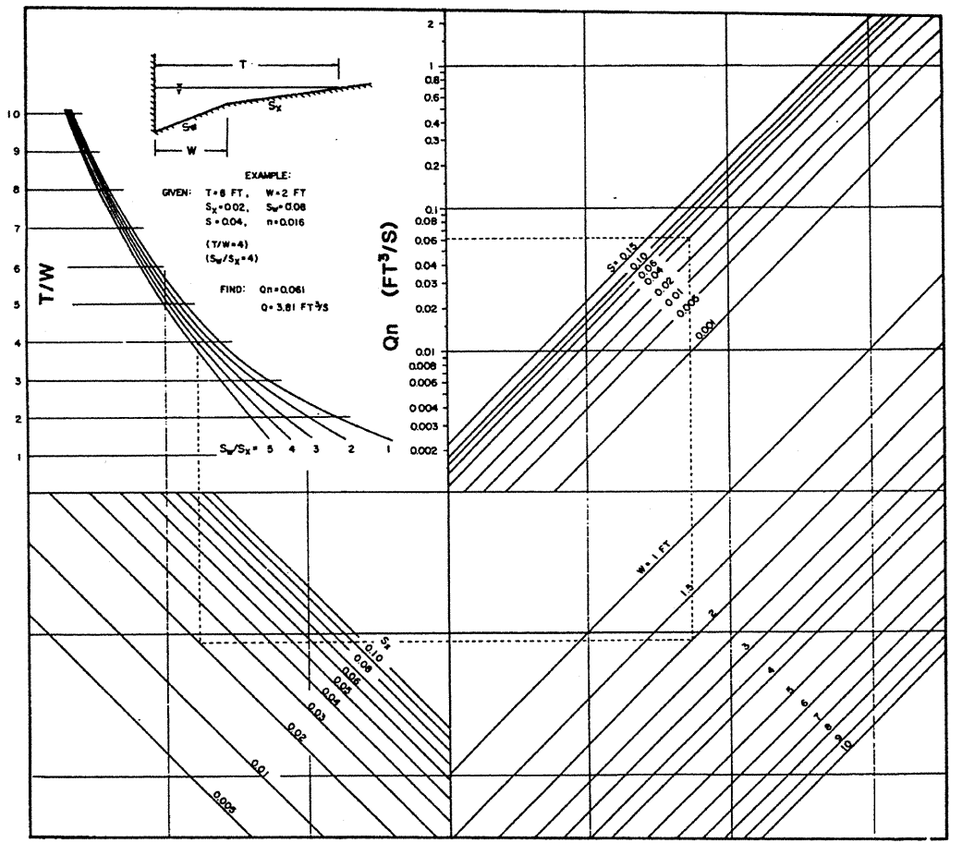
The width of flow or spread (T) in this example is 12 ft with the inlet receiving only 1.91 ft^3/s of the total 3.53 ft^3/s. An additional reduction due to clogging by 25% yields reception of 1.43 ft^3/s and 2.10 ft^3/s as carry-over to the next inlet. If this is not acceptable, it will be necessary to either reduce the inlet spacing or lengthen the inlet opening with multiple inlets.



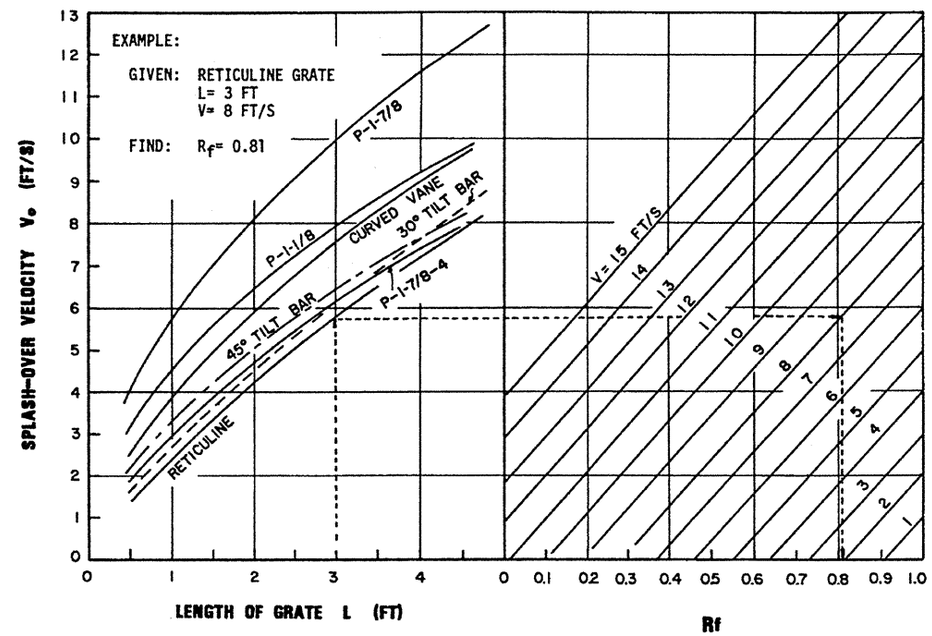
$$E_o = \frac{Q_w}{Q} = 1 - (1 - W/T)^{2.67}$$

Where: *Q = total gutter flow,* ft^3/s
Q_w = flow in width W, ft^3/s
W = width of depressed gutter or grate, ft
T = total spread of water in the gutter, ft

RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW
Figure 7-8B



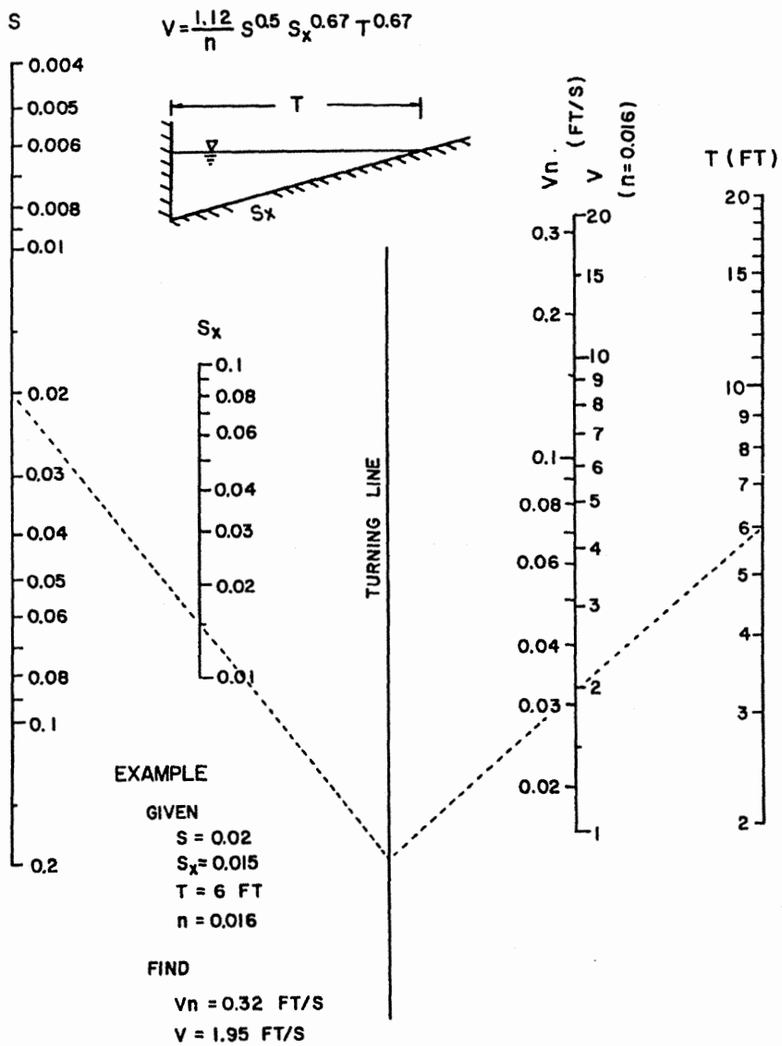
FLOW IN COMPOSITE GUTTER SECTIONS
Figure 7-8C



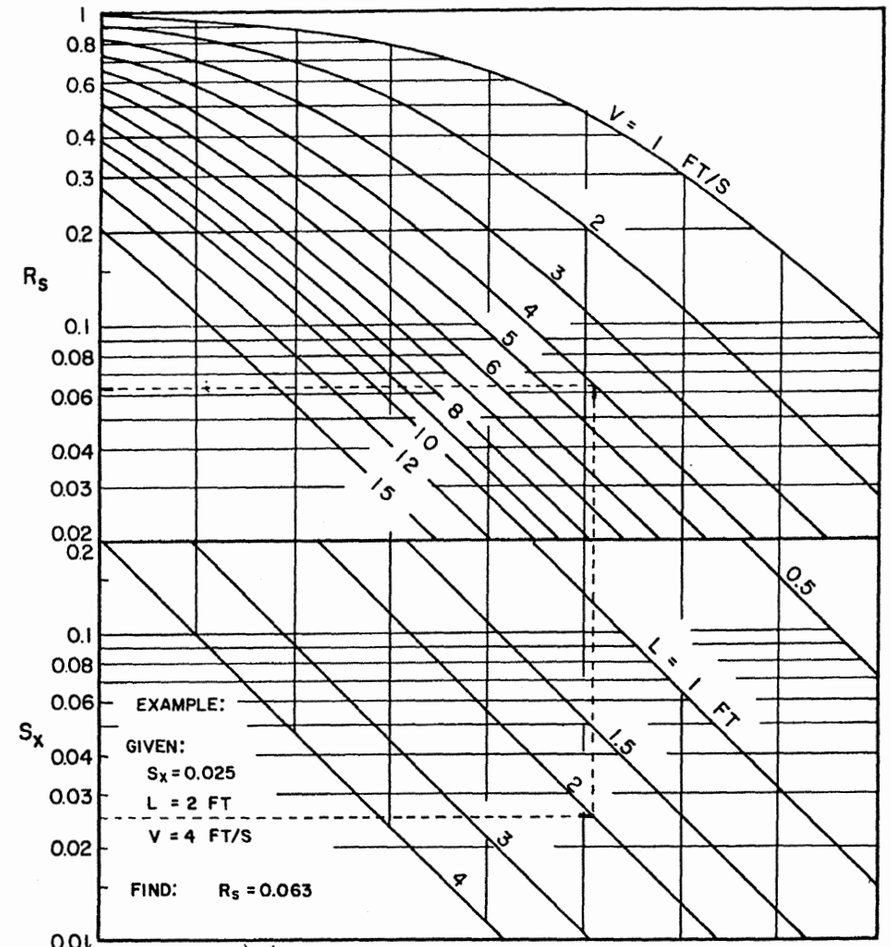
$$R_f = 1 - 0.09\,(V - V_o)$$

Where: V = *velocity of flow in the gutter, ft/s*
V_o = *gutter velocity where splash-over first occurs, ft/s*

GRATE INLET FRONTAL FLOW INTERCEPTION EFFICIENCY
Figure 7-8D



VELOCITY IN TRIANGULAR GUTTER SECTIONS
Figure 7-8E



$$R_s = \left(1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}\right)^{-1}$$

Where: L = *length of the grate, ft*
S_x = *cross slope, ft/ft*

GRATE INLET SIDE FLOW INTERCEPTION EFFICIENCY
Figure 7-8F

7-8.04.02 Curb-Opening Inlets (On Grade)

The following procedure should be used to determine the interception of curb-opening inlets on grade.

1. Refer to Figure 7-8A and C to determine the width of gutter flow or spread (T).

2. Refer to Figure 7-8B for use with Figure 7-8A to find the flow in width (W), less than the total spread (T).

3. Figure 7-8G provides the length of curb-opening inlet (L_T) necessary to intercept the total gutter flow.

4. The efficiency (E) of curb-opening inlets shorter than the total interception length is presented in Figure 7-8H.

5. The interception capacity of a curb-opening inlet is equal to the efficiency of the inlet multiplied by the total gutter flow:

$$Q_i = EQ = Q\,[1 - (1 - L/L_T)^{1.8}]$$

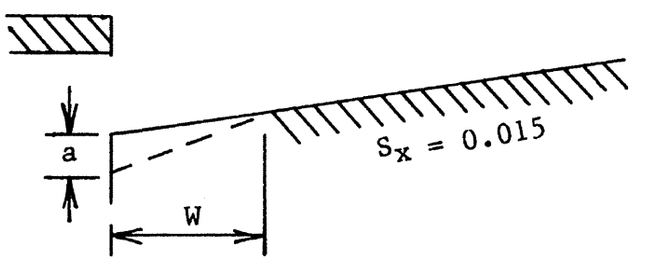
Example 7-8.3 illustrates the use of this procedure to determine the interception capacity of a curb-opening inlet on grade.

Curb-opening inlets having a curb opening height of 4 inches or greater have less tendency to clog. Where curb-opening inlets are applicable, combination gutter grate inlets and curb-opening inlets are an effective consideration. Compare the interception capacity of grate inlets to the interception capacity of curb-opening inlets for the same conditions and discharge (Q). Combination inlets are discussed in detail in the FHWA publication, *Drainage of Highway Pavements,* Hydraulic Engineering Circular No. 12, March 1984.

Example 7-8.3

Given:

n	= 0.016	L	= 5 ft increments
S	= 0.015	Q	= 3.53 ft^3/s
S_x	= 0.015	a	= 0.125 ft
S_w	= 0.0625	W	= 2.0 ft



Problem: Determine the interception capacity of the curb-opening inlet on grade.

Solution: Following the procedure in Section 7-8.04.02:

1. From Figure 7-8C, enter the Qn scale [Qn = 3.53 (0.016)] = 0.056; proceed through figure (given S = 0.015, W = 2.0 ft, S_x = 0.015), S_w/S_x = 0.0625/0.015 = 4.167; project horizontally to the left of the figure; and read T/W = 6, T = (6)(2) = 12 ft.

2. From Figure 7-8B, enter the horizontal scale, W/T = 2 ft/12 ft = 0.167; read upward to S_w/S_x = 0.0625/0.015 = 4.167; and project horizontally to E_o = 0.49.

3. From Figure 7-8G, with composite slopes, connect a line through n = 0.016 and S = 0.015 to intersect the first turning line; connect a line from the first turning point through S_e [$S_e = S_x + S_w' E_o$, $S_w' = a/W = 0.125/2.0 = 0.0625$, S_e = 0.015 + 0.0625 (0.49) = 0.0456] to the second turning line; project a line from the second turning point to discharge (given Q = 3.53 ft^3/s) intersecting L_T; and read L_T = 22 ft.

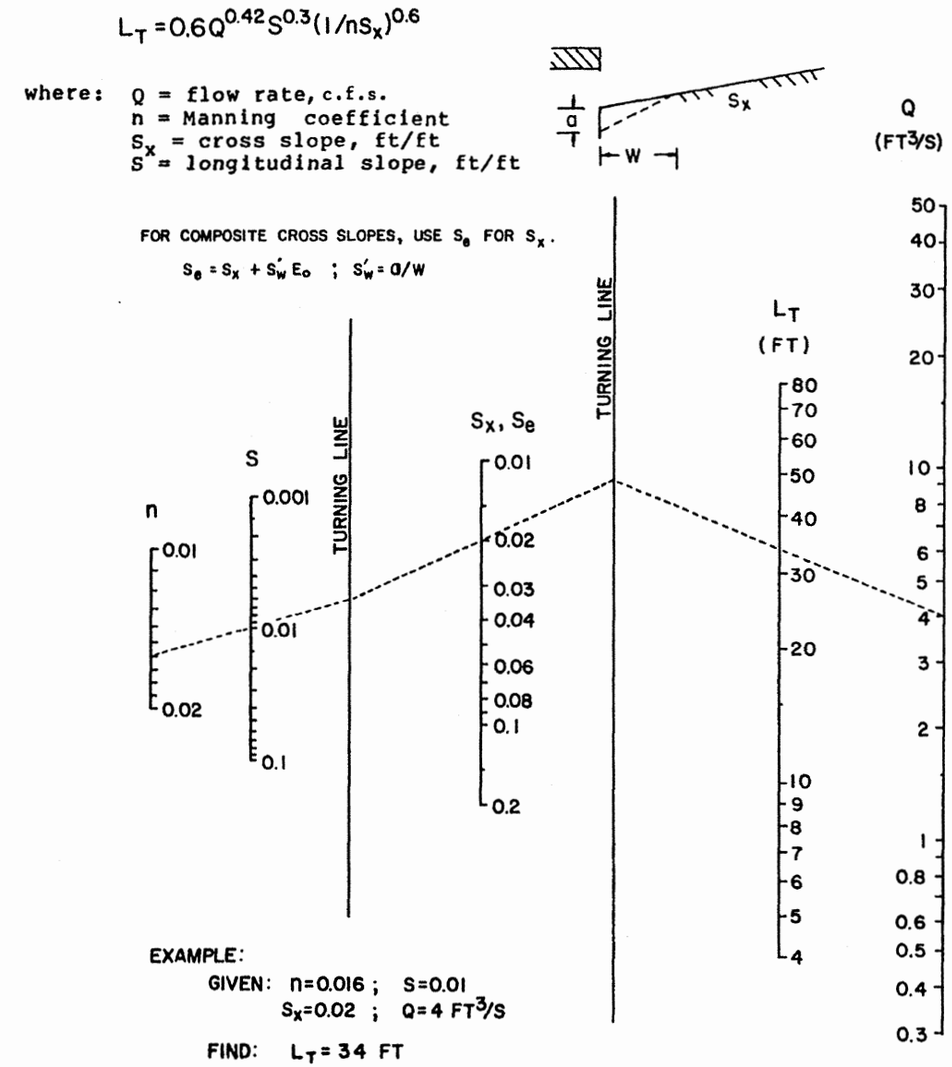
4. From Figure 7-8H, L is in 5-ft increments with a maximum of 15-ft for a single inlet; enter the horizontal scale, L/L_T = 5/22 = 0.227, 10/22 = 0.455, 15/22 = 0.682; read vertically to the curve; and project horizontally to E at 5-ft = 0.37, at 10-ft = 0.66, at 15-ft = 0.87.

5. The interception capacity of the curb opening (Q_i):

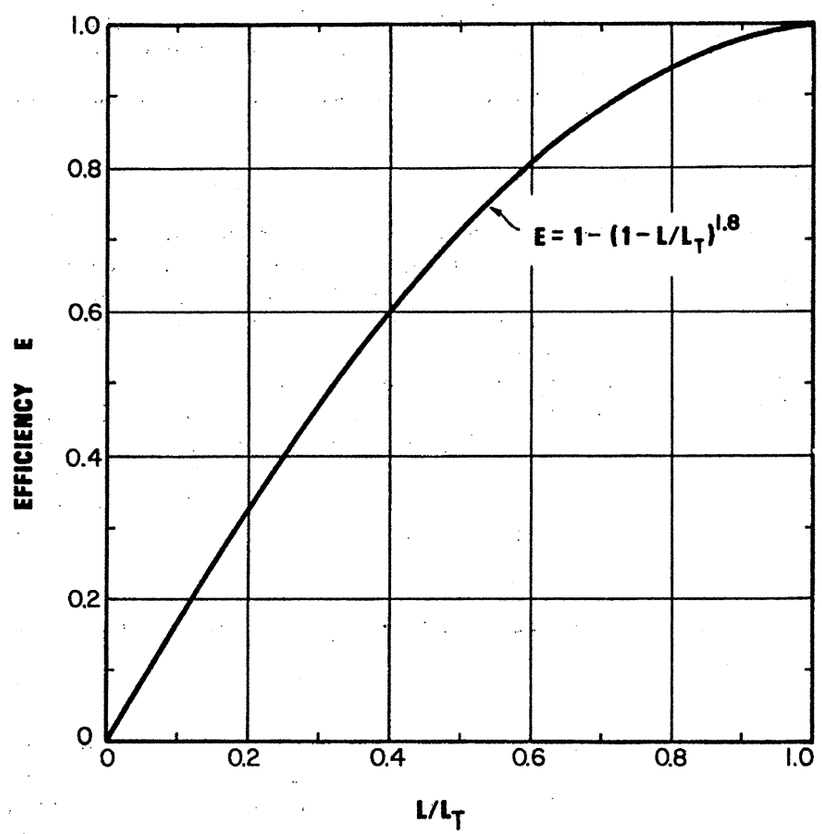
   $Q_i = E\,Q$

   L at 5-ft, Q_i = 0.37 (3.53) = 1.31 ft^3/s
   L at 10-ft, Q_i = 0.66 (3.53) = 2.33 ft^3/s
   L at 15-ft, Q_i = 0.87 (3.53) = 3.07 ft^3/s

In this example, 3.07 ft^3/s can be intercepted by a 15-ft curb-opening inlet with 0.46 ft^3/s carry-over of the total 3.53 ft^3/s, which is a reasonable design efficiency. Notice with similar design parameters (Example 7-8.2), the gutter grate inlet length of only 2.38 ft intercepted over one-half of this total gutter flow.



CURB-OPENING AND SLOTTED DRAIN INLET LENGTH FOR TOTAL INTERCEPTION
Figure 7-8G



$E = 1 - (1 - L/L_T)^{1.8}$

Where: L = *curb-opening length, ft*
L_T = *curb-opening length required for total gutter flow interception, ft*

CURB-OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY
Figure 7-8H

7-8.04.03 Slotted Inlets

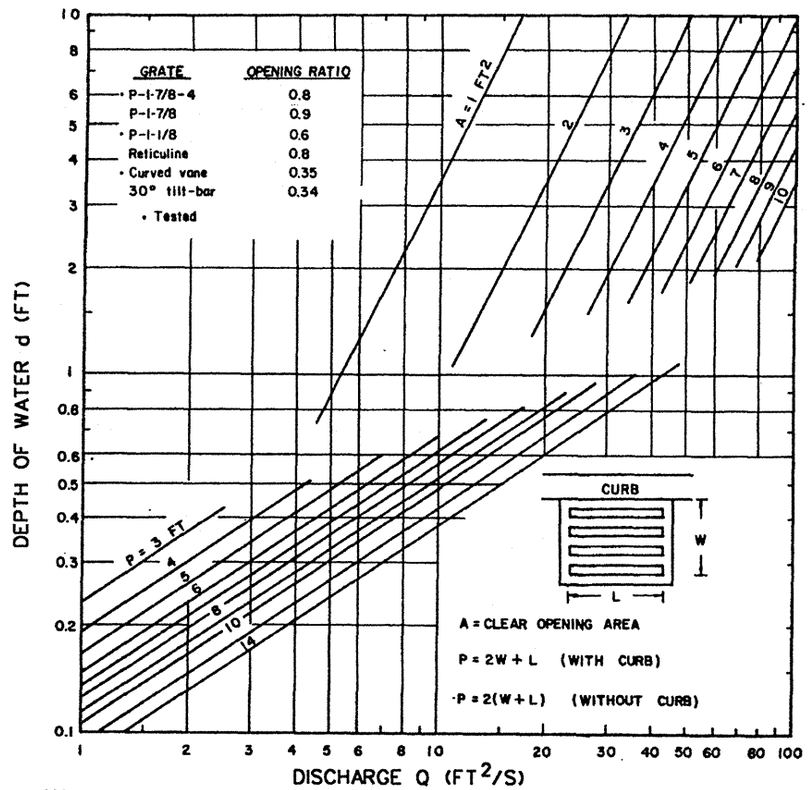
Slotted inlets can be used with curbed or uncurbed sections and provide effective pavement drainage. Deposition in the pipe may be encountered, but the inlet is accessible for high-pressure waterjet cleaning. Economic considerations dictate that adding length to a slotted inlet is less costly than adding length to a curb-opening inlet. However, at present, the main pipe line of the slotted inlet requires the use of corrugated ferrous metal pipe, should this be a limiting factor. When the slot opening is at least 1.75 in width, the slotted inlet functions similarly to the curb-opening inlet, and Figures 7-8G and H are applicable to both types of inlets on grade.

7-8.05 Inlets in Sag Vertical Curves or Sump Conditions

7-8.05 .01 Grade Inlets

Figure 7-8I provides the grade inlet capacity in sump conditions. A grate inlet will operate as a weir to depths dependent on bar configuration and grate size and as an orifice at greater depths. The effective opening of the grate and/or frame should be determined by the total clear opening width (W) between the bars and the total clear opening length (L) along the length of the grate where the effective perimeter (P) and clear opening area (A) are derived. Example 7-8.4 illustrates the use of Figure 7-8I to determine the spread (T) and the interception capacity of a grate inlet in sump conditions.

Spacing of inlets along the approaches of sag vertical curves is a critical design consideration. Clogging of grate inlets in sag conditions is a major concern, and a 50% minimum allowance is reasonable. For sag inlets, the effective clear opening area is reduced by 50%, and the perimeter will be reduced by about 25%. The grate capacity is then reconsidered with these reduced parameters, and either the inlet spacing or the number of necessary inlets is set accordingly. Consideration of a combination grate inlet and curb-opening inlet is warranted with the tendency of grate inlets to become clogged.



Weir

$Q_i = C_w P d^{1.5}$

Where: P = *perimeter of the grate in feet, disregarding bars and the side against the curb*
C_w = *3.0 grate inlet*
1.66 slotted inlet

Orifice

$Q_i = C_o A(2gd)^{0.5}$

Where: C_o = *Orifice coefficient* = *0.67*
A = *clear opening area of the grate, ft²*
g = *32.16 ft/s²*

GRATE INLET CAPACITY IN SUMP CONDITIONS
Figure 7-8I

Example 7-8.4

Given: $Q = 3.53\ ft^3/s$ Standard gutter grate: $L = 2.38$ ft, $W = 0.85$ ft, $S_x = 0.015$ ft/ft

Problem: Determine the spread (T) for the given flow in sump conditions.

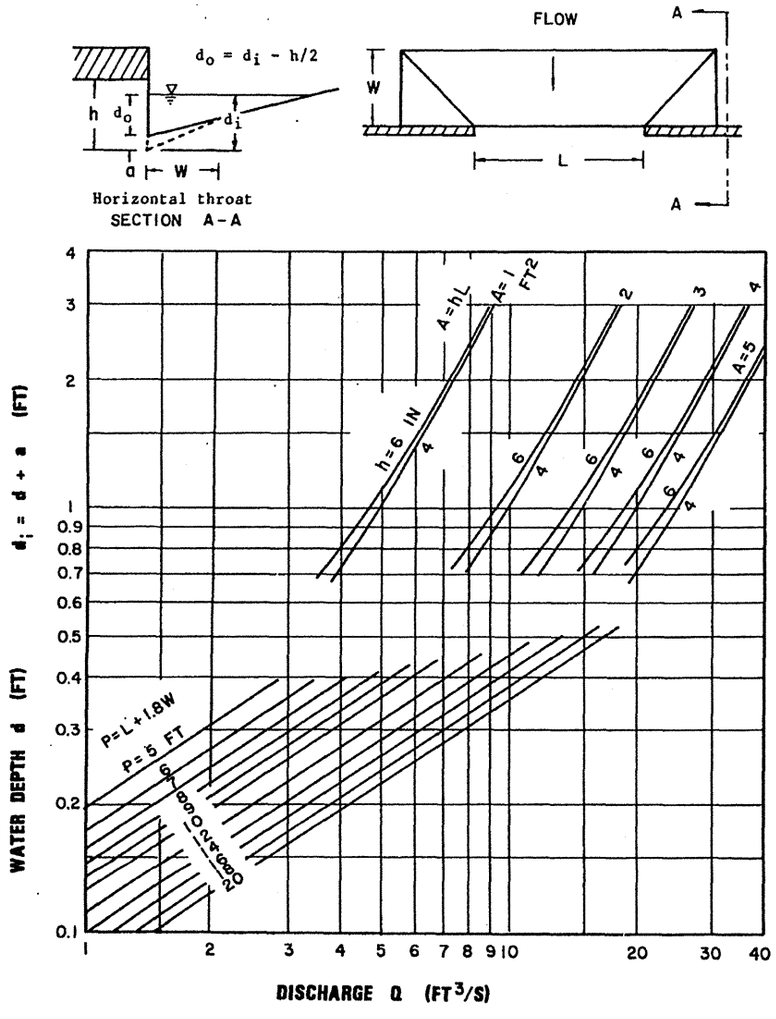
Solution: Perimeter of the grate (with curb), P = L + 2W = 2.38 ft + 2(0.85 ft) = 4.08 ft. Clear opening area of the grate, A = LW = (2.38 ft)(0.85 ft) = 2.03 ft².

From Figure 7-8I, enter the horizontal scale, Q = 3.53 ft³/s; read upward to P = 4.08 ft; project horizontally to the depth of the water, d, and read d = 0.44 ft; T (spread) = d/S_x = 0.44 ft/0.15 = 29 ft.

This ponding limit is probably prohibitive (see Section 7-8.02) and should normally be about 14 ft or less. Consider T = 14 ft and d = 14(0.015) = 0.21 ft. From Figure 7-8I, for this grate, Q = 1.2 ft³/s. The inlet has sufficient capacity; however, the allowable spread (T) is the controlling design parameter.

7-8.05.02 Curb-Opening Inlets

In sag conditions, curb-opening inlets operate as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 multiplied by the opening height. At depths between these two limits, the flow is in a transition stage. Figures 7-8J and 7-8K provide the curb-opening inlet capacity in sump conditions. Figure 7-8K is used when there is no additional depression at the entrance to the inlet. Example 7-8.5 illustrates the use of these figures to determine the interception capacity of a curb-opening inlet in sump conditions.



Weir (with depression):

$$Q_i = C_w (L + 1.8W)\, d^{1.5}$$

Where:

- C_w = 2.3 curb-opening inlet
  1.25 slotted inlet
- L = length of curb opening, ft
- W = lateral width of depression, ft
- d = d_o = depth at curb measured from the normal cross slope, ft; i.e., $d = TS_x$

Weir limitations:

$d \leq (h + a) / 12$

- h = height of curb-opening inlet, in
- a = depth of depression, in

Orifice:

$$Q_i = C_o hL(2gd_o)^{0.5} = C_o A[2g(d_i - \frac{h}{2})]^{0.5}$$

Where:

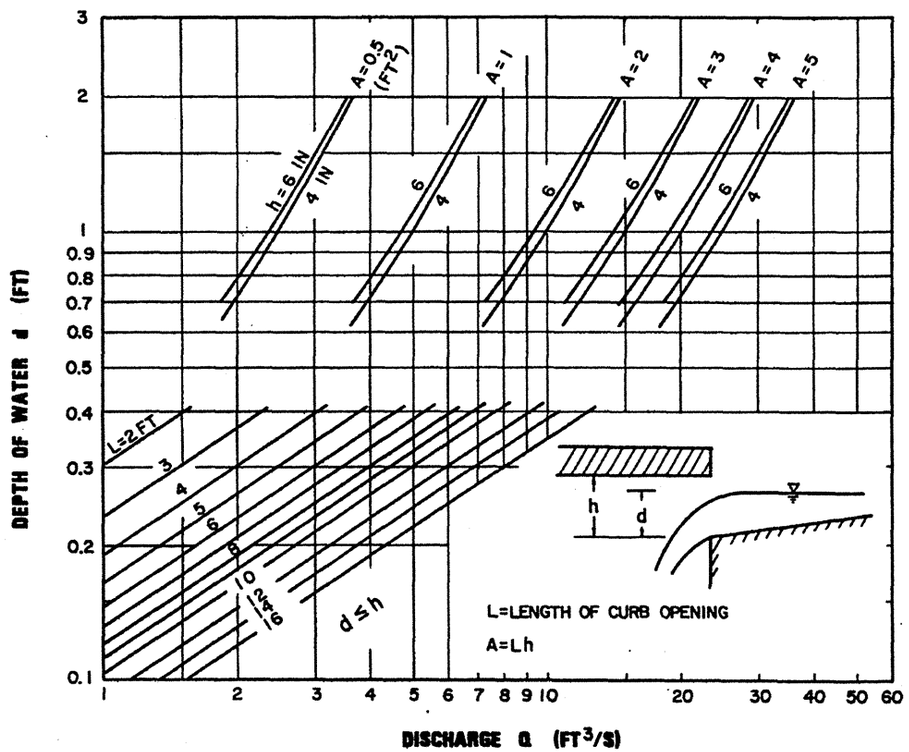
- C_o = orifice coefficient
  = 0.67
- h = height of curb-opening inlet, ft
- d = d_o = effective head on the center of the orifice throat, ft
- A = clear area of opening, ft^2
- d_i = depth at lip of curb opening, ft
- h = height of curb-opening orifice, ft = $TS_x + a/12$

Orifice Limitations:

$d > 1.4h$

- h = height of curb-opening inlet, in
- a = depth of depression, in

DEPRESSION CURB-OPENING INLET CAPACITY IN SUMP LOCATIONS
Figure 7-8J



Weir (without depression):

$$Q_i = C_w L d^{1.5}$$

Where:
- C_w = *2.3 curb-opening inlet*
  2.5 slotted inlet
- L = *length of curb opening, ft*
- d = *depth at curb measured from the normal cross slope, ft; i.e., $d = TS_x$*

Weir limitations:

$d \leq h$
h = *height of curb-opening inlet, in*

Orifice (both depressed and non-depressed curb-opening inlets):

$$Q_i = C_o h L (2gd_o)^{0.5} = C_o A[2g(d_i - \frac{h}{2})]^{0.5}$$

Where:
- C_o = *orifice coefficient*
  = *0.67*
- h = *height of curb-opening inlet, ft*
- d = d_o = *effective head on the center of the orifice throat, ft*
- A = *clear area of opening, ft²*
- d_i = *depth at lip of curb opening, ft*
- h = *height of curb-opening orifice, ft = $TS_x + a/12$*

Orifice Limitations:

$d > 1.4h$
h = *height of curb-opening inlet, in*

CURB-OPENING INLET CAPACITY IN SUMP LOCATIONS
(Non-depressed)
Figure 7-8K

Example 7-8.5

Given: Inlet dimensions: $L = 5$ ft, $h = 5$ in, $a = 2$ in

$W = 2$ ft
$S_x = 0.05$ ft/ft
$T = 8$ ft

Problem: Determine the interception capacity for the curb-opening inlet in sump conditions.

Solutions Depth of flow, $d = TS_x = 8\ (0.05) = 0.40$ ft; d is less than $(h + a)/12 = (5 + 2)/12 = 0.58$ (weir operation limitation).

From Figure 7-8J, enter the vertical scale, d = 0.40 ft; project horizontally to the perimeter, P [$P = L + 1.8W = 5 + 1.8\ (2) = 8.6$ ft]; read downward to the discharge scale, Q; and read $Q = 5\ ft^3/s$.

For an undepressed curb opening and the same conditions: Depth of flow, $d = TS_x = 0.40$ ft; d is less than h (weir operation limitation).

From Figure 7-8K, enter the vertical scale, d = 0.40 ft; project horizontally to the length, L = 5 ft; read downward to the discharge scale, Q; and read $Q = 3.8\ ft^3/s$.

Notice that the depressed curb-opening inlet has approximately 30% more capacity than the inlet without depression.

7-8.05.03 Slotted Inlets

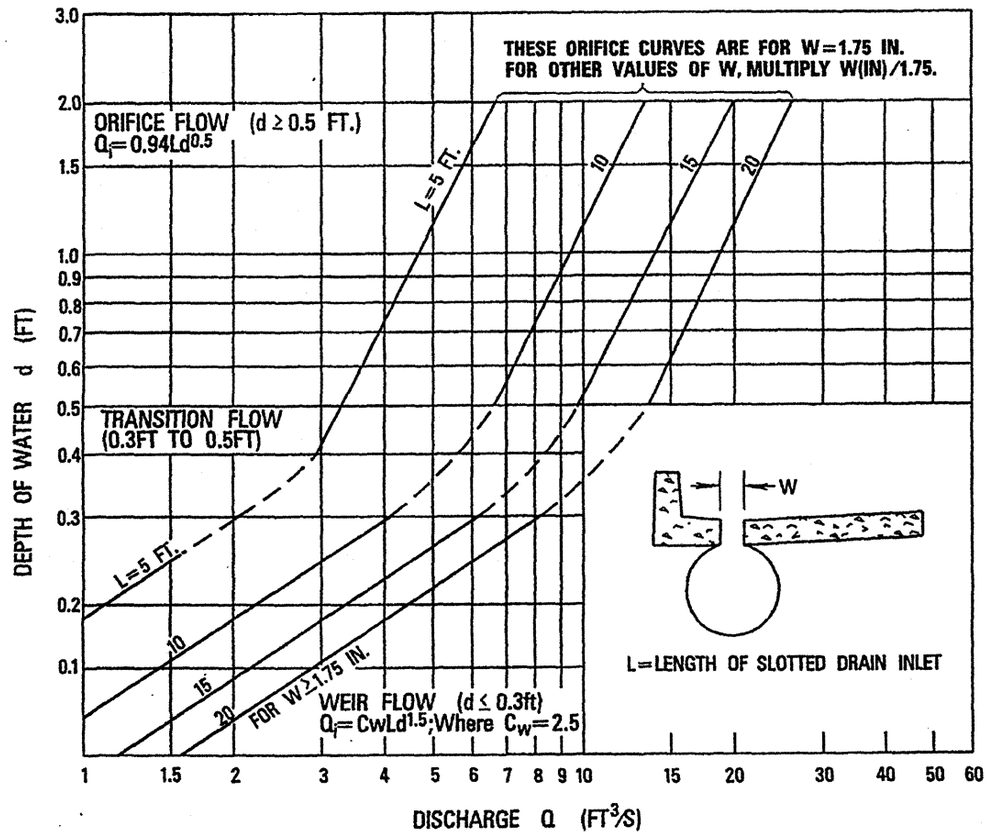
Figure 7-8L provides the slotted drain inlet capacity in sump locations. In sag locations, the inlet performs as a weir to depths of about 0.20 ft, and above 0.40-ft depth, the inlet performs as an orifice dependent on the slot width and length. The figure presents broken lines in the transition stage between weir and orifice performance. Example 7-8.6 illustrates the use of Figure 7-8L for the design of slotted inlets in sump conditions:

Example 7-8.6

Given: $Q = 5\ ft^3/s$
Maximum depth of water (d) = 0.30 ft

Problem: Determine the length of slotted inlet required for the given conditions assuming no clogging.

Solution: From Figure 7-8L, intersect discharge, $Q = 5\ ft^3/s$, vertically with depth of water, d = 0.30 ft, horizontally and read length, L = 15 ft. Normally, clogging should be considered. Similar to grate inlets, a margin approaching 50% should be allowed for clogging.



SLOTTED DRAIN INLET CAPACITY IN SUMP LOCATIONS
Figure 7-8L

7-8.06 Inlet Spacing

Inlet locations are established by the geometric features of the facility, the permissible spread of water on the pavement and the inlet intercept capacity. Computed inlet spacing requirements may be of considerable distances ranging possibly up to 600 ft. However, it is advised to keep the maximum distance to 300 ft with inlets often spaced at less than this distance. Where considerable over-the-curb flow occurs, inlets may be required at less than 100 ft apart.

Inlets in sag or sump conditions should have supplemental or flanking inlets placed within 50 ft to either side of the sag inlet. These flanking inlets may serve as relief or back-up inlets should the sag inlet become inoperative. This configuration should maintain a permissible spread of water on the pavement, especially where the water is trapped within the roadway with no over-the-curb relief possible.

In those areas where inlet usage is restricted to grate inlets and debris may not be readily passed, a clogging factor of at least 50% should be used in sag conditions and 25% used for inlets on grade. A liberal design of openings should be provided.

7-8.07 Storm Sewer Design Policies

The following policies should be maintained in the design of storm sewer systems:

1. Use a minimum size 18-in diameter concrete pipe for main lines and all laterals.

2. Refer to Table 7-8C for the minimum pipe slope required to maintain self-cleaning velocities. Progressive and/or equivalent velocities should be attained through the storm sewer design. Figure 7-8N provides the velocity in pipe conduits based on Q = VA.

3. Refer to Figure 7-8O for the pipe capacity on various gradients to determine the required pipe size.

4. At pipe or box size changes, strive to align the top inside surfaces (soffits) of the culvert, rather than the flow lines, forming a drop in the flow lines. This will ensure a more efficient performance of the storm sewer design.

5. Use the 10-year storm design frequency for roadway sections on grade and the 50-year storm for sags and zero grades.

6. Use a minimum 5-minute duration time while increasing the duration time as appropriate throughout the design.

7-8.08 Runoff Discharge Tabulation

Figure 7-8M provides the designer with sample worksheets necessary for the tabulation of the runoff discharge for storm sewer design. Documentation should begin in the uppermost segment of the storm sewer run.

Tabulate by station the amount of discharge accumulating along the sewer run. Work down the system gradient to the point of outfall release. Note that an outfall release may receive discharge from both back and ahead and from both sides of the roadway. A progressive time of concentration should be recorded for the run. This increasing time of concentration will reduce the amount of discharge from that determined previously for each individual contributing drainage area and should be considered for proper storm sewer design.

STORM SEWER INLET COMPUTATIONS

Inlet		D.A.	Q_a	Carry-over	Total Q_a	z	z/n	s	y	Ponded width (y)(z)	a	q_L	$L_a = \frac{Q_a}{q_L}$	L	L/L_a	a/y	Q/Q_a	$Q = Q_a \times Q/Q_a$	Carry-over	Remarks
No. 1	Sta. 2	No. 3	cfs 4	cfs 5	cfs 6	7	8	ft/ft 9	ft 10	ft 11	ft 12	cfs 13	ft 14	ft 15	16	17	18	cfs 19	cfs 20	21

STORM SEWER DISCHARGE COMPUTATIONS

From 1	To 2	Drainage Area No. 3	Total D.A. Ac. 4	Total C.A. 5	Length ft 6	Time of Concentration-Minutes: Along Sewer Line 7	Inlet Time 8	Used in Design 9	I In./Hr 10	Q cfs 11	Design: Dia. In. 12	Slope % 13	Cap. cfs 14	Vol. ft/sec 15	Remarks

STORM SEWER COMPUTATION FORMS
Figure 7-8M

7-8.09 Sizing Storm Sewer Lines

Once the runoff discharge has been tabulated for each segment of the storm sewer run, refer to Table 7-8C and Figure 7-8N and O to determine the pipe size necessary for the allowable gradients.

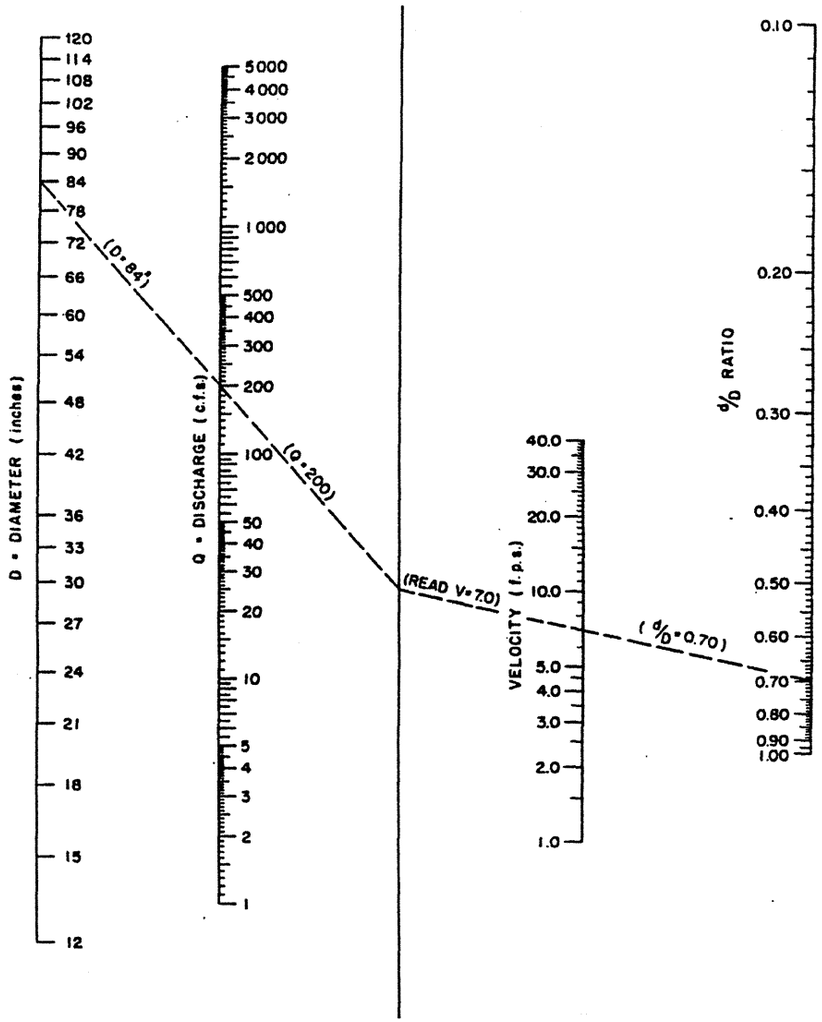
Storm sewer line gradients should be similar to the roadway grade. The same size of pipe will run until the cumulative discharge attains the pipe capacity. When an abrupt reduction in gradient is encountered, an increase of one or more pipe sizes larger may be required. Never reduce the pipe size even with an abrupt increase in gradient. Attempt to align the inside top (soffit) of pipes when increasing the size. The resulting drop in the flow line will improve the performance of the system. When, for various reasons, gradients must be minimized, the flow lines of concrete arch pipe consecutively increased in size may be aligned more effectively than round pipe. The designer should proceed in this manner along the entire system, sizing each individual section, including the outfall pipe.

A hydraulic gradient is the line of elevations to which the water would rise in successive piezometer tubes along a storm sewer run. Differences in elevations for the water surfaces in the successive tubes represent the friction loss for that segment of storm sewer. The friction slope is the slope of the line between the water surfaces. The storm sewer run will not operate under pressure if it is placed on a calculated friction slope corresponding to a certain quantity of water, cross-section and roughness factor. In this case, the surface of flow (hydraulic gradient) will be parallel to the top of pipe. This is the desirable condition.

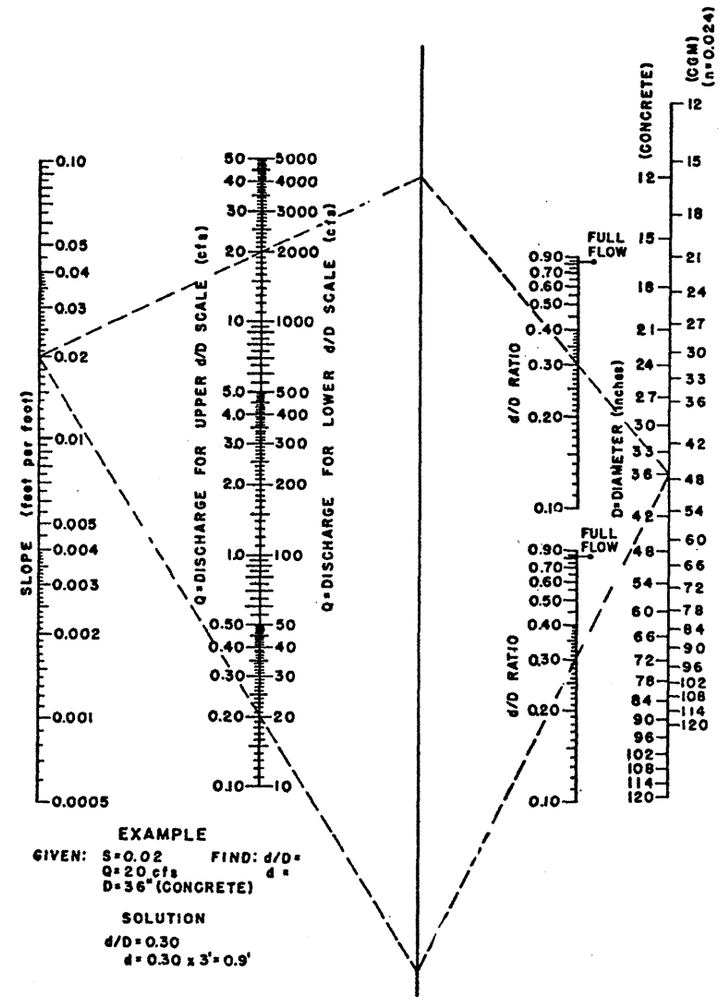
There may be reason to place the storm sewer run on a slope less than the friction slope. In that case, the hydraulic gradient would be steeper than the slope of the storm sewer run. Depending on the elevation of the hydraulic gradient at the downstream end of the run, it is possible to have the hydraulic gradient rise above the top of the pipe, thus, creating pressure on the storm sewer system until the hydraulic gradient at some point upstream is once again at or below the top of the pipe.

Table 7-8C
MINIMUM SLOPES REQUIRED FOR SEWERS TO MAINTAIN SELF-CLEANING VELOCITY OF 2.0 ft/s
(Slope in ft/ft)
(n values for Manning's formula)

INSIDE PIPE DIA.	25% FULL			50% FULL			75% FULL			FULL		
	DISCH. ft^3/s	SLOPE		DISCH. ft^3/s	SLOPE		DISCH. ft^3/s	SLOPE		DISCH. ft^3/s	SLOPE	
		n=0.013	n=0.024		n=0.013	n=0.024		n=0.013	n=0.024		n=0.013	n=0.024
6"	.077	.0100	.0340	.20	.0049	.0170	.32	.0039	.0130	.40	.0049	.0170
8"	.170	.0058	.0200	.44	.0029	.0096	.71	.0023	.0076	.88	.0029	.0096
10"	.210	.0051	.0180	.54	.0025	.0085	.88	.0020	.0066	1.08	.0025	.0085
12"	.310	.0040	.0140	.79	.0020	.0067	1.26	.0016	.0052	1.58	.0020	.0067
15"	.480	.0030	.0100	1.22	.0015	.0050	1.98	.0012	.0039	2.44	.0015	.0050
18"	.690	.0024	.0089	1.77	.0012	.0039	2.85	.0009	.0030	3.54	.0012	.0039
21"	.940	.0019	.0064	2.40	.0010	.0032	3.88	.0008	.0025	4.80	.0010	.0032
24"	1.230	.0016	.0054	3.15	.0008	.0027	5.04	.0006	.0021	6.30	.0008	.0027
30"	1.920	.0012	.0040	4.91	.0006	.0020	7.90	.0005	.0016	9.82	.0006	.0020
36"	2.760	.0010	.0032	7.08	.0005	.0016	11.40	.0004	.0012	14.20	.0005	.0016
42"	3.760	.0008	.0026	9.60	.0004	.0013	15.50	.0003	.0010	19.20	.0004	.0013
48"	4.910	.0007	.0022	12.60	.0004	.0011	20.30	.0003	.0008	25.20	.0004	.0011
54"	6.200	.0006	.0018	15.90	.0003	.0009	25.60	.0002	.0007	31.80	.0003	.0009
60"	7.680	.0005	.0016	19.60	.0003	.0008	31.60	.0002	.0006	39.20	.0003	.0008
72"	11.200	.0004	.0013	28.30	.0003	.0007	45.50	.0002	.0005	56.60	.0003	.0007
84"	15.200	.0003	.0010	38.50	.0002	.0006	62.00	.0002	.0004	77.00	.0002	.0006
96"	19.800	.0003	.0009	50.20	.0002	.0005	81.00	.0001	.0004	100.00	.0002	.0005
108"	25.100	.0003	.0008	63.50	.0002	.0004	102.00	.0001	.0003	127.00	.0002	.0004
120"	30.700	.0002	.0007	78.60	.0001	.0004	126.00	.0001	.0003	157.00	.0001	.0004



VELOCITY IN PIPE CONDUITS
(Based on Q = VA)
Figure 7-8N



Full Flow ≈ 0.83 d/D ratio

UNIFORM FLOW
FOR
PIPE CULVERTS
Figure 7-8O

Computation of the hydraulic grade line of a storm sewer run will not be necessary where:

1. the slope of the pipe sizes are chosen so that the slope is equal to or greater than the friction slope;

2. the top surfaces of successive pipes are aligned at changes in size (rather than flow lines being aligned); and

3. the surface of the water at the point of discharge does not rise above the top of the outlet.

The pipe will not operate under pressure in these cases and the slope of the water surface under capacity discharge will approximately parallel the slope of the pipe invert. Small head losses at inlets, manholes, etc., have a cumulative influence in effective function of the system and must be considered accordingly.

However, in cases where different sized pipe inverts are placed on the same grade causing the smaller pipe to discharge against a head, or when it is desired to check the storm sewer system against larger-than-design floods, the designer should compute the hydraulic grade line of the entire storm sewer system.

Begin with the tailwater elevation at the storm sewer outfall and progress upward along the length of the storm sewer run. For each segment, compute the friction loss and plot the elevation of the total head at each manhole and inlet. If the hydraulic grade line does not rise above the top of any manhole or above an inlet entrance, the storm sewer system is satisfactory. If it rises above these points, water blow-outs will occur through manholes and inlets. Pipe sizes or gradients can be increased as necessary to eliminate such blow-outs. A hydraulic gradient should have an original base elevation above the outlet tailwater elevation. Any backwater effects of a significant tailwater elevation should be carefully considered. Refer to FHWA hydraulic design publications for the detailed procedure of hydraulic grade line computation.

7-8.10 Subsurface Drainage (Storm Sewer)

Generally, the top one-third of the storm sewer pipe joints in the system are not sealed in order to permit subsurface drainage to enter joints within the limits of the storm sewer system. Standard Drawing UD-1 illustrates a detail to be included with the typical section sheets indicating the trench and pipe arrangement, the porous material surrounding the pipe, and the portion of the pipe joints to remain unsealed with the limits of filter cloth over the open joints. These partially not sealed pipe joints should not be subjected to pressure head flow.

Perforated pipe may extend to areas beyond the storm sewer lines. The connection of the subdrain to the junction should be placed well above the flow line of the junction box; otherwise, when a head accumulates in the junction box, the subdrain will empty underneath the structure. Perforated pipe should not be subject to a pressure head.

7-8.11 Junction Boxes and Manholes

Often, a junction box will be necessary to connect behind-the-curb interception points with the main line at locations not coincident with curb inlets. These connections may require flow line adjustments to the main line system.

Generally, manholes are not required in a storm sewer system due to the common practice of using a curb inlet with a manhole cover for servicing purposes. However, a manhole may be specified for an abrupt change in the direction of the pipes or when encountering utility lines. A maximum distance of 300 ft is recommended between manholes or other service entrances. Manholes can be either poured-in-place or precast structures.

7-9.0 SUBSURFACE DRAINAGE

7-9.01 Application

The purpose of subsurface drainage is to remove detrimental ground water to ensure the stability of the roadbed and side slopes. Most often, areas that require specific subsurface drainage will be determined during the process of field investigations.

However, there are times when water-bearing formations are not revealed until construction has begun. "Bleeding" backslopes and apparent spring disruptions are two of the more obvious indications of this excessive groundwater. Known spring and well points should be identified on the project plans. Standard Drawing UD-1 illustrates the details of normal underdrains Type I and Type II to be installed at spring sites and to be included in the plans when necessary.

Perforated pipe underdrains, with perforations placed downward, will be installed to intercept most subsurface drainage waters. These pipes, available in sizes of 6-in, 8-in and larger diameters, may form a network of interceptors and be routed to a central point of collection and outfall. Ordinarily, a selected permeable backfill material will surround the underdrain pipe.

Subsurface drainage is of primary importance in curb and gutter sections of the roadway. Normally, the top one-third of the storm sewer pipe joints in the system will remain unsealed (with filter cloth) to receive subsurface drainage. Along the limits of curb and gutter sections where storm sewer pipe is not required, perforated underdrain pipe will be installed to empty into the junction box connecting the storm sewer pipe. Connection into the structure should be well above the flow line and as high as practical above the storm sewer pipe. This will eliminate the possibility of perforated pipe emptying against a head developed in the junction box and forcing the water out of the perforations and under the structure.

7-9.02 Underdrains

Standard Drawing UD-1 provides typical underdrain installations for a normal underdrain and a typical installation where a storm drain is used as an underdrain. Surface drainage must not be allowed to discharge into an underdrain. Only when the underdrain invert is well above the head developed at that point in the storm sewer system should underdrains be permitted to empty into a roadway drainage system.

Underdrains of 6-in diameter should be allowed to run no more than 500 ft. For distance over 500 ft, the minimum diameter should be 8 in. A minimum of 0.20% pipe gradient is recommended. Cleanouts should be installed on long runs of underdrain pipe. This may consist of a vertical small-diameter steel water pipe through which a pressure hose can be connected to flush out the system. The vertical pipe should be capped. Outlets for underdrains should be spaced no greater than 1000 ft.

The soil test should be analyzed to determine the most appropriate alternative for using either metal or non-metal underdrain pipe at a specific location.

When underdrain lines departs from the area requisite of underdrains and as underdrains approach the outlet of the surface exit onto the side slope, the perforated pipe is changed to non-perforated pipe appropriate to the installation. The length of non-perforated pipe should be sufficient to provide the minimum pipe cover requirements. These outlet connections are generally into paved ditches to avoid re-saturation.

7-10.0 OPEN CHANNELS

The main reasons for the design and construction of open channels are to:

1. permit better highway alignment,

2. economize by eliminating culverts or bridges where a stream crosses the highway more than once in close proximity,

3. Improve flow conditions in channels at bridges and culverts, and

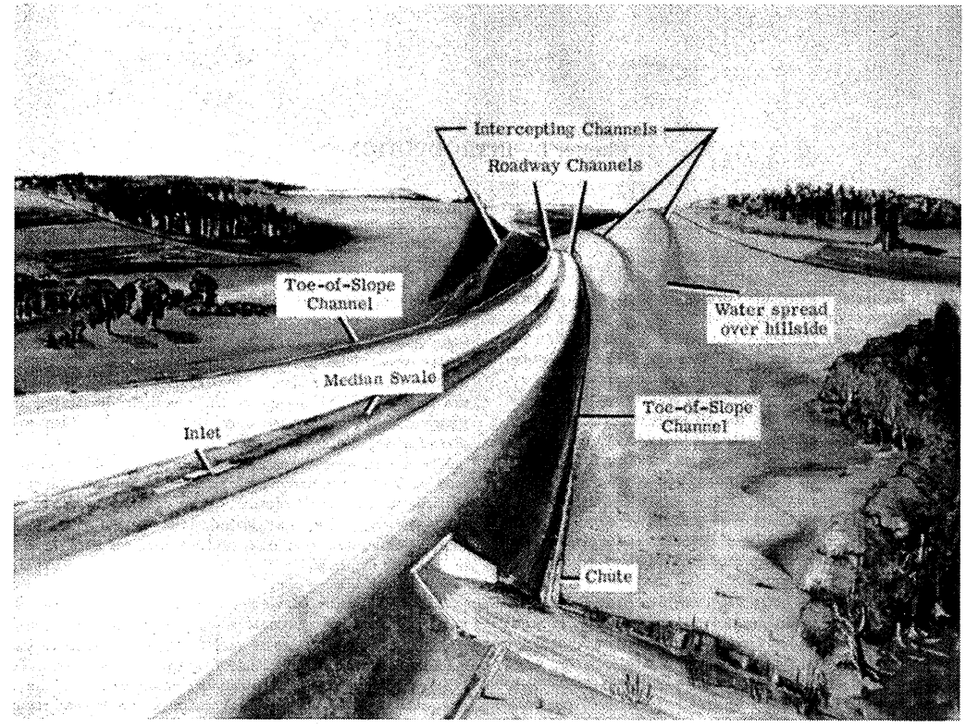
4. protect the highway against flood damage.

Channel changes in the vicinity of bridges will be designed by the Bridge Division. Most other open channels will be designed by the Roadway Design Division. Types of roadside drainage channels are illustrated in Figure 7-10A.

7-10.01 Size and Shape

Trapezoidal channels are usually the most economical type of open channel design. For larger flows, consideration should be given to a channel bottom width of 10 ft and more; however, wide channels less than 1-ft deep are inefficient and are not recommended. Also, hand-shaped 4-ft and 6-ft bottom ditches may be less economical than blade cut 8-ft bottom ditches. When only moderate capacity is required, a V-shaped channel may be adequate.

Under average conditions, side slopes ranging from 2H:1V to 3H:1V are satisfactory. To facilitate maintenance, side slopes for ditches in interchange areas should be no steeper than 4H:1V.



TYPES OF ROADSIDE DRAINAGE CHANNELS
Figure 7-10A

7-10.02 Capacity Computations

Open channels should be designed to safely accommodate the design discharge considering the channel slope and velocity. For a particular location, the design discharge will be derived (see Section 7-4.0) and the basic formula is:

$$Q = VA$$

Where:

Q = Discharge (cubic feet per second — ft^3/s)
V = Mean velocity (feet per second — ft/s)
A = Cross-sectional area of water (square feet — ft^2)

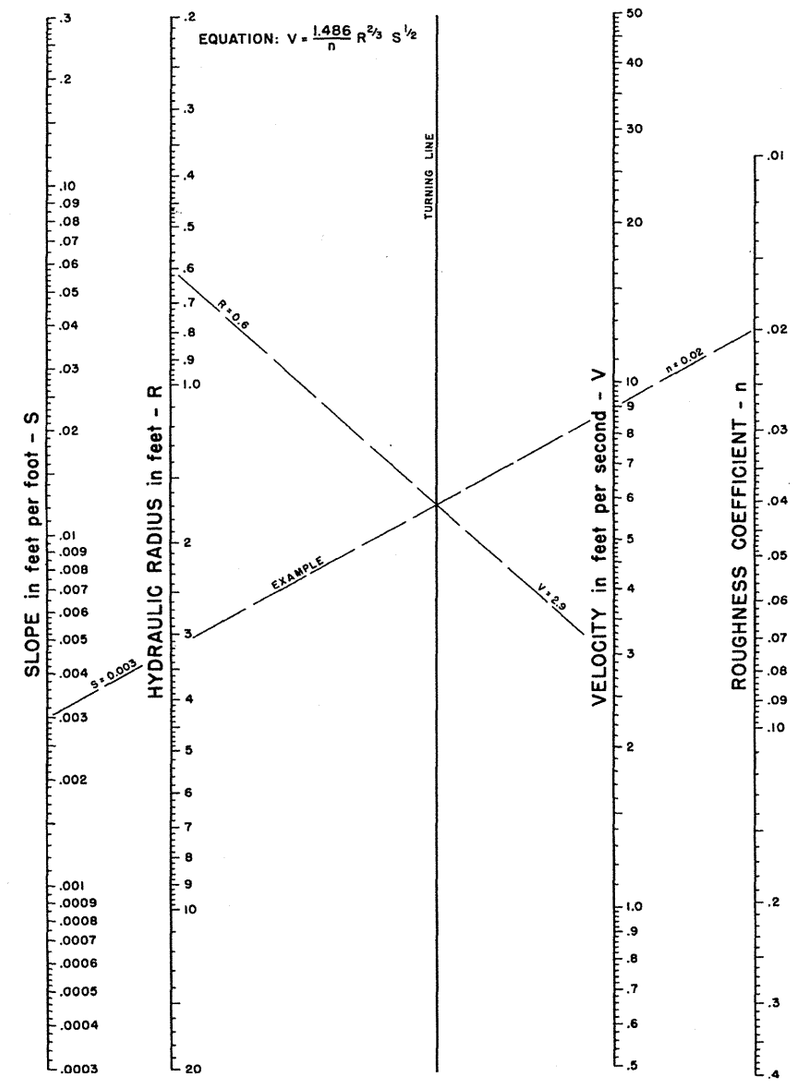
Assuming uniform flow, velocities can be computed by the Manning Equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where:

V = Mean velocity (ft/s)
n = Manning coefficient of channel roughness
R = Hydraulic radius (ft)
  R = A/WP
  A = cross-sectional area of water (ft^2)
  WP = wetted perimeter of channel (ft)
S = Channel slope (ft/ft)

By trial and error, various channel bottom widths and depths of flow are assigned, and the respective cross-sectional area of water (A) and wetted perimeter of channel (WP) are determined to derive the (VA) value equal to the design discharge (Q). Channel roughness coefficients (n) are provided in Table 7-6B. Figure 7-10B provides a nomograph for the solution of the Manning Equation. The FHWA Hydraulic Design Series No. 3 *Design Charts for Open-Channel Flow* provides a direct solution of the Manning Equation for various channel slopes and cross sections.



NOMOGRAPH FOR SOLUTION OF MANNING EQUATION
Figure 7-10B

7-10.03 Channel Alignment and Gradient

Changes in channel alignment should be as gradual as practical. The radius of curvature of the centerline of a channel should be at least three times the bottom width of a trapezoidal channel or three times the width of a rectangular channel. The gradient of an open channel is often naturally established. Generally, channels with gradients less than 0.12% should be paved to prevent standing water. A minimum gradient of 0.25% is desirable for unlined channels.

7-10.04 Channel Lining

The requirements for channel lining should be determined by the velocity of flow. Table 7-10A provides the recommended maximum permissible velocities for unlined channels. Depending on the type of channel bed, linings may be necessary when permissible velocities are exceeded. The types of channel lining to be considered include:

1. ditch liner (excelsior blanket, jute mesh or erosion control fabric);
2. bituminous treated roving;
3. soil reinforcing mat;
4. solid sod;
5. riprap;
6. bituminous paved ditch; and
7. portland cement concrete.

Ditch liners, bituminous treated roving, and soil reinforcing mat help protect the channel until vegetation can be established. The remaining linings are for permanent protection. The application of channel lining is discussed in detail in Chapter 8 under the topic "Special Side Ditch Treatment" and in HEC No. 15, *Design of Roadside Channels with Flexible Linings.*

Table 7-10A
RECOMMENDED PERMISSIBLE VELOCITIES FOR UNLINED CHANNELS

Type of Material in Excavation Section	Permissible Velocity (ft/s)	
	Intermittent Flow	Sustained Flow
Fine Sand (Noncolloidal)	2.5	2.5
Sandy Loam (Noncolloidal)	2.5	2.5
Silt Loam (Noncolloidal)	3.0	3.0
Fine Loam	3.5	3.5
Volcanic Ash	4.0	3.5
Fine Gravel	5.0	4.0
Stiff Clay (Colloidal)	6.0	4.5
Graded Material (Noncolloidal)		
Loam to Gravel	6.5	5.0
Silt to Gravel	7.0	5.5
Gravel	7.5	6.0
Coarse Gravel	8.0	6.5
Gravel to Cobbles (under 6 in)	9.0	7.0
Gravel and Cobbles (over 8 in)	10.0	8.0

7-10.05 Energy Dissipators

Open channels and culverts may significantly increase the erosion of the natural channel bed at the outlet. This is particularly true of culverts. Erosion and scour will occur if the outlet conditions produce excessive flow velocities. In these cases, either altering the design of the culvert or providing an energy dissipator should be evaluated. The FHWA Hydraulic Engineering Circular No. 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, should be used as a reference. If needed, the type of energy dissipator will depend on the following:

1. discharge volume and velocity;
2. culvert size, slope, roughness and outlet geometry;
3. natural channel material;
4. tailwater depth; and
5. Froude number (Fr) range.

The highest outlet velocities will be produced by long, smooth-barrel culverts on steep slopes. These culverts are the most likely candidates for energy dissipators. However, protection may also be necessary for culverts on mild slopes. Tailwater (TW) is also a major factor. High TW will reduce the outlet velocity and, therefore, the erosion potential. In addition, the erodibility of the natural stream material at the outlet will influence the amount of scour.

The Froude number is:

$$Fr = \frac{V}{(gd_m)^{1/2}} = \frac{V}{V_c}$$

Where:

d_m	=	mean depth of flow, ft
g	=	32.2 ft/s^2
V	=	mean velocity, ft/s
V_c	=	critical velocity, ft/s

The Froude number represents subcritical (Fr < 1), critical (Fr = 1) or supercritical (Fr > 1) flow. In the selection of energy dissipators, (Fr) will indicate which type is best for the hydraulic conditions.

7-10.05.01 Applicability

The designer should estimate the potential erosion or scour at the culvert outlet to determine the need for culvert modifications or an energy dissipator. If an existing culvert is being re-evaluated, a field inspection of the outlet should be made to examine the existing erosion. The soils information, maintenance history, culvert characteristics and discharge history (if available) should be reviewed.

The designer should estimate the potential scour of new and existing culverts by the procedures in HEC #14. The method provides an estimate of the depth, width and length of scour based on the tailwater depth, culvert dimensions and a time variable. The procedure is inexact and meant to serve as only an approximate means to estimate scour.

7-10.05.02 Culvert Modifications (New or Existing)

If the designer concludes that scour will occur, he should first consider altering the culvert characteristics to correct the problem. Possibilities include:

1. Increase the culvert size to reduce outlet velocity. This may be effective on mild slopes, but it is generally ineffective on steep slopes.

2. Consider multiple-pipe installations.

3. Consider corrugated ferrous metal pipe or other pipe appurtenances with a higher "n" value to reduce the flow velocity.

7-10.05.03 Types of Energy Dissipators

If it is not practical to modify the culvert, or if the design options of a proposed culvert are limited, an energy dissipator should be used at the outlet or within the culvert barrel. Many general types exist:

1. Hydraulic jump. Natural stilling basins at the culvert outlet may be used to convert the kinetic energy of supercritical flow into the potential energy of subcritical flow. This will reduce the flow velocity and increase the depth of flow.

2. Forced hydraulic jump. Several roughness elements (blocks, sills, etc.) may be used to force the water flow from supercritical to subcritical. They may be used inside a culvert (inlet control only), at the culvert outlet or in an open channel (for slopes up to 15%).

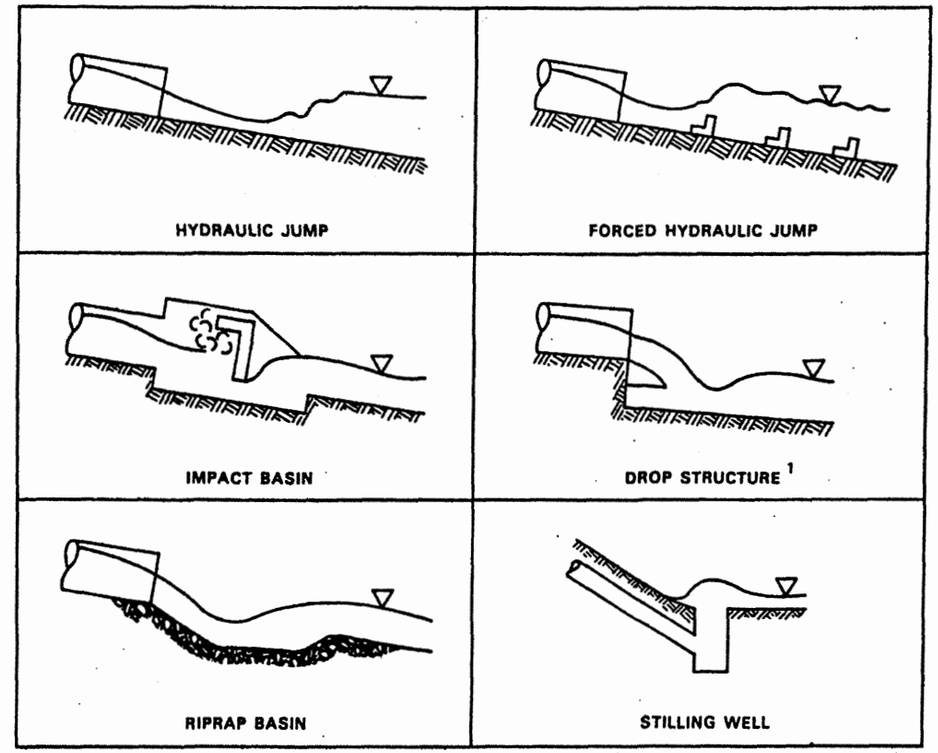
3. Impact basins. Vertical baffles are used at the culvert outlet to dissipate the kinetic energy. Various designs are applicable to a wide range of culvert types and sizes.

4. Drop structures. A steep slope for culverts or open channels may be converted to a series of gentle slopes with intermittent vertical drops into a stilling basin. This arrangement will prevent the erosive high velocities from developing.

5. Riprap basins. Large angular and rounded stones are used at the culvert outlet to dissipate the kinetic energy of the flow. A filter blanket should be used as a foundation for the stone.

6. Stilling wells. These dissipate the kinetic energy by forcing the flow to travel upward to reach the downstream channel.

Figure 7-10C illustrates these energy dissipators.



[1] A drop structure would typically be a series of these arrangements.

ENERGY DISSIPATORS
Figure 7-10C

7-10.05.04 Design Procedure

The following procedure should be used to select and design an energy dissipator. HEC #14 describes the analytical details:

1. Determine the pertinent data for the outfall (discharge volume and velocity; culvert size, slope and roughness; outlet conditions).

2. Determine the potential for scour at the outfall.

3. Evaluate the practicality of modifying the existing culvert to eliminate the potential scour. See Section 7-10.05.02.

4. If modification is impractical, evaluate several types of energy dissipators considering costs, limitations and effectiveness of each type.

5. Select the dissipator which is most economical and adaptable to the site.

6. Conduct the detailed design of the dissipator.

7-11.0 PHYSICAL STANDARDS

This section discusses the physical standards for pipe culverts and the criteria for installation. The Bridge Division is responsible for the physical requirements of concrete box culverts.

7-11.01 Alternative Types of Pipe Culverts

Pipe culverts are manufactured in various sizes and shapes from several types of material. Types of pipe available for use in Mississippi include:

1. Reinforced Concrete Pipe,
2. Reinforced Concrete Arch Pipe,
3. Corrugated (Ferrous) Metal Pipe,
4. Corrugated (Ferrous) Metal Arch Pipe,
5. Corrugated Aluminum Pipe (Non-Ferrous), and
6. Polyvinyl Chloride (PVC) and High Density Polyethylene (HDPE) Plastic Pipe.

The required type and necessary properties of pipe culverts should be determined by project site soils testing. The Department's policy on type of pipe culverts based on functional class and type of installation are presented in Table 7-5C. Design life, materials and alternatives for various conditions are provided in the table. An exception to pipe alternatives on Federally funded projects is where the specific culvert is a non-participating item and the type of pipe may then be specified.

Where alternatives for cross drains exist, consider RCP up to a 36-in diameter. When pipe is 36-in diameter or larger, alternatives of either RCP or 3-in x 1-in or 5-in x 1-in corrugated ferrous metal pipe are more comparable.

Lower headwater values can be achieved with arch pipe. Normally, arch pipe is used where the cover, outfall elevation or gradient is restrictive.

Corrugated ferrous metal and corrugated aluminum pipes should be specified in accordance with Table 7-5C. Aluminum pipe is rarely utilized.

PVC and HDPE plastic round pipe are also specified in accordance with Table 7-5C. These pipes may have limited applications used as ramp pipes and other side drain installations. The n Value is similar to RCP.

7-11.02 Types of Pipe Culverts (State Aid Division)

For State Aid Division projects having no Federal funds, the type of pipe culvert is selected based upon engineering judgment and is specified without alternatives. Whenever Federal funds are involved, alternatives are required according to Section 7-11.01.

7-11.03 Strength Identification

The strength of reinforced concrete pipe is indicated by class designations. Although other classes of concrete pipe are manufactured, the following three classes are most frequently designated:

1. Class III — Standard Strength Reinforced Concrete Culvert Pipe,
2. Class IV — Extra Strength Reinforced Concrete Culvert Pipe, and
3. Class V — Super Strength Reinforced Concrete Culvert Pipe.

Concrete arch pipe is identified as Class AII, Class AIII (Standard Strength) and Class AIV (Extra Strength). However, Class AIII is generally used and is understood, unless otherwise designated.

The strength of corrugated ferrous metal and corrugated aluminum pipe is indicated by the gage (thickness of metal). Corrugated ferrous metal pipe is available in different sizes of corrugation including 2⅔-in x ½-in, 3-in x 1-in and 5-in x 1-in. The first number indicates the distance between crests and the second number provides the depth of corrugations. With the same gage metal, the larger corrugation produces greater strength than the smaller corrugation. At present, the 3-in x 1-in and 5-in x 1-in corrugations are used respectively for 36-in and 48-in diameter pipe and larger.

7-11.04 Pipe Culvert Strength Requirements

Information needed by designers to designate the strength requirements for individual culverts is provided in the *Roadway Design Standard Drawings*, PI-1. The requirements are based principally on the size and shape of the culverts, the amount of cover over the pipe and, in some instances, the class of bedding and type of backfill.

The maximum cover value is based on the vertical distance from the top of the culvert to the elevation of the highest point of the finished road surface. Minimum cover is measured from the top of the culvert to the top of the subgrade. This will be the effective cover during construction operations.

In addition to culvert size and length, the following data should be provided on the plans to identify strength requirements:

1. Reinforced Concrete Pipe Culverts. Class III is understood unless otherwise indicated. Indicate class of pipe IV or V.

2. Concrete Arch Pipe Culverts. Class AIII is understood. No special identification is required except for special conditions.

3. Corrugated Ferrous Metal Pipe and Arch Pipe Culverts. Provide the gage of metal and the size of corrugations.

4. Corrugated Aluminum Pipe Culvert. Provide the gage of metal.

5. Polyvinyl Chloride (PVC) and High Density Polyethylene (HDPE) Plastic Pipe. See requirements in *MDOT Pipe Culvert Material Design Criteria.*

7-11.05 Special Provision Against Deterioration

In all installations, protective coatings are required for corrugated ferrous metal and aluminum culverts to retard deterioration. Other special considerations may be recommended with the field survey or as a result of soil testing. The designer should include soil test data in the project file and set appropriate culvert alternatives accordingly.

For the various protective coatings, the description of the culvert should be preceded by the phrase:

1. Zinc Coated,
2. Aluminum Coated,
3. Polymeric Coated, or
4. Bituminous Coated.

In specific cases, each of these coatings may require paved-inverts in which these descriptions are followed by the term "Paved-Invert" (e.g., Polymeric Coated Paved-Invert).

7-11.06 Bedding and Backfill

The *Standard Specifications* provide for the following three types of culvert bedding:

1. Class C is the standard bedding of natural soil shaped to 10% conduit height. Class C bedding satisfies most installation conditions.

2. Class B is a placed bedding of selected material shaped to 15% conduit height. It permits greater heights of cover or replaces otherwise unsuitable bedding conditions.

3. Class A consists of a continuous concrete cradle under the culvert, constructed on an approved foundation, but used only in unique conditions.

Class C bedding is assumed if no special note is provided on the plans. If either Class A or Class B bedding is required, the type of bedding should be indicated on the plans. The standard backfilling procedure defined in the *Standard Specifications* will normally be used, and no special notation is required on the plans.

In the case of very high fills as identified in Standard Drawing PI-1, it may be necessary to specify the Induced Trench Method of backfilling (previously known as the Imperfect Trench Method). This requirement must be indicated on the plans. A detail of the Induced Trench Method is provided in Standard Drawing PI-1 indicating the different pay limits. The actual limits of the induced trench is a separate pay limit from that of the remaining normal backfill limits. No less than Class IV Reinforced Concrete Pipe shall be permitted for induced trench installations. Both the class of pipe and bedding should be consistent throughout the pipe length.

7-11.07 Pipe Culvert End Sections

Four alternatives should be considered for the treatment of culvert ends at each installation. The alternatives are described below and criteria provided to guide in the selection of the appropriate end treatment.

1. Concrete Headwalls. Design details for standard cast-in-place concrete headwalls are provided in the *Roadway Design Standard Drawings* (i.e., those with Work Numbers HW and HWA). Alternatives are included for various combinations of size of culvert, embankment slope, degree of skew and number of pipes (single or multiple-line installation). Used less frequently, concrete headwalls could be specified:

a. for multiple lines of pipe in deep channelized ditch sections;

b. for structures sufficiently removed from the travelway where headwalls are more economical than other types of end sections; and

c. when right-of-way restrictions require special design abbreviated headwalls or straight headwalls parallel to the roadway. (Guardrail is usually required with this installation.)

2. Improved Entrance Pipe Sections. Design details for standard prefabricated concrete improved entrance pipe sections are provided in the *Standard Drawings*. Improved entrance structures can increase pipe efficiency and may reduce the required pipe size. This pipe end section should be installed whenever practical, with due consideration to exceptions for headwall use. Recommendations for installation of an improved entrance section are to align the culvert with the existing channel (regardless of skew), use the improved entrance section upstream, and provide a flared end section downstream. Although special cases will arise, skew angles greater than 45° should be avoided.

3. Flared End Sections. Design details for standard prefabricated concrete and metal flared end sections are provided in the *Standard Drawings*, Working Numbers FE-1, FE-1A and FE-1B. Flared end sections should be specified for most pipe installations except conditions covered under Headwalls and Improved Entrance structures. A maximum 45° skew angle should be maintained. For installations where the improved entrance does not significantly enhance culvert performance, flared end sections should be used. In general, only smaller drainage areas and moderate pipe sizes will be included in this category.

4. No End Structure. In most cases, culverts under rural driveway ramps do not require an end structure. The regular terminal section of culvert pipe should extend to the normal fill slope and natural ground intersection. This detail is provided on Standard Drawing RD-1. Designers should consider slope protection in the form of riprap or slope paving at the ends of culverts with no end structures. Place the groove or bell end of the pipe to the upstream for better hydraulic efficiency.

7-11.08 Multiple Pipes

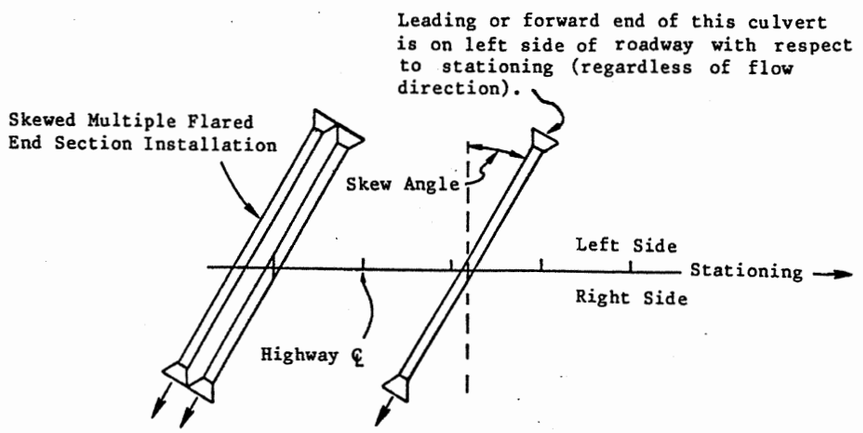
Where multiple pipes will be installed and concrete headwalls are specified, the proper spacing between adjacent pipes is defined in the *Standard Drawings* for headwalls.

When flared end sections are specified, the spacing between multiple pipes should be set to fix the maximum flare of the end sections in contact with each other provided the minimum required space between pipes is maintained. Concrete grouting and slope paving at these end section connections may be necessary and should be considered. Appropriate dimensions are provided in the *Standard Drawings* for flared end sections. Multiple flared end sections on skewed pipes should be on a straight alignment normal to the pipes, just as a non-skewed installation. Figure 7-11A provides a detail of a skewed pipe installation.

7-11.09 Skewed Installations

Generally, pipe culvert installations should be designed to conform as closely as possible to natural drainage channels. A "standard skew" headwall (15°, 30° and 45°) usually can be installed satisfactorily, even though the degree of skew may be slightly different. The amount of skew should be indicated on the plans.

The degree of skew is measured as the angle between the pipe installation and a line perpendicular to the highway centerline. Figure 7-11A provides detailed information for skewed pipe installations. A culvert skew angle is described in terms of which end is forward, left forward or right forward. For example, if the skew angle in the figure is 30°, the proper description of the skew (to be provided on the plans) is "SKEW 30° LEFT FORWARD," which may take the abbreviated from "SK. 30° LT. FWD.".



SKEWED PIPE INSTALLATIONS
Figure 7-11A

7-11.10 Box Culverts

The *Bridge Division Standard Drawings* include design details and guides for estimating quantities for box culvert installations. Designers normally can make direct reference on the plans to the drawing number, size of box, skew, headwalls and other pertinent information. Estimated quantities should be computed from the guides and recorded on the plans.

The Bridge Division should provide supplementary drawings for any non-standard conditions including size, skew, headwalls, improved entrance structures, etc.

7-11.11 Culvert Length

The required length of pipe culverts normally should be determined by plotting the installation to scale on a cross section at the location and measuring the plotted lengths. For skewed installations, a plan view also should be drawn to scale. When improved entrance and flared end sections are specified, they should be identified separately and considered in the reduction of overall culvert length.

The lengths of pipe culverts should be estimated and recorded to the nearest multiple of 4-ft increments. The length of box culverts should be calculated to the nearest 0.1 ft. Detailed information on dimensions and quantities for box culverts should be documented on computation sheets approved by the Department.

7-11.12 Structure Excavation

Detailed procedures and tables for structure excavation are found in the Department publications, "Structure Excavation Tables for Pipe Culverts, Headwalls, Box Culverts and Minor Structures." Tables may be used for State Aid work. The structure excavation widths are 1 ft on either side plus the inside pipe diameter (2 ft + inside pipe diameter) and 1 ft on either side plus the outside box culvert width (2 ft + outside box width).

7-12.0 REFERENCES

1. "Floodplain Management Regulations for the State of Mississippi," House Bill No. 8, Mississippi Legislature, Adopted August 1, 1979, Amended June 10, 1982.

2. "Procedures for Coordinating Highway Encroachments on Floodplains with FEMA," FHWA, June 7, 1982.

3. *Highway Drainage Guidelines*, AASHTO, 1992.

4. Federal-Aid Policy Guide (FAPG) Transmittals 1 and 12, Title 23, Highways, Code of Federal Regulations (CFR), Part 650, Subpart A, (23 CFR 650A), December 9, 1991.

5. *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12 (HEC #12), FHWA, March 1984.

6. Technical Paper No. 40, "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," U.S. Dept. of Commerce, Weather Bureau, May 1961.

7. NOAA Technical Memorandum NWS HYDRO-35, "Five-Minute to 60-Minute Precipitation Frequency for the Eastern and Central United States," June 1977.

8. "Flood Characteristics of Mississippi Streams," U.S.G.S., 1991, (Rural).

9. "Flood Characteristics of Urban Watersheds in the United States," U.S. Geological Survey Water-Supply Paper 2207, 1983.

10. "A Preliminary Study of the Effect of Urbanization on Floods in Jackson, MS," U.S. Geological Survey Research, 1967.

11. "A Rainfall-Runoff Modeling Procedure for Improving Estimates of T-year (Annual) Floods for Small Drainage Basins," U.S. Geological Survey, Water-Resources Investigations 78-7.

12. *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (HDS #5), FHWA, September 1985.

13. *Design Charts for Open-Channel Flow*, Hydraulic Design Series No. 3 (HDS #3), FHWA, August 1961, Reprint December 1980.

14. *Concrete Pipe Design Manual*, American Concrete Pipe Association, March 1990.

15. *Roadway Design Standard Drawings*, Governor's Highway Safety Program, Mississippi State Highway Department, FHWA, Issued November 1, 1979, Revisions to Date.

16. *Hydraulic Characteristics of Two Bicycle-Safe Grate Inlet Designs*, FHWA-RD-74-77, November 1974.

17. *Bicycle-Safe Grate Inlet Study*, FHWA-RD-78-4, May 1978.

18. *Design of Stable Channels with Flexible Linings,* Hydraulic Engineering Circular No. 15 (HEC #15), FHWA, April 1988.

19. *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular No. 14 (HEC #14), FHWA, November 1975.

20. *The Design of Encroachments on Flood Plains Using Risk Analysis*, Hydraulic Engineering Circular No. 17 (HEC #17), FHWA, April 1981.

21. *Hydrology*, Hydraulic Engineering Circular No. 19 (HEC #19), FHWA, August 1985.

22. *Handbook of Steel Drainage Highway Construction Products*, American Iron and Steel Institute, April 1983.

23. *Structure Excavation Tables for Pipe Culverts, Headwalls, Box Culverts and Minor Structures,* Mississippi State Highway Department, May 1979.

24. Department of Transportation and/or Highway Department Drainage Manuals from the states of Alabama, California, Florida, Georgia, Louisiana, Massachusetts, North Carolina, Texas and Virginia.

25. AASHTO *Model Drainage Manual*, 1991.

CULVERT DESIGN FORM SHEET OF

SQUAD DESIGNER DATE PROJECT HWY NO. COUNTY

HYDROLOGICAL DATA

METHOD _____

DRAINAGE AREA _____ AC.; _____ SQ. MI. (100 % 75 %)

DRAIN LENGTH _____ FT.; _____ MI. (100 % 85 % 0 % 10 %)

EL. MAXI. _____ EL. MIN.; _____

DRAIN SLOPE _____ FT./FT.; _____ FT./MI.

TIME OF CONCENTRATION (t_c) _____ MIN.

DESIGN INTENSITY (I_{DESIGN}) _____ Q DES. / Q100 / Q2 _____ IN./HR.

RATIONAL RUNOFF (C) _____

INTENSITY-DURATION REGION _____

CHANNEL SHAPE _____

$Q_{(DESIGN=___)}$ _____ C.F.S.

$Q_{(BASE=100)}$ _____ C.F.S.

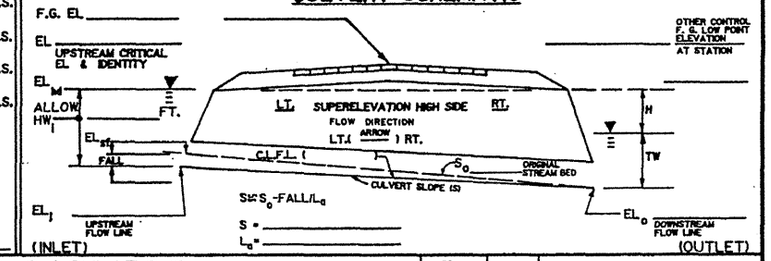
$Q_{(OVER-TOPPING=___)}$ _____ C.F.S.

$Q_{(M.A.S.=2)}$ _____ C.F.S.

CULVERT IN PLACE _____

STATION _____ SKEW ___°; ___ FWD. WORK SHEET NUMBER _____

CULVERT SCHEMATIC



CULVERT DESCRIPTION				HEADWATER CALCULATIONS																
				INLET CONTROL				OUTLET CONTROL												
SHAPE-ENTRANCE-MATERIAL	SIZE	TOTAL FLOW Q (CFS)	FLOW PER BARREL Q/N (1)	HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	L_aS	HW_{ho} (8)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY (f. p. s.) (9)	COMMENTS

ENCROACHMENT RISK ASSESSMENT (BASE STORM): CHECK ✓:

X - YR. OVERTOPPING STORM DISCHARGE	HW ELEV.	1. HAZARD TO PEOPLE & PROPERTY			2. OTHER FACTORS, USE & LOCATION				RISK ACCEPTABLE - TRADITIONAL DESIGN USED
		TRAFFIC NO / YES	ROADWAY NO / YES	BACKWATER NO / YES	NEED FOR EMERGENCY USE NO / YES	LACKS DETOUR NO / YES	ON REGULATORY FLOODWAY NO / YES		RISK SIGNIFICANT - ANALYSIS REQUIRED

TECHNICAL FOOTNOTES

(1) USE Q/NB FOR BOX CULVERTS, (CFS).

(2) $HW_i/D = HW/D$ OR HW_i/D - FROM DESIGN CHARTS.

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON NATURAL GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION).

(5) TW BASED ON DOWNSTREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW OR $(d_c + D)/2$ (WHICHEVER IS GREATER).

(7) $H = \left[1 + k_e + 29n^2 L/R^{1.33}\right] V^2/2g$.

(8) $HW_{ho} = H + h_o - L_aS$; $EL_{ho} = EL_o + H + h_o$.

(9) $V_{hi} = Q/A$ @ d_n (HDS NO.3); $V_{ho} = Q/A$ @ h_o.

SUBSCRIPTS

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

CULVERT BARREL SELECTED

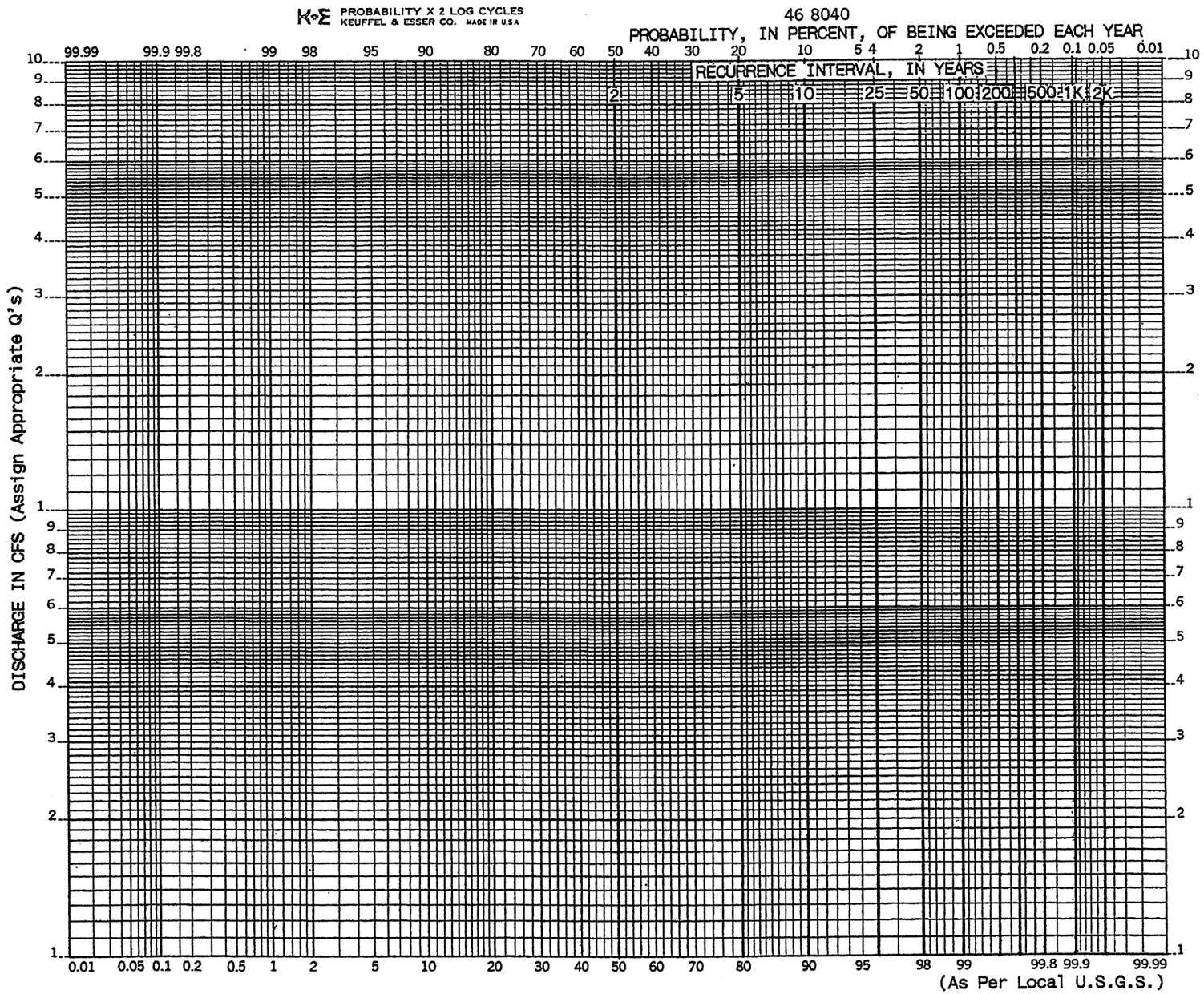
SIZE _____

SHAPE _____

ENTRANCE _____

MATERIAL _____ n _____

MISSISSIPPI DEPARTMENT OF TRANSPORTATION 8/95 VRB BY CH


K+E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 8040
PROBABILITY, IN PERCENT, OF BEING EXCEEDED EACH YEAR
99.99 99.9 99.8 99 98 95 90 80 70 60 50 40 30 20 10 5 4 2 1 0.5 0.2 0.1 0.05 0.01
RECURRENCE INTERVAL, IN YEARS
2 5 10 25 50 100 200 500 1K 2K
DISCHARGE IN CFS (Assign Appropriate Q's)
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
(As Per Local U.S.G.S.)

Design Curves for Side-Tapered Improved Inlets

The following figures present curves to aid in the design of side-tapered improved inlets. Shown on the curves is the headwater vs. discharge for various sizes of reinforced concrete box culverts with side-tapered inlets. The inlet faces have been sized so that the culverts will operate in throat control for slopes 10% and flatter as long as the culverts are operating in inlet control. Requirements for use of the design curves are:

1. Slope. The slope is 10% or flatter.

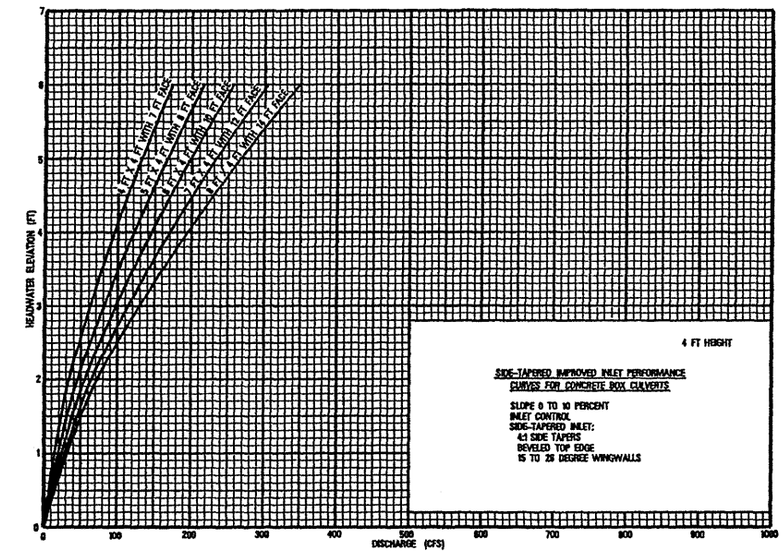
2. Control. The culvert operates in inlet control.

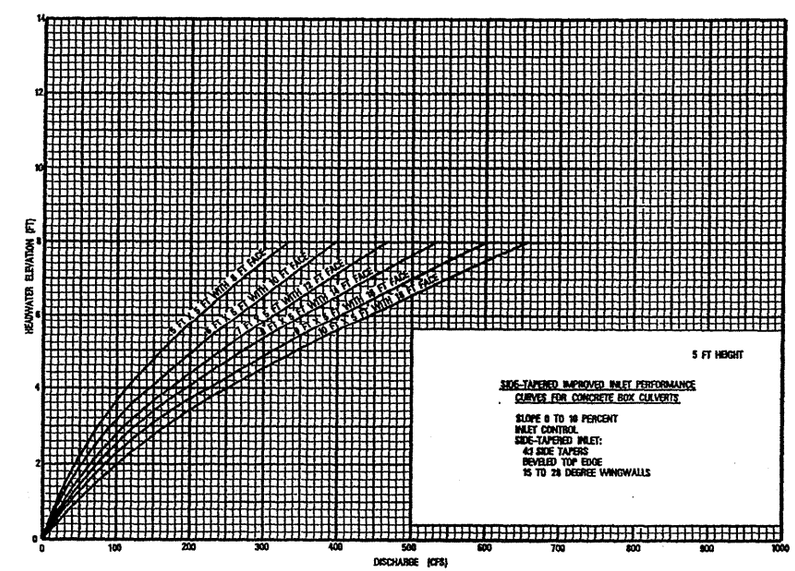
3. Inlet. The inlet is a 4:1 Side-Tapered Improved Inlet with beveled top edge and 15° to 26° wingwalls.

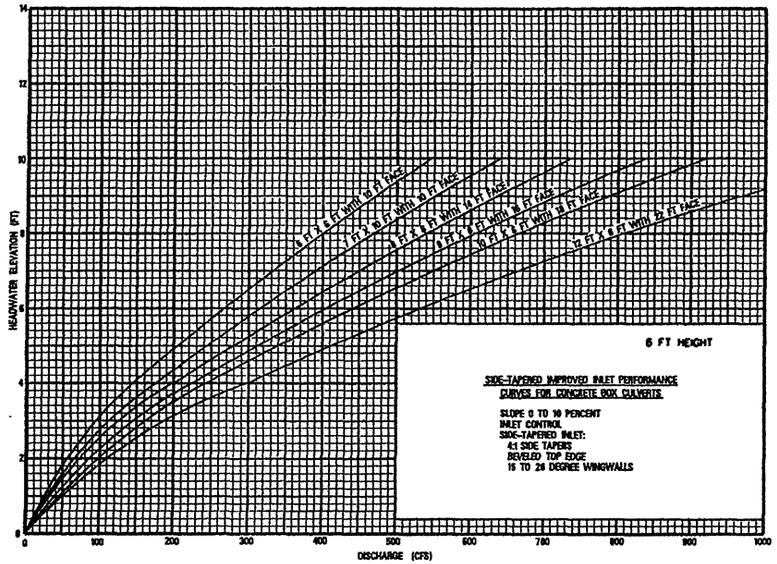
The curves are designed in accordance with HDS 5, *Hydraulic Design of Highway Culverts* using the HY8 computer program. On flat slopes, verify that the culvert is actually in inlet control.

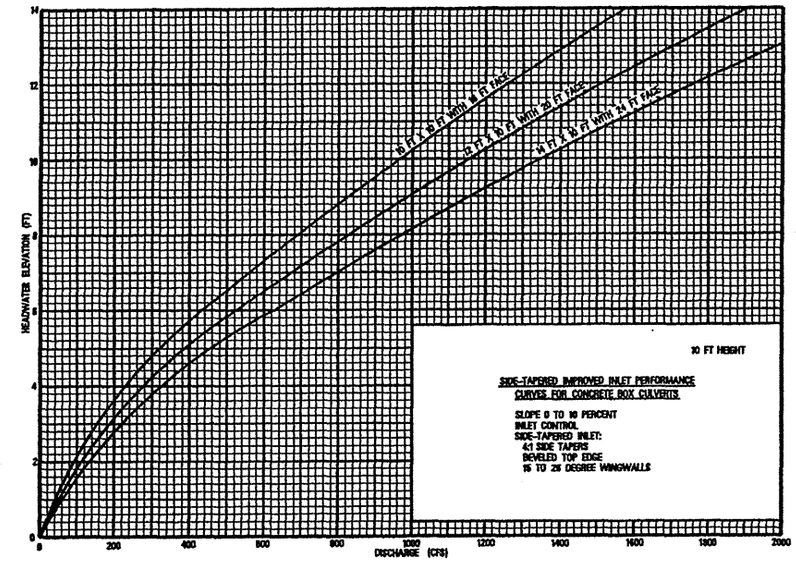
When trial sizes of box culverts are in Inlet Control and have acceptable Outlet Control values, consider Side-Tapered Improved Inlets/Entrances (refer to the design curves). Either the size of box culverts may be decreased or otherwise only nearly acceptable to marginal Headwater values may become within acceptable levels. This also applies to existing box culverts in Inlet Control with acceptable Outlet Control values, where unacceptable Headwater values from undersized box culverts may become acceptable with the addition of these Improved Inlets/Entrances.

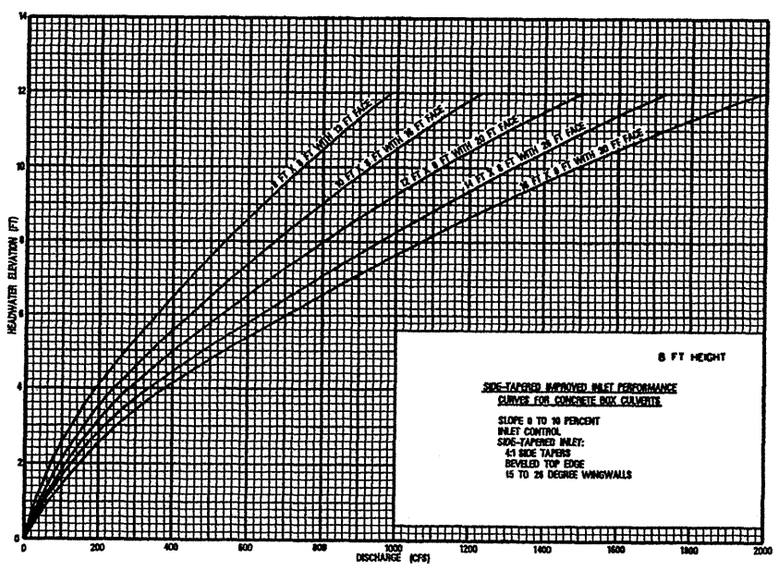
With the trial size box culvert, locate the Side-Tapered Improved Inlet Performance Curves for the given height. At the design discharge, vertically intersect the curve of the trial size with specific face, horizontally read the Headwater depth. Compare this depth with the non-improved Headwater depth, if the improved depth is significantly less and acceptable, a smaller size box culvert may be considered and this sequence repeated – select the appropriate size box culvert accordingly. Existing box culverts are considered similarly where an Improved Inlet/Entrance may lower unacceptable Headwater values to within acceptable levels; however, existing box culverts generally are not decreased in size.

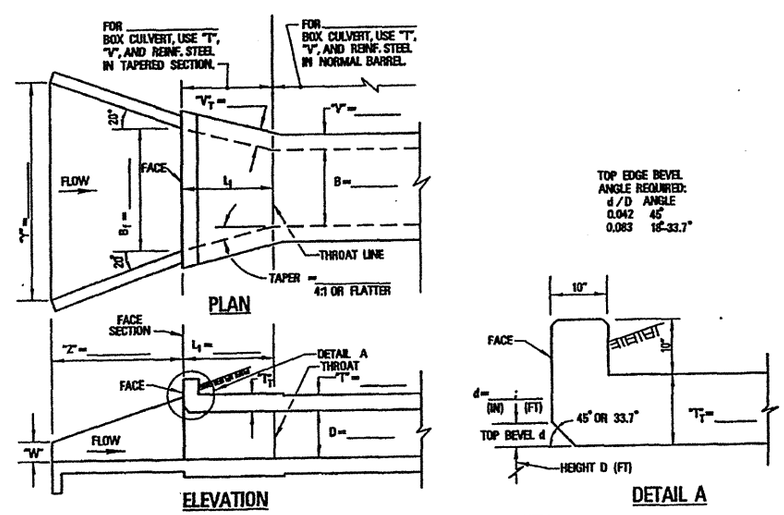












SIDE-TAPERED IMPROVED INLETS

Chapter 8

Special Design Elements

MISSISSIPPI DESIGN MANUAL

Chapter 8

SPECIAL DESIGN ELEMENTS

Section		Page
8-1.0	ROADSIDE DEVELOPMENT	8-1
8-1.01	Erosion Control	8-1
8-1.01.01	Vegetation	8-1
8-1.01.02	Responsibility	8-1
8-1.01.03	Estimating Quantities	8-1
8-1.02	Special Side Ditch Treatment	8-3
8-1.02.01	Responsibility	8-3
8-1.02.02	Design Criteria	8-3
8-1.02.03	Ditch Gradient	8-4
8-1.02.04	Drainage Areas	8-4
8-1.02.05	Ditch Treatment Design	8-4
8-1.03	Other Erosion Control	8-6
8-1.03.01	Slope Paving	8-6
8-1.03.02	Riprap	8-7
8-1.04	Landscaping and Rest Area Development	8-7
8-1.04.01	Responsibilities	8-7
8-1.04.02	Safety Considerations	8-8
8-2.0	ACCESSIBILITY FOR HANDICAPPED INDIVIDUALS	8-8
8-2.01	Buildings	8-8
8-2.02	Bus Stops	8-8
8-2.03	Parking	8-9
8-2.03.01	Off-Street Parking	8-9
8-2.03.02	On-Street Parking	8-11
8-2.04	Accessible Route	8-12
8-2.05	Sidewalks	8-12
8-2.05.01	Criteria for Sidewalks on Accessible Routes	8-12
8-2.05.02	Criteria for Sidewalks on Public Right-of-Way	8-13
8-2.06	Stairs	8-13
8-2.07	Ramps	8-14
8-2.08	Curb-Cut Ramps	8-15
8-2.08.01	Location	8-15
8-2.08.02	Crossing Controls	8-16
8-2.08.03	Types	8-17
8-3.0	AT-GRADE RAILROAD/HIGHWAY CROSSINGS	8-17
8-3.01	Responsibilities	8-17
8-3.02	Design	8-18
8-3.02.01	Sight Distance	8-18
8-3.02.02	Horizontal Alignment	8-18
8-3.02.03	Vertical Alignment	8-18
8-4.0	WEIGH STATIONS	8-21
8-4.01	Detail Design Responsibilities	8-21
8-5.0	AIRWAY/HIGHWAY CLEARANCES	8-21
8-6.0	BICYCLE ACCOMMODATION	8-21
8-6.01	General	8-21
8-6.02	Bikeway Classifications	8-22
8-6.03	Selection Guidelines	8-22
8-6.03.01	Bicycle Paths	8-22
8-6.03.02	Bicycle Lanes	8-22
8-6.03.03	Shared Roadway	8-23
8-6.04	Design	8-23
8-7.0	REFERENCES	8-24

Chapter 8

SPECIAL DESIGN ELEMENTS

This chapter discusses the design criteria, procedures and responsibilities for the special design elements that must be addressed by the highway designer. Proper design of these elements will contribute to highway safety and aesthetics and will improve the operational efficiency of the facility.

8-1.0 ROADSIDE DEVELOPMENT

8-1.01 Erosion Control

8-1.01.01 Vegetation

Most highway projects will require that certain erosion control measures be implemented (e.g., providing vegetation cover on disturbed and barren soil areas). This will prevent soil erosion and improve the appearance of the roadside. The "Vegetation Schedule" sheet that is prepared by the designer will specify the work items to be performed, the pay item numbers, the rates of application, the seasonal requirements and any special notes related to the project. The vegetation schedule may include the following items:

1. topsoil application,
2. ground preparation,
3. fertilizing,
4. seeding,
5. mulching,
6. solid sodding,
7. watering, and
8. insect pest control.

A typical vegetation schedule is illustrated in Figure 8-1A. Vegetation schedules are included with the detail sheets at contract plan assembly. Typical erosion control details are provided in the *Roadway Design Standard Drawings*.

8-1.01.02 Responsibility

The responsibility for erosion control design belongs to the design squad assigned to each project. The design squad will prepare a vegetation schedule sheet that will become a part of the contract plans. The squad leader will determine the disturbed area to be vegetated and will estimate quantities for each item listed on the vegetation schedule.

8-1.01.03 Estimating Quantities

The erosion control items should be included sequentially (i.e., by the pay item number) in the summary of quantities for the project. The procedures for estimating erosion control items are as follows:

1. Topsoil. Normally, the station limits requiring topsoil will be specified on the vegetation schedule. The total quantity of topsoil (cubic yards) is determined by multiplying each area (square feet) by its required topsoil thickness (feet) and dividing by 27 to convert to cubic yards. The cumulative sum of these calculations will yield the total quantity of topsoil required for the project. This value then should be multiplied by a factor of 1.25 to account for a 25% shrinkage in material. Topsoil that is furnished from within the right-of-way is paid for on a square yard basis for placement.

VEGETATION SCHEDULE						
EROSION CONTROL ITEMS		SEASONAL APPLICATIONS — RATES AND DATES				REQUIREMENTS
		SPRING & SUMMER		FALL & WINTER		
Pay Item No.	Items	Rates	Dates	Rates	Dates	
211-B	Topsoil for Slope Treatment (LVM)	4 in. thick	March 1 to September 1	4 in. thick	September 1 to March 1	Topsoil required on slope areas (sandy) at approximate station limits listed below or as directed by the engineer.
212-B	Standard Ground Preparation	per square yard	March 1 to September 1	per square yard	September 1 to March 1	Ground preparation required on areas to receive solid sodding or seeding, as applicable.
213-A⑥	Agricultural Limestone	3 tons/acre	March 1 to September 1	3 tons/acre	September 1 to March 1	Limestone shall be mechanically spread uniformly and incorporated into the soil prior to planting.
213-B	Combination Fertilizer (13-13-13)	1000 lbs/acre	March 1 to September 1	1000 lbs/acre	September 1 to March 1	Fertilizer shall be mechanically spread uniformly and incorporated into the soil prior to planting.
213-C①	Superphosphate	1000 lbs/acre (est.)	March 1 to December 1	—	—	Superphosphate (for bid item purposes).
214-A②	Seeding (bermudagrass)	20 lbs/acre	March 1 to September 1	20 lbs/acre	September 1 to March 1	Seed required on disturbed areas. Unhulled seed may be required during the dormant season as directed.
214-A②	Seeding (bahiagrass)	25 lbs/acre	March 1 to September 1	25 lbs/acre	September 1 to March 1	Seed required on disturbed areas.
214-A③	Seeding (tall fescue)	—	—	20 lbs/acre	August 1 to April 1	Seed required on disturbed areas.
214-A⑤	Seeding (sericea lespedeza)	25 lbs/acre	March 1 to September 1	25 lbs/acre	September 1 to March 1	See note ⑤ below.
214-A③	Seeding (crimson clover)	—	—	20 lbs/acre	August 1 to April 1	Seed required on disturbed areas.
215-A	Vegetative Material for Mulch	2 tons/acre (est.)	March 1 to September 1	2 tons/acre (est.)	September 1 to March 1	The engineer will designate the rates of application (see Subsection 215.03.3).
216-A	Solid Sodding	per square yard	March 1 to September 1	per square yard	September 1 to March 1	Solid sod required on areas specified in the contract or by the engineer.
219-A	Watering	20 gallons/square yd (est.)	March 1 to September 1	20 gallons/square yd (est.)	September 1 to March 1	To be used as directed in planting and establishing solid sod.
220-A④	Insect Pest Control	per acre	—	per acre	—	See Section 220.

① *All areas that have been vegetated, under this contract for at least sixty (60) days, shall receive additional application(s) of fertilizer(s) of the type(s) and rate(s) of applications as determined by soil tests or as directed during the growing seasons the contract is in force. Ground preparation will not be required for the additional applications. Payment for all fertilizers acceptably applied as an additional application(s) will be made in accordance with superphosphate bid item 213-C.*

② *Proposal quantities estimated on the basis that 100% of the area will be seeded.*

③ *Proposal quantities estimated on the basis that 50% of the area will be seeded.*

④ *Quantity estimated on the basis 50% of the area vegetated may require treatment.*

⑤ *Sow, in addition to other specified seeds, on high fill and backslope areas selected by engineers during construction. Proposal quantities estimated on the basis that 50% of the area will be seeded.*

⑥ *The actual rate/acre to be determined by soil test during construction.*

TOPSOIL MAY BE REQUIRED	
FROM STATION	TO STATION
9+00.000	9+85.000
10+00.000	14+75.000
20+00.000	22+35.000
23+85.000	25+50.000

TEMPORARY EROSION CONTROL ITEMS REQUIRED FOR DETOURS		
ITEMS	RATES	SEASONAL LIMITATIONS
Standard Ground Preparation	—	—
Combination Fertilizer (13-13-13)	500 lbs/acre	—
Seeding (bermudagrass)	10 lbs/acre	—
Seeding (tall fescue)	20 lbs/acre	August 1 to April 1
Vegetative Material for Mulch	2 tons/acre	—

EXAMPLE VEGETATION SCHEDULE

Figure 8-1A

2. Ground Preparation. The total area of ground preparation (square yards) may include seeding and/or solid sodding as specified on the vegetation schedule. The method of determining the disturbed area for standard ground preparation is employed by measuring from 10 ft outside the construction limits to the limits shown on the typical sections. Quantity measurements may be taken either from the slopes illustrated in the cross-sections or directly from the plan sheets. Areas measured along slopes will be increased by 25%. Areas measured from the plan sheets will be increased by 50%. This adjusted quantity will be used for the Standard Ground Preparation pay item.

3. Fertilizer. The quantity of fertilizer (tons) is determined by first converting the area of ground preparation (square yards) to an equivalent area in acres and then multiplying this value by the rate of application (tons per acre) for each fertilizer type (e.g., agricultural limestone, combination fertilizer and superphosphate).

4. Seeding. The quantity of seed (pounds) is determined by first subtracting the solid-sodding area from the ground-preparation area and converting the resulting area (square yards) to an equivalent area in acres. This value then is multiplied by the rate of application (pounds per acre) for each seed type. The estimated percent of the area to be seeded is provided on the vegetation schedule.

5. Mulching. The quantity of vegetative material for mulch (tons) is determined by first subtracting the solid-sodding area from the ground-preparation area and converting the resulting area (square yards) to an equivalent area in acres. This value then is multiplied by the rate of application (tons per acre) of mulch.

6. Watering. The quantity of water (gallons) is determined by first multiplying the area of required solid-sodding (square yards) by the rate of application (gallons per square yards) of water and then rounding this value to the nearest whole thousand gallons.

7. Insect Pest Control. The quantity of insect-pest-control area (acres) is determined by first converting the ground-preparation area (square yards) to an equivalent area in acres and then dividing the resulting value by two.

8-1.02 Special Side Ditch Treatment

8-1.02.01 Responsibility

Certain physical conditions (e.g., steep ditch gradients and large volumes of surface runoff) make it difficult to establish vegetation before erosion occurs. Under these conditions, special side ditch treatments may be necessary. The designer should conduct the analysis and design the appropriate side ditch treatment in accordance with the criteria and procedures found in the following sections. Before determining the erosion control ditch treatment, the erosion resistance of the soil (e.g., average, highly erosive) should be obtained from an authoritative source and recorded in the project file. Where unusual conditions exist, the designer may consult with the Hydraulics Section for guidance in the design of ditches, channels and other erosion control measures.

8-1.02.02 Design Criteria

The main factors affecting ditch design include:

1. type of soil,
2. ditch gradient,
3. surface runoff (drainage area), and
4. ditch side slopes.

The alternative side ditch treatments in order of increasing effectiveness for erosion control are as follows:

1. normal seeding and mulching,
2. ditch liners,
3. solid sod or bituminous treated roving,
4. soil reinforcing mats,
5. riprap, or
6. paved ditches.

8-1.02.03 Ditch Gradient

To analyze side ditch treatments, the designer should plot ditch grades on a working profile copy. The gradients should be plotted for all side ditches and for situations where the natural ground slopes toward the toe of fill. A close approximation from the cross sections will be sufficiently accurate. Notations should be made on the working profile copy to indicate where the side ditch flow is released and carried away by a natural drainage channel or intercepted by a drop inlet or cross drain culvert.

8-1.02.04 Drainage Areas

The drainage area should be outlined on the working plan copy to display the entire area that contributes surface runoff to the ditch. This area should include any portion of the pavement surface which is sloped toward the ditch, all construction slopes and any area beyond the construction limits that drains toward the ditch. Table 8-1A provides criteria for side ditch treatments based on the computed drainage area.

8-1.02.05 Ditch Treatment Design

The design of side ditch treatment normally will involve the following steps:

1. Select the proper columns in Table 8-1A.

2. Select the appropriate ditch grade (percent) in Table 8-1A.

3. Identify the maximum permissible area for each type of treatment.

4. Start the area determination process at the upstream end of the drainage area and continue until the area reaches the upper limit for normal seeding and mulching (no special treatment). The ditch liner treatment will begin at this location.

5. Continue the area determination method until the upper limit for ditch liner treatment is reached. The solid sod or bituminous treated roving will begin at this point.

6. Continue the area determination method until the upper limit for solid sod or bituminous treated roving is reached. Where soils permit, increase this upper limit by 25% and begin the soil reinforcing mat treatment until this new limit is reached. Otherwise, begin either riprap or paved ditch treatment and continue to the point of termination.

7. Where the ditch reaches an outlet, the designer should end the indicated ditch treatment and start the area determination method from a zero reading at the upstream end of the next drainage area. Ditch treatments are generally run to the nearest 25-ft interval.

Details of typical ditch treatments are provided in the *Roadway Design Standard Drawings.* When moderate capacity is required, the simple "V" shape ditch should be used. For situations involving relatively large volumes of surface runoff, a flat bottom ditch with the appropriate treatment should be considered.

Generally, concrete should be specified when paved ditches are required. Bituminous paving will be specified where unusual soil conditions may adversely affect concrete. The designer will be advised of these conditions by the District.

Table 8-1A
CRITERIA FOR SIDE DITCH TREATMENT

UPPER AREA LIMIT FOR SIDE DITCH TREATMENT (acres)①②													
Ditch Grade (%) ⑤	Foreslope — Backslope 6:1 — 6:1 10:1 — 3:1			Foreslope — Backslope 6:1 — 4:1 6:1 — 3:1			Foreslope — Backslope 4:1 — 4:1			Foreslope — Backslope 4:1 — 3:1			Ditch Grade (%) ⑤
	No Treatment	Ditch Liner	Bituminous Treated ③ Roving ④	No Treatment	Ditch Liner	Bituminous Treated ③ Roving ④	No Treatment	Ditch Liner	Bituminous Treated ③ Roving ④	No Treatment	Ditch Liner	Bituminous Treated ③ Roving ④	
0.2	1.73	3.14	7.39	1.43	2.62	6.15	1.14	2.08	4.87	0.99	1.83	4.22	0.2
0.3	1.73	3.56	9.07	1.46	2.97	7.51	1.16	2.37	5.98	1.01	2.05	5.19	0.3
0.4	1.83	3.98	10.45	1.51	3.24	8.65	1.21	2.62	6.87	1.04	2.27	5.98	0.4
0.5	1.75	4.00	10.67	1.46	3.31	8.87	1.16	2.64	7.02	1.01	2.30	6.10	0.5
0.6	1.53	3.51	9.49	1.26	2.92	7.86	1.01	2.32	6.25	0.89	2.03	5.46	0.6
0.7	1.33	3.14	8.55	1.09	2.57	7.02	0.89	2.08	5.61	0.77	1.80	4.87	0.7
0.8	1.24	2.87	7.81	1.04	2.40	6.45	0.82	1.90	5.14	0.72	1.66	4.45	0.8
0.9	1.14	2.64	7.19	0.94	2.20	5.98	0.74	1.73	4.74	0.67	1.53	4.13	0.9
1.0	1.04	2.45	6.72	0.84	2.03	5.56	0.69	1.63	4.42	0.59	1.41	3.85	1.0
1.5	0.77	1.88	5.19	0.62	1.53	4.30	0.49	1.24	3.41	0.44	1.09	2.97	1.5
2.0	0.62	1.56	4.37	0.52	1.28	3.61	0.40	1.01	2.89	0.37	0.91	2.50	2.0
2.5	0.52	1.36	3.83	0.42	1.11	3.16	0.35	0.89	2.52	0.30	0.77	2.20	2.5
3.0	0.44	1.16	3.36	0.40	0.99	2.79	0.30	0.79	2.22	0.27	0.69	1.93	3.0
3.5	0.42	1.09	3.09	0.35	0.91	2.57	0.27	0.72	2.03	0.25	0.62	1.78	3.5
4.0	0.37	0.96	2.72	0.30	0.79	2.25	0.25	0.64	1.79	0.22	0.54	1.56	4.0
4.5	0.35	0.89	2.52	0.27	0.74	2.10	0.22	0.59	1.66	0.20	0.49	1.46	4.5
5.0	0.30	0.82	2.35	0.27	0.69	1.98	0.20	0.54	1.58	0.17	0.47	1.36	5.0
5.5		0.57	2.25		0.47	1.85		0.37	1.48		0.32	1.28	5.5
6.0		0.49	2.03		0.42	1.71		0.32	1.33		0.30	1.16	6.0
7.0		0.44	1.80		0.37	1.48		0.30	1.21		0.27	1.04	7.0
8.0		0.37	1.51		0.32	1.26		0.25	0.99		0.22	0.86	8.0
9.0		0.32	1.26		0.27	1.06		0.20	0.84		0.20	0.74	9.0
10.0		0.25	1.01		0.22	0.84		0.17	0.67		0.15	0.59	10.0

Notes:

① *Values in this table are the upper limits for each particular type of side ditch treatment. Riprap or paving is required beyond the limits of these values.*
② *Values in this table are for average erosion resistant soils. For highly-erosive soils, use only 60% of the upper area limit.*
③ *Values in this column are the upper area limits for bituminous treated roving. For solid sod, use 80% of the upper area limit.*
④ *Where soils permit, increase bituminous treated roving by 25% for soil reinforcing mat. Riprap or paving will begin beyond this limit.*
⑤ *Ditch grades exceeding 3% require area reductions when definite drainage areas are unknown. Riprap or paving should begin where abrupt changes in ditch grades occur (e.g., 2.0%± to 6.0%±).*

The location and type of special ditch treatments should be provided on the plan sheets. Table 8-1B provides the plan sheet symbols and the unit of measure for each type of side ditch treatment. If plan sheet information (e.g., the topography, construction notes, survey data) causes the sheet to become difficult to read, the designer may omit the ditch treatment symbols and prepare an itemized list of quantities that will identify each type of treatment by stationing.

8-1.03 Other Erosion Control

8-1.03.01 Slope Paving

To prevent erosion, concrete slope paving should be provided underneath bridges at grade-separated structures. The details of slope paving are normally provided on the bridge sheets, and the quantities are included in the Bridge Summary of Quantities.

For typical rural projects, the slope paving is limited to the area beneath the bridge, extending 4 ft outside the vertical limits of the bridge as shown in "Illustration (A)" of Figure 8-1B. For urban areas, additional slope paving should be provided to wrap around the ends of the embankment as shown in "Illustration (B)" of Figure 8-1B. Details and quantities for the additional wrap around slope paving should be provided on the roadway plans. The portion of the slope paving that is 4 ft outside the vertical limits of the bridge is a roadway item and that portion beneath and within this 4-ft limit of the bridge is a bridge item.

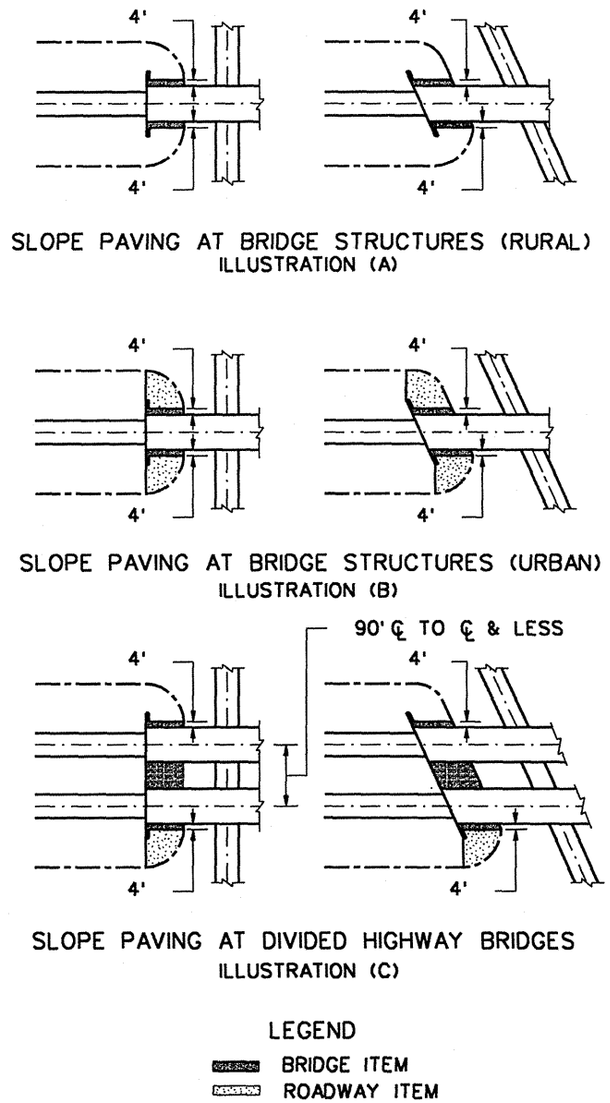
For divided highways with 88 ft or less between the centerlines of the roadways, the slope paving is continuous between the two bridge ends as shown in "Illustration (C)" of Figure 8-1B. When the centerlines are separated by more than 88 ft, the slope paving should follow the limits of the single bridge examples.

Table 8-1B

SYMBOLS AND UNIT OF MEASURE FOR SIDE DITCH TREATMENTS

Ditch Treatment	Plan Sheet Symbol	Unit of Measure
Ditch Liner		square yard
Solid Sod		square yard
Bituminous Treated Roving		square yard
Soil Reinforcing Mat		square yard
Riprap		ton
Paved Ditch (PC concrete)		cubic yard
Paved Ditch (bituminous)	varies	ton

Note: Press on tapes may be used for the plan sheet symbols.



SLOPE PAVING AT BRIDGE STRUCTURES
Figure 8-1B

8-1.03.02 Riprap

To prevent excessive erosion, riprap may be required at various locations within the highway right-of-way. The designer may consult with the Hydraulics Section for guidance in the use of riprap. The following locations should be evaluated for riprap treatment:

1. culvert outfall and intake ends,
2. bends in stream channels,
3. abrupt flowline changes,
4. excessive flowline grades,
5. the point of confluence of two or more streams or channels, and
6. existing erosion problem areas.

8-1.04 Landscaping and Rest Area Development

8-1.04.01 Responsibilities

Roadside development work may be specified during the project planning process. This work may include landscaping of the roadside or development of roadside rest areas. The architect in the Roadway Design Division is responsible for the design of these projects and the preparation of contract plans.

For landscaping work, the contract plans will normally include:

1. a title sheet and general layout,
2. a summary of quantities,
3. a vegetation schedule,
4. plan sheets showing individual plantings, and
5. the applicable *Roadway Design Standard Drawings* (e.g., Typical Planting Details for Trees and Shrubs).

For roadside rest areas, the contract plans should include additional details on:

1. site preparation,
2. buildings,
3. water and sewer installations, and
4. other facilities.

Site preparation for rest areas may be included in the contract plans for construction of the highway facility.

8-1.04.02 Safety Considerations

Adequate clearance between the travelway and roadside obstacles is an important safety consideration. The roadside clear zone must be free of any obstructions. The application of the clear zone criteria is presented in Section 9-2.0.

Large trees are potential hazards and should not be maintained within the required clear zone. The designer should consider removing or relocating trees from within the required clear zone. This is an especially important consideration at grade points, low fills, shallow cuts, diverging roadways and other locations where a vehicle is more likely to leave the roadway. In addition, the planting and maintenance of trees and shrubs, especially at intersections, interchanges and roadside parks, should be conducted so that a driver's line of sight to signs and decision points is not obscured and that the required sight distance is maintained.

8-2.0 ACCESSIBILITY FOR HANDICAPPED INDIVIDUALS

Many highway elements can affect the accessibility and mobility of handicapped individuals. These include sidewalks, parking lots, buildings at transportation facilities, overpasses and underpasses. The Department's accessibility criteria complies with the 1990 *Americans with Disabilities Act* (ADA) and the *ADA Accessibility Guidelines for Buildings and Facilities* (*ADA Guidelines*). The following sections present accessibility criteria from the *ADA Guidelines* which specifically apply to highway-related facilities. Designers are required to meet the criteria presented in these sections. When other agencies or local codes contain accessibility standards which exceed the *ADA Guidelines*, then the stricter criteria may be required. This will be determined on a case-by-case basis.

8-2.01 Buildings

For interior accessibility criteria, the following will apply:

1. New. All new buildings, airport terminals, rest areas, weigh stations and transit stations (e.g., stations for rapid rail, light rail, commuter rail, intercity bus, intercity rail, high-speed rail and other fixed guideway systems) shall meet the accessibility criteria set forth in the *ADA Guidelines*. The designer should review the *ADA Guidelines* to determine the appropriate accessibility requirements for building interiors, including rest rooms, drinking fountains, elevators, telephones, etc.

2. Existing. In general, for alterations made to existing buildings or facilities, the designer must meet the accessibility requirements for the alteration made to the facility, unless it is prohibitively expensive to do so. The designer should review the *ADA Guidelines* to determine the appropriate criteria and, if required, the designer should identify where exceptions may be allowed by the *ADA Guidelines*.

8-2.02 Bus Stops

The following accessibility criteria apply to the construction of bus stops:

1. Bus Stop Pads. New bus stop pads constructed to be used in conjunction with a lift or ramp shall meet the following criteria:

a. A firm, stable surface must be provided.

b. It must have a minimum clear length of 96 in. (measured from the curb or roadway edge) and minimum clear width of 60 in. (measured parallel to the roadway) depending on the legal or site constraints.

c. It must be connected to streets, sidewalks or pedestrian paths by at least one accessible route.

d. The slope of the pad parallel to the roadway must be the same as the roadway to the maximum extent practical.

e. For drainage purposes, a maximum cross slope of 2% perpendicular to the roadway is allowable.

2. Bus Shelters. Where new or replaced bus shelters are provided, they must be installed or positioned to permit a wheelchair user to enter from the public way and reach a location within the shelter having a minimum clear floor area of 30 in. by 48 in. An accessible route shall be provided from the shelter to the boarding area.

3. Signs. All new bus route identification signs should be sized based on the maximum dimensions permitted by local, State or Federal regulations or ordinances. The signs shall have an eggshell, matte or other non-glare finish. The characters or symbols shall contrast with their background (i.e., light characters on a dark background or dark characters on a light background).

8-2.03 Parking

8-2.03.01 Off-Street Parking

The following criteria apply to off-street handicapped parking spaces:

1. Minimum Number. Table 8-2A provides the criteria for the minimum number of accessible spaces. A typical handicapped stall layout is shown in Figure 8-2A.

   One out of every eight accessible spaces, but not less than one, shall have an access aisle 96 in. wide and must be designated as van accessible.

2. Location. Parking spaces for disabled individuals and accessible passenger loading zones that serve a particular building shall be the spaces or zones closest to the nearest accessible entrance on an accessible route. In separate parking structures or lots that do not serve a particular building, parking spaces for disabled individuals shall be located on the shortest possible circulation route to an accessible pedestrian entrance of the parking facility. In buildings with multiple access entrances with adjacent parking, accessible parking spaces shall be dispersed and located closest to the accessible entrances.

3. Signing. Parking spaces for the handicapped shall be designated by above-grade signs with white lettering against a blue background and shall bear the international symbol of access. The sign shall not be obscured by a vehicle parked in the space. Van-accessible spaces shall have an additional sign stating the space is "Van-Accessible" below the symbol of accessibility.

4. Dimensions. The parking spaces designated for the handicapped shall be at a minimum 96 in. wide with an additional 60 in. minimum access aisle or 96 in next to van-accessible spaces. Parking access aisles shall be part of an accessible route to the building or facility entrance. Parked vehicular overhangs shall not reduce the clear width of an accessible circulation route.

Table 8-2A

MINIMUM NUMBER OF ACCESSIBLE SPACES FOR HANDICAPPED USERS

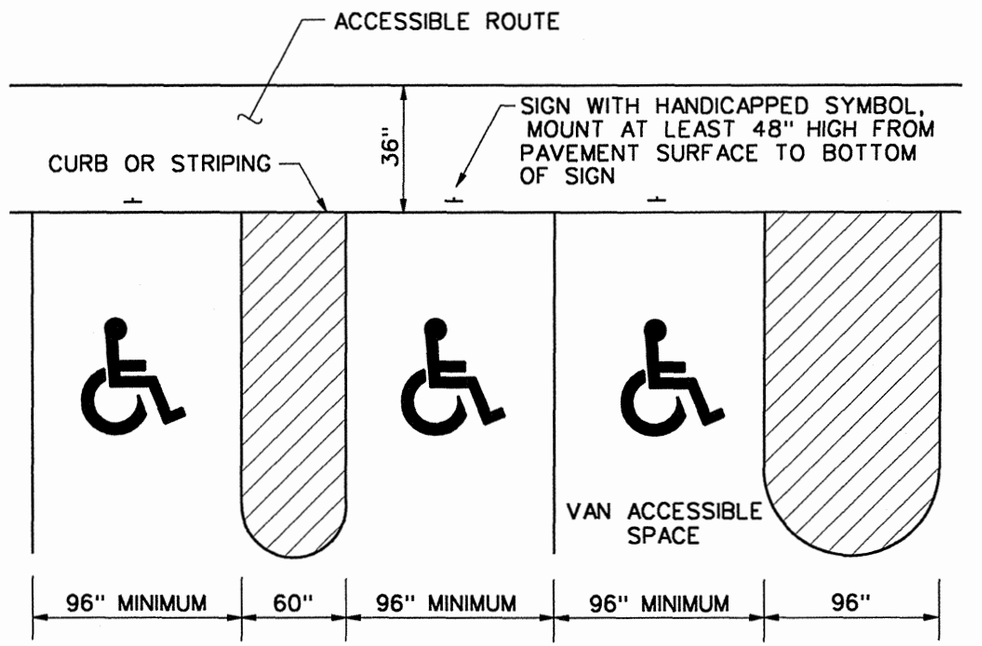
Total No. of Parking Spaces	Minimum Number of Accessible Spaces
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 to 300	7
301 to 400	8
401 to 500	9
501 to 1000	2% of total
1001 and over	20 plus 1 for each 100 over 1000

Notes:

a. If one or more passenger loading zones are provided, then at least one passenger loading zone shall comply with Comment #5 in Section 8-2.03.01.

b. Parking spaces for side-lift vans are accessible parking spaces and may be used to meet the requirements of this Section.

c. The total number of accessible parking spaces may be distributed among closely spaced parking lots, if greater accessibility is achieved.



Note: Two accessible parking spaces may share a common access aisle.

HANDICAPPED PARKING STALL DIMENSIONS (Off-Street Parking)

Figure 8-2A

Parking spaces and access aisles shall be level with surface slopes not exceeding 2% in all directions. Any parking garage or terminal should have a 114 in. clearance at its entrance, exit and along the route to and from at least two parking spaces which have a 114 in. vertical clearance.

5. Passenger Loading Zones. Passenger loading zones shall provide an access aisle at least 60 in. wide and 20 ft adjacent and parallel to the vehicular pull-up space. If there are curbs between the access aisle and the vehicular pull-up space, then a curb-cut ramp complying with Section 8-2.08 shall be provided. Vehicular standing spaces and access aisles shall be essentially level. Surface slopes shall not exceed 2% in all directions.

8-2.03.02 On-Street Parking

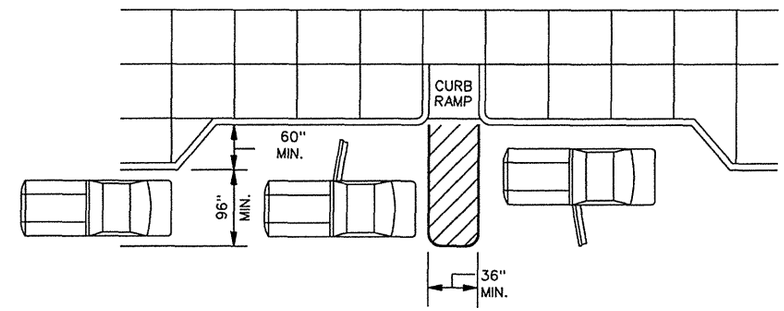
Where new on-street paid or time-limited parking is provided and designated in districts zoned for business uses, the on-street parking design should desirably meet the following accessibility criteria:

1. Minimum Number. Table 8-2A provides the criteria for the minimum number of on-street accessibility spaces.

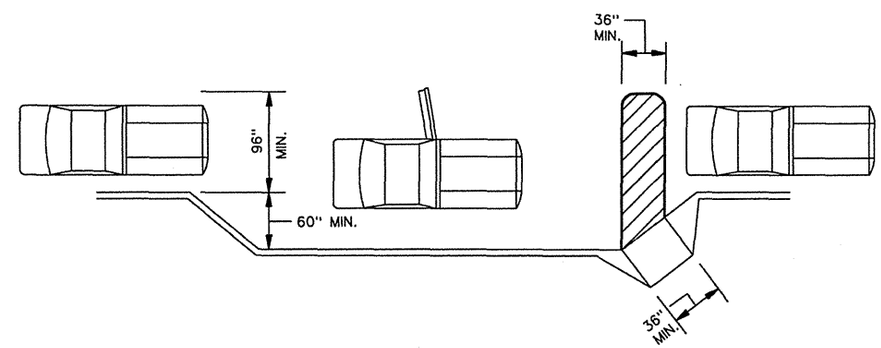
2. Location. On-street accessibility parking spaces should be dispersed throughout the project area. To the maximum extent feasible, accessible on-street parking should be located in level areas.

3. Dimensions. At a minimum, a 96 in. parking space with a 60 in. access aisle should be provided. This is illustrated in Figure 8-2B. The travel lane should not encroach into the access aisle.

4. Signs. Parking spaces for the handicapped shall be designated by above-ground signs with white lettering against a blue background, and the signs shall bear the international symbol of access.



(a) TWO ACCESSIBLE PARALLEL PARKING SPACES IN SERIES, SEPARATED BY AN ACCESSIBLE AISLE, WITH BOTH DRIVER-SIDE AND PASSENGER-SIDE ACCESS DEMONSTRATED.



(b) SINGLE ACCESSIBLE PARALLEL PARKING SPACE WITH DRIVER-SIDE ACCESS DEMONSTRATED; PASSENGER SIDE ACCESS CAN BE PROVIDED BY PARKING IN LINE WITH STANDARD ON-STREET SPACES.

HANDICAPPED PARKING
(On-Street Parking)

Figure 8-2B

5. Curb-Cut Ramps (Adjacent to Accessible Space). If there are curbs adjacent to an on-street accessible parking space, then a curb-cut ramp complying with Section 8-2.08 will be provided. Access parking spaces adjacent to intersections may be served by the sidewalk curb-cut ramp at the intersection, provided that the path of travel from the access aisle to the curb-cut ramp is within the pedestrian crossing area.

6. Parking Meters. Where provided, parking meter controls shall be a maximum of 42 in. above the sidewalk or pedestrian circulation path. Controls and operating mechanisms shall be operable with one hand and will not require tight grasping, pinching or twisting of the wrist. The force required to activate controls shall be no greater than 5 lbf. A firm, stable and slip-resistant area (30 in by 48 in), with the least possible slope, should be provided at the controls and be connected to the sidewalk by a continuous passage that is a minimum of 36 in. wide.

8-2.04 Accessible Route

An accessible route is a continuous, unobstructed path connecting all accessible elements and spaces in a building, facility or site. A "site" is defined as a parcel of land bounded by a property line or a designated portion of a public right-of-way. A "facility" is defined as all or any portion of buildings, structures, site improvements, complexes, equipment, roads, sidewalks, passageways, parking lots, or other real or personal property on a site. Interior accessible routes may include corridors, floors, ramps, elevators, lifts and clear floor space at fixtures. Exterior accessible routes may include parking access aisles, curb-cut ramps, crosswalks at vehicular ways, sidewalks, ramps and lifts.

Accessible routes must be provided as follows:

1. At least one accessible route within the boundary of the site shall be provided from public transportation stops, accessible parking, accessible passenger loading zones, and public streets or sidewalks to the accessible building entrance they serve. The accessible route shall, to the maximum extent feasible, coincide with the route for the general public.

2. At least one accessible route shall connect accessible buildings, facilities, elements and spaces that are on the same site.

3. At least one accessible route shall connect accessible buildings or facility entrances with all accessible spaces and elements and with all accessible dwelling units within the building or facility.

For highway projects, the application of the accessible route criteria applies to definitive sites which are related to highway purposes. These include rest areas, recreational areas, park-and-ride lots, etc. Section 8-2.05 provides the accessibility requirements for sidewalks.

8-2.05 Sidewalks

8-2.05.01 Criteria for Sidewalks on Accessible Routes

For sidewalks on accessible routes, the following accessibility criteria shall be met:

1. Width. The minimum clear width shall be 36 in, except at doors which may have a minimum width of 32 in.

2. Passing Space. If the sidewalk has less than 60 in. clear width, then passing spaces at least 60 in. by 60 in. shall be located at reasonable intervals not to exceed 200 ft. A T-intersection between two walks is an acceptable passing space.

3. Surface. All sidewalk surfaces shall be stable, firm and slip resistant. The longitudinal gradient should be flush and free of abrupt changes. However, changes in level up to 0.25 in. may be vertical and without edge treatment. Changes in level between 0.25 in. and 0.50 in. shall be beveled with a slope

no greater than 50%. Changes greater than 0.50 in. shall be accommodated with a ramp (see Section 8-2.07).

Gratings should not be placed within the walking surface. If, however, gratings are located in walking surfaces, then they shall have spaces no greater than 0.50 in. wide in one direction. If gratings have elongated openings, then they shall be placed so that the long dimension is perpendicular to the dominant direction of travel.

4. Slope. The sidewalk cross slope shall not exceed 2%. If the longitudinal gradient exceeds 5%, the sidewalk must meet the accessibility criteria for ramps (see Section 8-2.07).

5. Protruding Objects. Objects projecting from walls (e.g., signs, telephones, canopies) with their leading edges between 27 in. and 80 in. above the finished sidewalk shall not protrude more than 4 in. into any portion of the sidewalk. Freestanding objects mounted on posts or pylons may overhang their mountings up to a maximum of 12 in. when located between 27 in. and 80 in. above the sidewalk or ground surface. Protruding objects less than 27 in. or greater than 80 in. may protrude any amount provided that the effective width of the sidewalk is maintained. Where the vertical clearance is less than 80 in., a barrier shall be provided to warn the blind or visually impaired person.

6. Buffer Area. Sidewalks shall be separated from roadways by curbs, buffer areas or by other separations, which shall be continuous except where interrupted by driveways, alleys or connections to accessible elements.

7. Bus Stops. Where bus passenger loading areas or bus shelters are provided on or adjacent to sidewalks, they must comply with the criteria in Section 8-2.02.

8. Curb-Cut Ramps. All curb-cut ramps on an accessible route must comply with the criteria in Section 8-2.08.

8-2.05.02 Criteria for Sidewalks on Public Right-of-Way

All sidewalks on public ROW that are not part of an accessible route should desirably meet the criteria presented in Section 8-2.05.01. However, considerable flexibility is required to meet the adjacent roadway conditions and to provide practical designs. The criteria in Section 8-2.05.01 should be used as guidance, unless noted as follows:

1. Slopes. The flattest longitudinal slope practical should be provided. Preferably, the longitudinal slope should not exceed 8.3% (12:1) or the longitudinal slope of the adjacent street. Sidewalk slopes greater than 5% do not require the use of handrails as defined in Section 8-2.07.

2. Buffer Area. Sidewalks adjacent to the curb or roadway may be offset to avoid a non-conforming cross slope at driveway aprons by diverting the sidewalk around the apron.

3. Street Furniture. Street furniture such as signal controller cabinets, light standards, strain poles, utility poles, mailboxes, sign supports, etc., should not be placed within the required sidewalk width. In those locations where it is impractical to provide the minimum sidewalk width, an accessible width of 36 in. must be maintained.

8-2.06 Stairs

Stairs shall not be part of an exterior accessible route or a sidewalk on public ROW because they cannot be safely negotiated by individuals in wheelchairs. Where stairs are used as part of an access route to a building or facility not subject to the ADA requirements, they should be designed to be accessible by other handicapped individuals. Therefore, the design of stairs must comply with Section 4.9 of the *ADA Guidelines*. This includes, for example, the provision of handrails.

8-2.07 Ramps

Any part of an accessible route with a slope greater than 5% shall be considered a ramp and shall conform to the *ADA Guidelines*. This includes the provision of handrails. The following criteria must be met for ramps on accessible routes:

1. Slope and Rise. The least possible slope should be used for any ramp. Table 8-2B provides the maximum allowable ramp slopes for new construction. Curb-cut ramps and ramps to be constructed on existing sites or in existing buildings or facilities may have slopes and rises as shown in Table 8-2C, if space limitations prohibit the use of a 12:1 slope or less.

2. Width. The minimum clear width of a ramp shall be 36 in.

3. Landings. Ramps shall have level landings at the bottom and top of each run. Landings shall have the following features:

   a. The landing shall be at least as wide as the ramp run leading to it.

   b. The landing length shall be a minimum of 60 in. clear.

   c. If ramps change direction at landings, the minimum landing size shall be 60 in. by 60 in.

   d. See Table 8-2B for the maximum distance between landings.

4. Handrails. If a ramp has a rise greater than 6 in. or a horizontal run greater than 72 in., then it shall have handrails on both sides. Handrails are not required on curb-cut ramps. When handrails are required, they shall have the following features:

   a. Handrails shall be provided along both sides of ramp segments. The inside handrail on switchback or dogleg ramps shall always be continuous.

Table 8-2B

ALLOWABLE RAMP DIMENSIONS (New Construction)

Slope	Maximum Rise	Maximum Run
Steeper than 16:1 but no steeper than 12:1	30 in.	30 ft
Steeper than 20:1 but no steeper than 16:1.	30 in.	40 ft

Note: A slope steeper than 12:1 is not allowed.

Table 8-2C

ALLOWABLE RAMP DIMENSIONS (Existing Sites, Buildings and Facilities)

Slope	Maximum Rise	Maximum Run
Steeper than 10:1 but no steeper than 8:1	3 in.	2 ft
Steeper than 12:1 but no steeper than 10:1	6 in.	5 ft

Note: A slope steeper than 12.5% (1:8) is not allowed.

b. If handrails are not continuous, they shall extend at least 12 in. beyond the top and bottom of the ramp segment and shall be parallel with the floor or ground surface.

c. The clear space between the handrail and the wall shall be 1.5 in.

d. Gripping surfaces shall be continuous.

e. Top of handrail gripping surfaces shall be mounted between 34 in. and 38 in. above ramp surfaces.

f. Ends of handrails shall be either rounded or returned smoothly to floor, wall or post.

g. Handrails shall not rotate within their fittings.

5. Cross Slope and Surfaces. The cross slope of ramp surfaces shall be no greater than 2%. Ramp surfaces shall comply with the criteria for "Surface" for sidewalks (Section 8-2.05).

6. Edge Protection. Ramps and landings with dropoffs shall have curbs, walls, railings or projecting surfaces that prevent people from slipping off the ramp. Curbs shall be a minimum of 2 in. high.

7. Outdoor Conditions. Outdoor ramps and their approaches shall be designed so that water will not accumulate on walking surfaces.

8-2.08 Curb-Cut Ramps

Curb-cut ramps and other provisions for the handicapped are required on all projects involving the provision of curbs and sidewalks at all pedestrian crosswalks. For the purpose of this section, a pedestrian crosswalk is defined as the portion of a street ordinarily included within the prolongation or connections of lateral lines of sidewalks at intersections. It also includes any portion of a highway or street distinctly indicated as a crossing for pedestrians by lines or other markings on the surface, except such prolonged or connecting lines from an alley across a street.

8-2.08.01 Location

When determining the need for a curb-cut ramp, the designer should consider the following:

1. Curb-cut ramps will be constructed at all crosswalks which extend from a paved sidewalk in an intersection and will be provided on all corners. At T-intersections, the designer must ensure that curb-cut ramps are located on the side opposite the minor intersecting road. Curb-cut ramps may not be required for minor maintenance-type activities, signing projects, pavement marking projects, roadway lighting projects or other minor activities that do not disturb existing accessibility. Applicability on these types of projects will be determined on a case-by-case basis.

2. There should be full continuity of use throughout; i.e., opposing ramps must always be provided even if outside project limits.

3. Curb-cut ramps shall be located or protected to prevent their obstruction by parked vehicles.

4. Curb-cut ramps should be positioned so as not to cause a safety hazard for blind pedestrians.

5. Curb-cut ramps at marked crossings shall be wholly contained within the markings, excluding any flared sides.

6. A diagonal curb-cut ramp shall be wholly contained within the painted markings, including any flared sides. There shall be at least 24 in. of full-height curb within the crosswalk. In addition, there shall be at least 48 in. between the gutter line and the corner of the two intersecting crosswalks. See Figure 8-2C for an illustration of these criteria.

7. Curb-cut ramps shall be provided along all accessible routes.

8. The function of the curb-cut ramp must not be compromised by other highway features (e.g., guardrail, catch basins, utility poles, signs).

9. Curb-cut ramps are required at all curbed intersections with connecting sidewalks. However, an exception request may be approved for locations where there are valid reasons to restrict or prohibit pedestrian access.

8-2.08.02 Crossing Controls

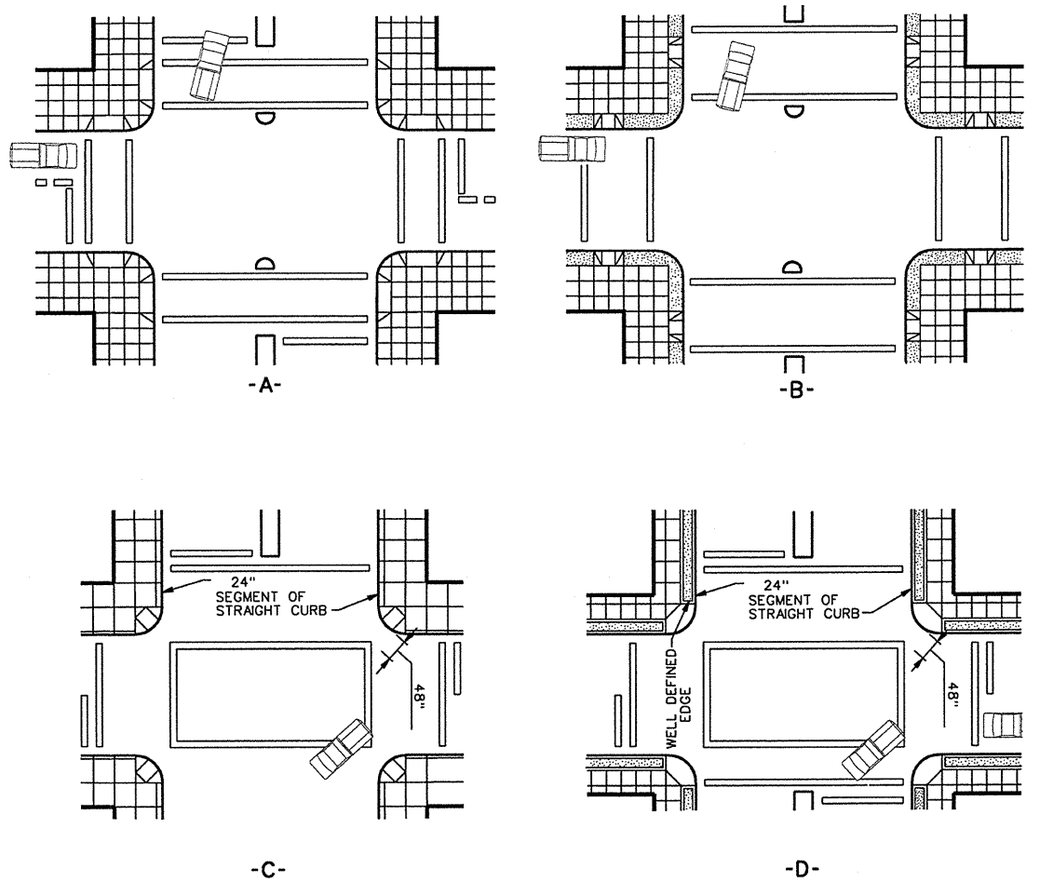
If a pedestrian crosswalk and curb-cut ramp are present at an intersection with a traffic signal that has pedestrian detectors (pushbuttons), the following will apply:

1. Location. Controls should be located as close as practical to the curb-cut ramp and, to the maximum extent feasible, shall permit operation from a level area immediately adjacent to the controls.

2. Surface. A firm, stable and slip-resistant area, a minimum of 36 in. by 48 in., should be provided to allow a forward or parallel approach to the controls.

3. Mounting Height. Pedestrian-actuated crossing controls shall be a maximum of 42 in. above the sidewalk.

4. Controls. Pushbuttons shall be raised or flush and shall be a minimum of 2 in. in the smallest dimension. The force required to activate controls shall be no greater than 5 lbf.



Notes: See the Roadway Design Standard Drawings for design details on curb-cut ramps.

CURB-CUT RAMPS AT MARKED CROSSINGS

Figure 8-2C

8-2.08.03 Types

Details for the construction of curb-cut ramps are illustrated in the *Roadway Design Standard Drawings*. The designer must ensure that the curb-cut ramp type is appropriate for the site and location. The following provides information for each type of curb-cut ramp used by the Department:

1. Type I. Flared type with grass border area. This design may be used at mid-block locations or at corners of intersections. The traffic stop line should be placed at least 48 in. in advance of the curb-cut ramp.

2. Type II. Diagonal type. This design is normally used at intersections with small turning radii. The usage of the diagonal curb-cut ramp should be avoided wherever practical due to its effect on the crosswalk width. The bottom of diagonal curb ramps should have a 48-in. minimum clear space within the marked crossing as shown. The curb-cut ramps should also have a 24-in. segment of straight curb located on each side of the ramp within the markings. This design may also be used at mid-block locations where the sidewalk is adjacent to the curb line and on the straight side of a T-intersection. If a 48-in. wide bypass area at the back of the ramp is not available, see Type IV.

3. Type III. Flared type at corners. This design is normally used at intersections with large turning radii. The curb-cut ramps at marked crossings should be contained within the markings, excluding the flared sides.

4. Type IV. Tangent type. This design may be used where there is not at least a 48-in. wide bypass area available at the back of the ramp to an obstruction.

5. Islands. Any raised islands in a crossing should be cut through level with the street or have curb-cut ramps at both sides with a level area at least 48 in. long in the portion of the island intersected by the crossing for safe refuge.

All curb-cut ramps should be constructed of portland cement concrete. Additional notes can be found in the *Standard Drawings*.

8-3.0 AT-GRADE RAILROAD/HIGHWAY CROSSINGS

8-3.01 Responsibilities

Projects with at-grade railroad/highway crossings will require special consideration by the designer. The railroad company is responsible for all work necessary for the adjustment of its tracks to meet altered or established highway grades, and the railroad will construct the roadway grade crossing as indicated on approved MDOT plans. The railroad company will also install flashing light signals, automatic gates, guardrails and other protective devices as required by the Department. All work performed by the railroad company will be conducted under an agreement between the railroad company and the Department. The agreement will be on a force account basis or with a contract that will allow the Department to reimburse the railroad company for all construction costs.

Plans for field inspection should be submitted to the Office of Intermodal Planning, Rails Manager as soon as they are available. Two sets of plans should be provided. It is desirable that a railroad representative participate in the field inspection. The Rails Manager will be responsible for negotiations with the railroad company and will prepare the required railroad agreements. Following the field inspection, the Roadway Design Division will prepare the final plans and submit plan copies to the Rails Manager for further processing with the railroad company.

If a project requires a railroad right-of-way easement or use permit, the Right-of-Way Division will prepare the necessary documents and coordinate this effort with the railroad company.

8-3.02 Design

8-3.02.01 Sight Distance

There are two sight distance applications that must be addressed at railroad/highway grade crossings:

1. Case A. Allowing an approaching vehicle to either cross or stop.
2. Case B. Allowing a stopped vehicle to cross the railroad tracks.

Figure 8-3A and Figure 8-3B illustrate the conditions under which Case A and Case B sight distances apply. Table 8-3A presents the sight distances that are required for both Case A and Case B. If actual field conditions differ from those presented in Figure 8-3A and Figure 8-3B (e.g., skewed crossings, more than one set of tracks), the designer should reference the AASHTO *A Policy on Geometric Design of Highways and Streets* to make adjustments to the criteria in Table 8-3A.

The sight distances for Case B should be available at all railroad/highway crossings. Case A is important at crossings with only passive control devices (i.e., crossbucks, pavement markings, advance warning signs). Where practical, the criteria for Case A should also be met at crossings with active control devices (i.e., flashing lights, automatic gates). The designer should note, however, that the Case A sight distances may be difficult to attain in the field. Where these distances are not practical, the following presents potential countermeasures:

1. installing active warning devices where only passive devices currently exist;

2. employing speed control signs, flashing advance warning lights and other devices to lower approaching vehicular speeds to be consistent with the available sight distance; and/or

3. forcing all vehicles to a complete stop.

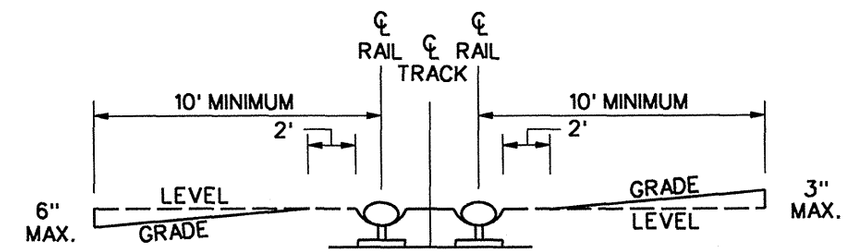
8-3.02.02 Horizontal Alignment

Where practical, the alignment of the highway and railroad crossing should intersect at an angle of 90°, and neither the highway nor the railroad should be on a horizontal curve. If these objectives are met, this will enhance driver safety and comfort; it will reduce maintenance problems; and it will improve roadway rideability. The designer should refer to Chapter 3 for the Department's criteria on horizontal alignment for highways.

8-3.02.03 Vertical Alignment

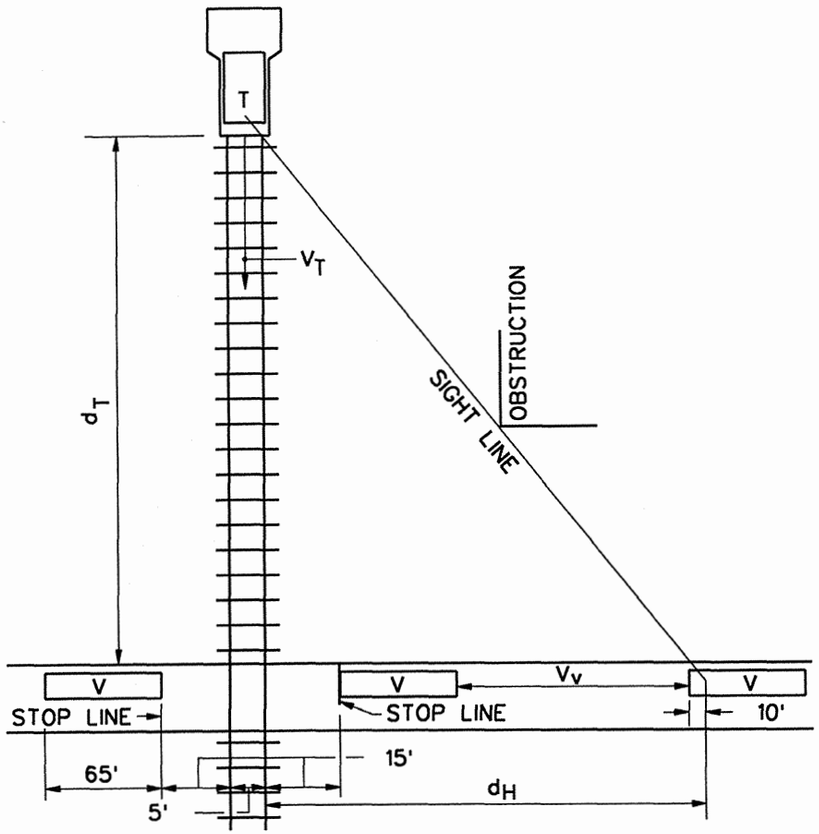
Desirably, the highway will be relatively level where it crosses the railroad. Where vertical curves are provided, they should meet the Department's criteria for vertical alignment presented in Chapter 4.

Figure 8-3C presents a minimum design for vertical alignment at railroad crossings to prevent low-clearance vehicles from bottoming out on the tracks. This design should be provided unless railroad track superelevation dictates otherwise.



PROFILE AT RAILROAD/HIGHWAY GRADE CROSSING

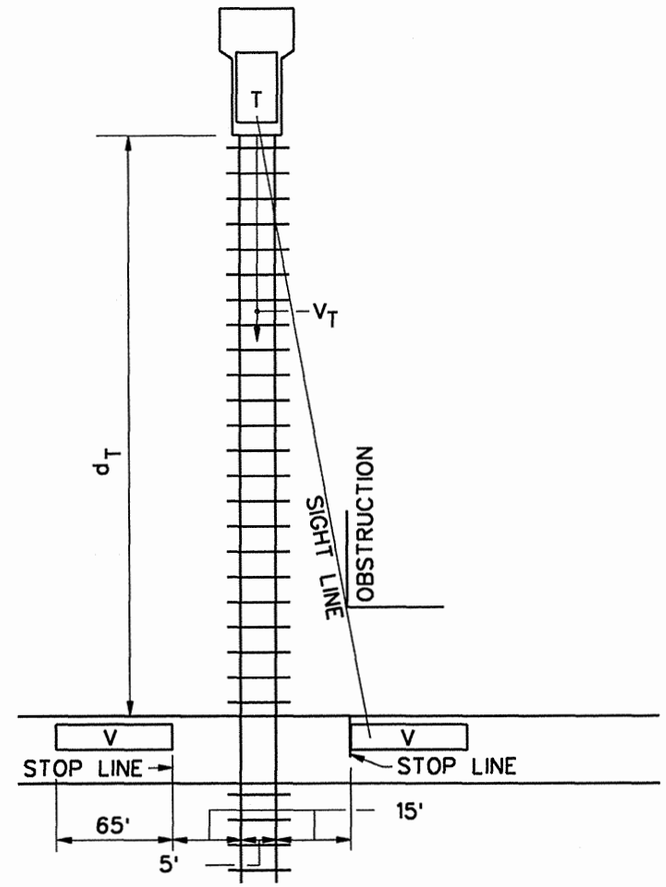
Figure 8-3C



d_T = *sight distance leg along the railroad tracks to permit the maneuvers described for d_H (feet)*
V_T = *velocity of train (mph)*
d_H = *sight distance leg along the highway allowing a vehicle proceeding to speed V_V to cross tracks safely even though a train is observed at a distance d_T from the crossing or to safely stop the vehicle without encroachment of the crossing area (feet)*
V_V = *velocity of vehicle (mph)*

CASE A: APPROACHING VEHICLE TO SAFELY CROSS OR STOP AT RAILROAD CROSSING

Figure 8-3A



d_T = *sight distance along railroad tracks (feet)*
V_T = *velocity of train (mph)*

CASE B: DEPARTURE OF VEHICLE FROM STOPPED POSITION TO CROSS SINGLE RAILROAD TRACK

Figure 8-3B

Table 8-3A

SIGHT DISTANCE REQUIREMENTS FOR RAILROAD/HIGHWAY GRADE CROSSINGS

Train Speed (mph)	Case B Departure from Stop	Case A Moving Vehicle						
		Design Speed (mph)						
	0	10	20	30	40	50	60	70
	Distance Along Railroad from Crossing, d_T(ft)							
10	240	145	103	99	103	112	122	134
20	480	290	207	197	207	224	245	269
30	719	435	310	296	310	337	367	403
40	959	580	413	394	413	449	489	537
50	1200	725	517	493	517	561	611	671
60	1439	870	620	591	620	673	734	806
70	1679	1015	723	690	723	786	856	940
80	1918	1160	827	789	827	898	978	1074
90	2158	1305	930	887	930	1010	1101	1209
		Distance Along Highway from Crossing, d_H(ft)						
		69	132	221	338	486	659	865

Note: Values apply to a 65-ft truck crossing a single set of tracks at approximately 90° on grades of 3% or less.

8-4.0 WEIGH STATIONS

The need for a new weigh station facility should be evaluated whenever a proposed project involves an existing weigh station or crosses the State line. The Roadway Design Division Engineer, or assigned representative, will contact the Office of Enforcement of the Mississippi Department of Transportation for recommendations on a replacement facility.

If a new weigh station is warranted, the Roadway Design Division Engineer, or assigned representative, will initiate and coordinate the necessary planning actions with the Law Enforcement Bureau, the District Office and others as required. After site selection and completion of surveys, the Roadside Development Section will design the weigh station facility and will prepare the contract plans, the specifications and the cost estimate. The site preparation for the weigh station normally will be included in the roadway contract. The building and facility construction should be let under a separate contract.

8-4.01 Detailed Design Responsibilities

The architect in the Roadway Design Division will coordinate the design of weigh station projects and the preparation of contract plans and specifications. The plans for weigh station projects should include:

1. site preparation and drainage,
2. buildings,
3. scales,
4. site lighting and electrical power,
5. water and sewer installation,
6. radio tower installation,
7. signing and flagpole, and
8. erosion control and landscaping.

8-5.0 AIRWAY/HIGHWAY CLEARANCES

Highways in the vicinity of airports must be designed to provide minimum clearance between the highways and the navigable airspace. Navigable airspace is defined as the space above the approach surfaces adjacent to the runway. The bottom of the approach surface is established by a specified guide path ascending upward from the end of the runway. Highway appurtenances such as overhead signs or light standards must not penetrate the navigable airspace. The designer is referred to the following publications for design criteria related to airway/ highway clearances:

1. *Proposed Construction or Alteration of Objects that May Affect the Navigable Airspace*, Advisory Circular 70/7460-2I, Federal Aviation Administration.

2. *Objects Affecting Navigable Airspace*, Federal Aviation Regulations Part 77, Federal Aviation Administration.

3. *Airport Design*, Advisory Circular 150/5300-13, Federal Aviation Administration.

If there is an airport within 2 miles of a proposed highway project, the Mississippi Department of Transportation Aeronautics Commission should be contacted to identify the class of airport and verify the current status of airway/highway clearance regulations.

8-6.0 BICYCLE ACCOMMODATION

8-6.01 General

The majority of bicycling needs will be met on public roads with little dedicated space for bicyclists. In rural areas, bicycling space will typically be the roadway shoulder. In urban areas, bicycling space may be provided by a shared roadway with wide curb lanes or dedicated space such as designated bicycle lanes. Separate bicycle facilities may be considered where bicyclists would become involved with high traffic volume

roadways. For pedestrian safety, sidewalks should not be considered as bicycle facilities except for child bicyclists along low-volume residential streets.

8-6.02 Bikeway Classifications

The following definitions apply to bikeway classifications:

1. Bikeway. Any road, path or way which in some manner is specifically designated as being open to bicycle travel, regardless of whether such facilities are designated for the exclusive use of bicycles or will be shared with other transportation modes.

2. Shared Roadway. Any roadway upon which a bicycle lane is not designated and which may be legally used by bicycles, regardless of whether the facility is specifically designated as a bikeway.

3. Bicycle Lane. A portion of a roadway which has been designated by striping, signing and pavement markings for the preferential or exclusive use of bicyclists. It is distinguished from the travel portion of the roadway by a physical or symbolic barrier.

4. Bicycle Path. A bikeway physically separated from motorized vehicular traffic by an open space or barrier and either within the highway right-of-way or within an independent right-of-way. Bicycle paths may be two-directional or, where the path parallels a roadway, a single lane on both sides of the road.

8-6.03 Selection Guidelines

Each type of bikeway facility has its own advantages and disadvantages. Care must be exercised in choosing the appropriate type of facility for a given site. The following guidelines assist in making decisions on bikeway facility selection. Each route is unique and must be evaluated individually.

8-6.03.01 Bicycle Paths

Bicycle paths are normally constructed explicitly for use by bicycles. The cyclist is provided with a clear-cut route and is protected from many hazardous conflicts. However, bicycle paths are extremely expensive to construct due to right-of-way and construction costs. The following guidelines may justify a bicycle path:

1. high vehicular speed on adjacent roadway;

2. high vehicular traffic volume on adjacent roadway;

3. high percentage of trucks on the adjacent roadway;

4. high bicycle traffic volume;

5. substantial anticipated increase in vehicular and/or bicycle traffic volumes;

6. absence of suitable alternative routes;

7. around schools, playgrounds, parks or other areas where children are expected;

8. demonstration that the facility will serve a definite purpose; and

9. reasonable indication that the bicycle path will be the safest and most economical method of providing a bicycle facility.

8-6.03.02 Bicycle Lanes

The occupation of a portion of a roadway by a bicycle lane implies a reasonable degree of safety for the cyclist. Conditions are generally less severe than those which suggest a bicycle path. Some sort of physical or symbolic barrier must be employed

to delineate the bicycle portion of the roadway. Commonly, this is a painted stripe on the roadway surface.

The cost of installing a bicycle lane is normally much less than that of a bicycle path. Another advantage of bicycle lanes is the relatively minor space requirements. They can be installed in many areas where the construction of paths would be impractical. In practice, bicycle lanes are often the most practical means of developing bikeways.

The following guidelines may justify a bicycle lane:

1. moderate to low vehicular speed on adjacent roadway;
2. moderate to low vehicular traffic volume on adjacent roadway;
3. moderate bicycle traffic volume;
4. anticipated increase in bicycle traffic volume;
5. insufficient land to construct bicycle paths without major disruptions to the surroundings;
6. demonstration that the facility will serve a definite purpose; and
7. an indication that the bicycle lane will be a safe and feasible method of providing a bicycle facility.

8-6.03.03 Shared Roadway

There are conditions where a shared roadway is the only practical method of establishing a bikeway. Because a shared roadway is designated only by bikeway signs, it is implied that the roadway provides safe conditions for both cyclist and motorist. Where some type of bikeway is warranted, shared roadways should be allowed only where the existing conditions either do not justify the greater expense of a higher type facility or prevent their installation.

The following guidelines may justify a shared roadway:

1. low vehicular speed on roadway;
2. low vehicular traffic on roadway;
3. low percentage of trucks on roadway;
4. moderate bicycle traffic volume;
5. anticipated increase in bicycle traffic volume;
6. demonstration that the facility would serve a definite purpose;
7. an indication that the shared roadway will be a safe and feasible method of providing a bicycle facility; and
8. a higher grade bikeway facility is not warranted.

8-6.04 Design

For the design criteria of bicycle facilities, refer to the AASHTO *Guide for the Development of Bicycle Facilities*. The *Guide* contains criteria on the design of bicycle facilities which includes, for example, railroad crossings, intersections, criteria for horizontal/vertical alignment, pavements, traffic control devices, etc. Specifically for the widths of bicycle facilities, the *Guide* recommends the following:

1. Bicycle Lanes. The minimum width is 4 ft; the desirable width is 5 ft.

2. Bicycle Paths. The minimum width for a one-directional path is 5 ft. For a two-directional path, the minimum width is 8 ft; the desirable width is 10 ft.

8-7.0 REFERENCES

1. *Guide for Transportation Landscape and Environmental Design*, AASHTO, 1991.

2. *Accessibility Guidelines for Building and Facilities*, U.S. Architectural and Transportation Barriers Compliance Board, 1991, 1994.

3. FHWA-IP-84-6, *Guidelines for Making Pedestrian Crossing Structures Accessible*, August, 1984.

4. FHWA-RD-84/082, *Warrants for Pedestrian Over and Underpasses*, 1984.

5. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 1994.

6. *Proposed Construction or Alteration of Objects that May Affect the Navigable Airspace*, Advisory Circular 70/7460-2I, Federal Aviation Administration.

7. *Objects Affecting Navigable Airspace*, Federal Aviation Regulations Part 77, Federal Aviation Administration.

8. *Airport Design*, Advisory Circular 150/5300-13, Federal Aviation Administration.

9. *Guide for the Development of Bicycle Facilities*, AASHTO, 1991.

Chapter 9

Roadside Safety and Traffic Control

MISSISSIPPI DESIGN MANUAL

Chapter 9

ROADSIDE SAFETY AND TRAFFIC CONTROL

Section		Page
9-1.0	DEFINITIONS/NOMENCLATURE	9-1
9-2.0	CLEAR ZONES	9-2
9-2.01	General	9-2
9-2.02	Fill Slopes (Uncurbed)	9-2
9-2.03	Cut Slopes (Uncurbed)	9-5
9-2.04	Curbs	9-7
9-2.05	Auxiliary Lanes	9-8
9-3.0	ROADSIDE BARRIER WARRANTS	9-8
9-3.01	Range of Treatments	9-8
9-3.02	Roadside Hazards	9-8
9-3.03	Embankments (Fill Sections)	9-9
9-3.04	Bridge Rails	9-9
9-3.05	Intersecting Slopes	9-9
9-3.06	Traffic Signals/Luminaire Supports/Utility Poles	9-10
9-4.0	ROADSIDE BARRIER TYPES	9-10
9-4.01	"W" Beam	9-12
9-4.02	Thrie Beam	9-12
9-4.03	Half-Section Concrete Median Barrier (CMB)	9-12
9-5.0	ROADSIDE BARRIER LAYOUT	9-12
9-5.01	Length of Need	9-12
9-5.02	Flare Rates	9-12
9-5.03	Lateral Placement	9-14
9-5.04	Barrier Gaps	9-14
9-5.05	Placement on Slopes and Behind Curbs	9-14
9-5.06	Terminal Treatments	9-15
9-5.06.01	General	9-15
9-6.0	ROADWAY/BRIDGE RAIL INTERACTION	9-15
9-6.01	Guardrail Transitions	9-15
9-6.01.01	Type	9-15
9-6.01.02	Length of Need	9-15
9-6.02	Sidewalks	9-18
9-6.02.01	Bridge Rail Location	9-18
9-6.02.02	Design Details	9-18
9-6.03	Pedestrian Rails	9-18
9-6.04	Bicycle Rails	9-18
9-7.0	MEDIAN BARRIERS	9-20
9-7.01	Warrants	9-20
9-7.01.01	Freeways	9-20
9-7.01.02	Non-Freeways	9-20
9-7.02	Types	9-20
9-7.02.01	"W" Beam	9-21
9-7.02.02	Thrie Beam	9-21
9-7.02.03	Concrete Median Barrier	9-21
9-7.03	Median Barrier Selection	9-23
9-7.04	Median Barrier Layout	9-23
9-7.04.01	Placement on Slopes	9-23
9-7.04.02	Divided Median Barriers	9-24
9-7.04.03	Barrier-Mounted Obstacles	9-24
9-7.04.04	Terminal Treatments	9-24
9-8.0	IMPACT ATTENUATORS	9-24
9-8.01	Warrants	9-24
9-8.02	Types	9-24
9-8.02.01	QuadGuard Family	9-24
9-8.02.02	REACT-350	9-25
9-8.02.03	Inertial System (Sand Barrels)	9-25
9-8.03	Impact Attenuator Selection	9-25
9-8.04	Impact Attenuator Design	9-25
9-8.04.01	Deceleration	9-25
9-8.04.02	Side Impacts	9-27
9-8.04.03	Placement	9-27
9-9.0	TRAFFIC CONTROL CRITERIA	9-27
9-9.01	Permanent Traffic Control	9-27
9-9.01.01	Criteria for No-Passing Zones	9-27
9-9.01.02	Pavement Markings	9-28
9-9.01.03	Signing	9-38
9-9.01.04	Traffic Signals	9-38
9-9.01.05	Lighting	9-39
9-9.02	Temporary Traffic Control in Construction Zones	9-39
9-9.02.01	General	9-39
9-9.02.02	Traffic Control Plans (TCP)	9-39
9-9.02.03	Geometric Design	9-40
9-9.02.04	Roadside Safety	9-41
9-9.02.05	Capacity	9-42
9-9.02.06	Two-Way Traffic on Divided Highways	9-42
9-9.02.07	Detours/Diversions	9-42
9-9.02.08	Temporary Crossovers	9-43
9-9.02.09	Bridge Replacement Projects	9-43
9-9.02.10	Temporary Pavement Markings	9-44
9-10.0	REFERENCES	9-44

Chapter 9

ROADSIDE SAFETY AND TRAFFIC CONTROL

9-1.0 DEFINITIONS/NOMENCLATURE

1. Barrier Warrant. A criterion that identifies an area of concern which should be shielded by a traffic barrier, if judged to be practical.

2. Critical Parallel Slope. Slopes which cannot be safely traversed by a run-off-the-road vehicle. Depending on the encroachment conditions, a vehicle on a critical slope may overturn. For most embankment heights, fill slopes steeper than 3:1 are considered critical.

3. Impact Angle. For a longitudinal barrier, the angle between a tangent to the face of the barrier and a tangent to the vehicle's path at impact. For a crash cushion, it is the angle between the axis of symmetry of the crash cushion and a tangent to the vehicular path at impact.

4. Impact Attenuator (Crash Cushion). A traffic barrier used to safely shield fixed objects or other hazards of limited dimension from approximately head-on impacts by errant vehicles.

5. Length of Need. Total length of a longitudinal barrier, measured with respect to the centerline of roadway, needed to shield an area of concern. The length of need is measured to the last point of full-strength rail. See Figure 9-5A.

6. Median Barrier. A longitudinal barrier used to prevent an errant vehicle from crossing the median of a divided highway. This prevents collisions between traffic traveling in opposite directions.

7. Non-Recoverable Parallel Slope. Slopes which can be safely traversed but upon which an errant motorist is unlikely to recover. The run-off-the-road vehicle will likely continue down the slope and reach its toe. If a fill slope is between 3:1 (inclusive) and 4:1 (exclusive), it is considered a non-recoverable parallel slope.

8. Parallel Slopes. Cut and fill slopes for which the toe runs approximately parallel to the flow of traffic.

9. Recoverable Parallel Slope. Slopes which can be safely traversed and upon which an errant motorist has a reasonable opportunity to stop and return to the roadway. Fill slopes 4:1 and flatter are considered recoverable.

10. Roadside Barrier. A longitudinal barrier used to shield hazards located within an established clear zone. Roadside barriers include guardrail, half-section concrete median barriers, etc.

11. Roadside Clear Zones. The distance beyond the edge of traveled way that should be clear of any non-traversable hazards or fixed objects.

12. Roadside Hazards. A general term to describe roadside features which cannot be safely impacted by a run-off-the-road vehicle. Roadside hazards include both fixed objects and non-traversable roadside features (see Section 9-3.02).

13. Shy Distance. The distance from the edge of traveled way beyond which a roadside object will not be perceived as an immediate hazard by the typical driver to the extent that he will change vehicular placement or speed.

14. Transverse (or Intersecting) Slopes. Cut and fill slopes for which the toe runs approximately perpendicular to the flow of traffic. Transverse slopes or intersecting slopes are typically formed by intersections between the mainline and approach, median crossovers or side roads.

15. Traversable Slope. A slope or cross section in which a vehicle can safely cross. Parallel slopes 3:1 or flatter are considered traversable.

9-2.0 CLEAR ZONES

9-2.01 General

The clear zone widths presented in this *Manual* must be placed in proper perspective. The distances imply a degree of accuracy that does not exist. They do, however, provide a good frame of reference for making decisions on providing a safe roadside area. Each application of the clear zone distance must be evaluated individually, and the designer must exercise good judgment. In general, the designer should provide as much clear zone as can be obtained practically.

When using the recommended clear zone distances, the designer should consider the following:

1. Project Scope of Work. The clear zone distances in Table 9-2A apply to all freeway projects and to new construction/reconstruction projects on non-freeways. Chapter 11 presents the clear zone criteria for 3R projects on non-freeways.

2. Context. If a formidable obstacle lies just beyond the zone, it may be appropriate to remove or shield the obstacle if costs are reasonable. Conversely, there may be cases where the clear zone should not be achieved at all costs. Limited right-of-way or unacceptable construction costs may lead to installation of a barrier or perhaps no protection at all.

3. Boundaries. The designer should not use the clear zone distances as boundaries for introducing roadside hazards such as bridge piers, non-breakaway sign supports, utility poles or landscaping features. These should be placed as far from the roadway as practical.

4. Roadside Cross Section. The recommended clear zone distance will be based on the type of roadside cross section. Sections 9-2.02 and 9-2.03 present a schematic for the various sections.

5. Highway Design Adjustments. The recommended clear zone distance should be adjusted for the highway design speed, roadside cross section, traffic volumes and the presence/absence of curbs.

6. Measurement. All clear zone distances are measured from the edge of traveled way, except where an auxiliary lane is present. See Section 9-2.05.

9-2.02 Fill Slopes (Uncurbed)

Table 9-2A presents the Department's criteria for clear zones on fill slopes which run parallel to the highway on uncurbed facilities. Figure 9-2A illustrates the types of fill slopes. The following discusses the application of the table:

1. Recoverable Fill Slopes. For parallel fill slopes 4:1 and flatter (Figure 9-2A(a)), the recommended clear zone distance can be determined directly from Table 9-2A.

Table 9-2A

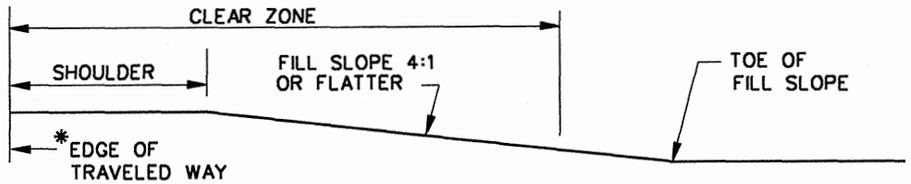
RECOMMENDED CLEAR ZONE DISTANCES (In Feet)
(Fill Slopes/Uncurbed Facilities)

Design Speed (mph)	Design ADT	Fill Slopes		
		Recoverable (Figure 9-2A(a))		Non-recoverable (Figure 9-2A(b)) (Steeper than 4:1 and 3:1 or flatter)
		6:1 or Flatter	5:1 to 4:1	
40 or Less	Under 750	7-10	7-10	SEE PROCEDURE IN SECTION 9-2.02.
	750 - 1500	10-12	12-14	
	1501 - 6000	12-14	14-16	
	Over 6000	14-16	16-18	
45 - 50	Under 750	10-12	12-14	
	750 - 1500	14-16	16-20	
	1501 - 6000	16-18	20-26	
	Over 6000	20-22	24-28	
55	Under 750	12-14	14-18	
	750 - 1500	16-18	20-24	
	1501 - 6000	20-22	24-30	
	Over 6000	22-24	26-32*	
60	Under 750	16-18	20-24	
	750 - 1500	20-24	26-32*	
	1501 - 6000	26-30	32-40*	
	Over 6000	30-32*	36-44*	
65 - 70	Under 750	18-20	20-26	
	750 - 1500	24-26	28-36*	
	1501 - 6000	28-32*	34-42*	
	Over 6000	30-34*	38-46*	

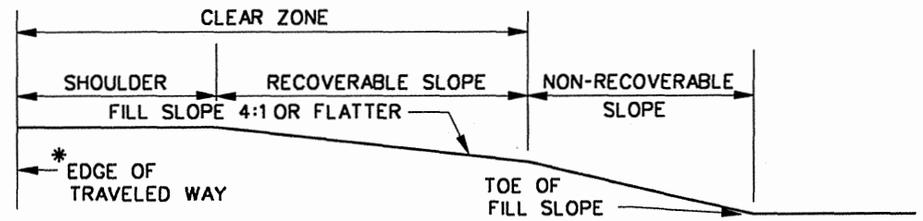
** The clear zone distance may be limited to 30 ft for practicality and to provide a consistent roadway template.*

Notes:

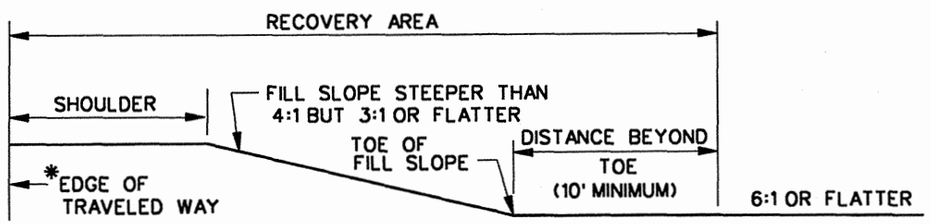
1. *This table applies to all freeways and new construction/reconstruction projects on non-freeways.*
2. *All distances are measured from the edge of traveled way or from the edge of an auxiliary lane, if present.*
3. *The "Design ADT" will be the total ADT for both directions of travel for the design year used for the project. This applies to both divided and undivided facilities.*
4. *On interchange ramps, the clear zone on each side of the ramp will be equal to the clear zone on the highway mainline.*



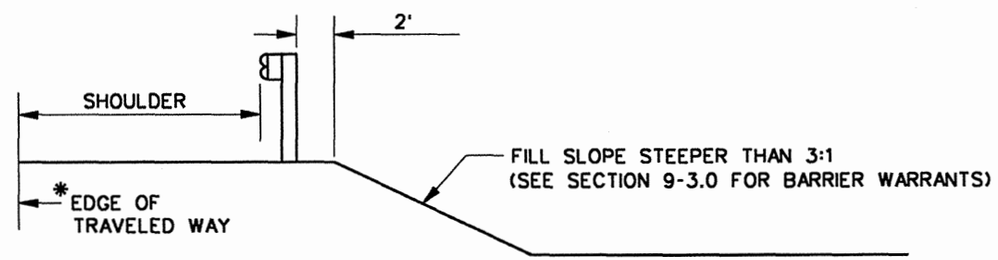
RECOVERABLE PARALLEL FILL SLOPE (a)



BARN-ROOF PARALLEL FILL SLOPE (c)
(RECOVERABLE/NON-RECOVERABLE)



NON-RECOVERABLE PARALLEL FILL SLOPE (b)



CRITICAL PARALLEL FILL SLOPE (d)

* IN SOME CASES, THE EDGE OF THE AUXILIARY LANE. SEE SECTION 9-2.05

CLEAR ZONE APPLICATION FOR PARALLEL FILL SLOPES
(Uncurbed)
Figure 9-2A

2. Non-Recoverable Fill Slopes. For parallel fill slopes between 3:1 (inclusive) and 4:1 (exclusive) (Figure 9-2A(b)), the recommended recovery area includes a distance beyond the toe of the slope which replaces/compensates for the clear zone distance not provided along the non-recoverable fill slope. Where the clear zone requires this additional recovery area beyond the toe of the slope, sufficient right-of-way must be purchased to ensure that the area can be maintained and cleared of hazards for the required clear zone distance. The following procedure is used to determine the clear zone involving non-recoverable fill slopes:

   a. The slope beyond the toe of the non-recoverable fill slope must be 6:1 or flatter. Determine the clear zone distance for the 6:1 or flatter slope from Table 9-2A for the applicable design speed and traffic volume.

   b. To determine the recommended distance beyond the toe, subtract the distance from the edge of traveled way to the slope break from the clear zone distance determined in Step #2a.

   c. If the distance in Step #2b is greater than or equal to 10 ft, the distance in Step #2b will be the clear zone beyond the toe. If the distance in Step #2b is less than 10 ft, the clear distance will be 10 ft beyond the toe.

3. Barn-Roof Fill Slope (Recoverable/Non-Recoverable). Barn-roof fill slopes may be designed with a recoverable slope (or safety slope) leading to a non-recoverable slope (Figure 9-2A(c)). The clear zone should desirably be provided entirely on the recoverable slope (i.e., the shoulder and recoverable slope should equal the clear zone distance). The slope after the break is typically 3:1. If the clear zone based on the recoverable slope extends beyond the slope break between the recoverable and non-recoverable slope, use the procedure in Comment #2 to determine the lateral extent of the clear recovery area.

4. Barn-Roof Fill Slope (Recoverable/Recoverable). Barn-roof fill slopes may be designed with two recoverable slope rates; the second slope is steeper than the slope adjacent to the shoulder. This design requires less right-of-way and embankment material than a continuous, flatter slope. Although a "weighted" average of the slopes may be used, a simple average of the clear zone distances for each slope is sufficiently accurate, if the variable slopes are approximately the same width. If one slope is significantly wider, the clear zone computation based on that slope alone may be used.

5. Critical Fill Slope. Fill slopes steeper than 3:1 are critical (Figure 9-2A(d)). These fill slopes typically require a barrier and, therefore, there is no clear zone application. See Section 9-3.0 for barrier warrants.

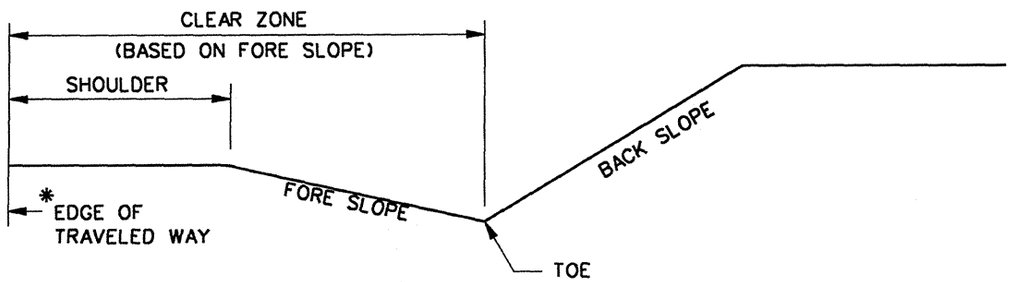
9-2.03 Cut Slopes (Uncurbed)

Ditch sections, as illustrated in Figure 9-2B, are typically constructed in roadside cuts without curbs. General Department policy is that the toe of the back slope will be located outside of the clear zone on new construction/reconstruction projects. Where this is not practical, the procedure in this Section will apply.

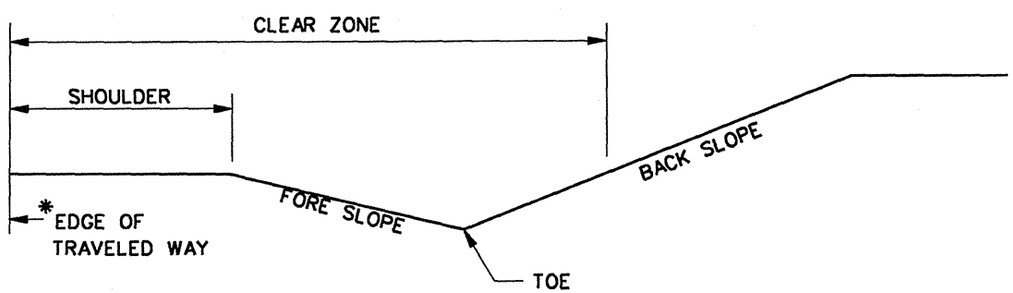
The applicable clear zone across a ditch section will depend upon the fore slope, the back slope, the horizontal location of the toe of the back slope, and various highway factors. The designer will use the following procedure to determine the recommended clear zone distance:

1. Check Fore Slope. Use Table 9-2A to determine the clear zone based on the ditch fore slope.

2. Check Location of the Toe of Back Slope. Based on the distance from Step #1, determine if the toe of the back slope is within the clear zone. If the toe is at or beyond the clear zone, then the designer usually need only consider roadside hazards within the clear zone on the fore slope or within the ditch. If the toe is within the clear zone, the designer should evaluate the practicality of relocating the toe of back slope. If the toe of back slope will remain within the clear zone, Step #3 will apply.



TOE OF BACK SLOPE NOT WITHIN CLEAR ZONE (a)



TOE OF BACK SLOPE WITHIN CLEAR ZONE (b)

* IN SOME CASES, THE EDGE OF THE AUXILIARY LANE. SEE SECTION 9-2.05

CLEAR ZONE APPLICATION FOR CUT SLOPES
(Uncurbed, Earth Section)

Figure 9-2B

3. Determine Clear Zone on Back Slope (Earth Cuts). If the toe of the back slope is within the clear zone distance from Step #1 above, provide a clear zone on the back slope. This clear zone will be a distance beyond the toe of back slope as follows:

   a. Calculate the percentage of the clear zone (from Step #1) available to the toe of the back slope.

   b. Subtract this percentage from 100% and multiply the results by the clear zone for the back slope in Table 9-2B.

   c. Add the available clear zone to the toe of the back slope to the value determined in Step #3b. This yields the required clear zone from the edge of traveled way to a point on the back slope.

4. Clear Zones (Rock Cuts). In rock cuts with steep back slopes, no clear zone is required beyond the toe of back slope. The rock cut should be relatively smooth to minimize the hazards of vehicular impact. If the face of the rock is rough or rock debris occurs in the ditch section, a barrier may be warranted.

Example 9-2.1 (Earth Ditch Section)

Given: ADT = 7000
V = 65 mph
Fore Slope = 6:1
Back Slope = 3:1
Toe of back slope is 20 ft from edge of traveled way.

Problem: Determine the clear zone application across the ditch section.

Solution: Using the procedure in Section 9-2.03:

Table 9-2B

RECOMMENDED CLEAR ZONE DISTANCES (In Feet)
(Back Slopes In Earth Cuts)

Design Speed (mph)	Design ADT*	Back Slopes		
		3:1	5:1 to 4:1	6:1 or Flatter
40 or Less	Under 750	7 - 10	7 - 10	7 - 10
	750 - 1500	10 - 12	10 - 12	10 - 12
	1501 - 6000	12 - 14	12 - 14	12 - 14
	Over 6000	14 - 16	14 - 16	14 - 16
45-50	Under 750	8 - 10	8 - 10	10 - 12
	750 - 1500	10 - 12	12 - 14	14 - 16
	1501 - 6000	12 - 14	14 - 16	16 - 18
	Over 6000	14 - 16	18 - 20	20 - 22
55	Under 750	8 - 10	10 - 12	10 - 12
	750 - 1500	10 - 12	14 - 16	16 - 18
	1501 - 6000	14 - 16	16 - 18	20 - 22
	Over 6000	16 - 18	20 - 22	22 - 24
60	Under 750	10 - 12	12 -14	14 - 16
	750 - 1500	12 - 14	16 - 18	20 - 22
	1501 - 6000	14 - 18	18 - 22	24 - 26
	Over 6000	20 - 22	24 - 26	26 - 28
65 - 70	Under 750	10 - 12	14 - 16	14 - 16
	750 - 1500	12 - 16	18 -20	20 - 22
	1501 - 6000	16 - 20	22 - 24	26 - 28
	Over 6000	22 - 24	26 - 30	28 - 30

* *The "Design ADT" will be the total ADT for both directions of travel for the design year used for the project. This applies to both divided and undivided facilities.*

1. Check Fore Slope. Table 9-2A yields a minimum clear zone of 30 ft for a 6:1 fore slope.

2. Check Location of Toe of Back Slope. The toe of back slope is within the clear zone. Therefore, Step #3 applies.

3. Determine Clear Zone on Back Slope. Using the procedure in Step #3:

   a. The percentage of the clear zone available to the toe of back slope is 20/30 = 67%.

   b. Subtracting this percentage from 100% yields: 100 - 67 = 33%. Table 9-2B yields a minimum clear zone on a 3:1 back slope of 22 ft. Multiplying this by 33% yields: (22) (0.33) ≈ 7 ft.

   c. Adding 7 ft to 20 ft yields 27 ft. Therefore, the total clear zone is 27 ft from the edge of traveled way or 7 ft up the 3:1 back slope.

9-2.04 Curbs

The values in Table 9-2A apply to uncurbed sections of highway. Where curbs are present, the following will apply:

1. Urban Facilities. A minimum horizontal, obstruction-free clearance of 1.5 ft should be provided as measured from the face of the curb to the edge of the obstruction. This applies to both barrier and mountable curbs, and it applies regardless of any shoulder width, auxiliary lane or parking lane. If practical, the designer should provide the clear zone values for uncurbed facilities.

2. Rural Facilities. For specific field conditions, it may be acceptable to use mountable curbs on rural facilities. However, the clear zone will be determined

assuming that the facility is uncurbed; i.e., the clear zone criteria presented in Table 9-2A will apply to all rural facilities whether curbed or uncurbed.

9-2.05 Auxiliary Lanes

For auxiliary lanes such as climbing lanes and passing lanes, the clear zone will be measured from the edge of the auxiliary lane; where turning lanes are located at intersections, the clear zone will be measured from the edge of the traveled way. However, in urban areas where the 1.5 ft horizontal clearance applies, this distance will be measured from the face of the curb whether or not any type of auxiliary lane is present.

9-3.0 ROADSIDE BARRIER WARRANTS

9-3.01 Range of Treatments

If a roadside hazard is within the clear zone, the designer should select the treatment which is judged to be the most practical and cost-effective for the site conditions. The range of treatments listed in order of preference include:

1. eliminate the hazard (flatten embankment, remove rock outcroppings, etc.);
2. relocate the hazard to outside the clear zone;
3. where applicable, make the hazard breakaway (sign posts, luminaire supports);
4. shield the hazard with a roadside barrier; or
5. do nothing.

The selected treatment will be based upon the traffic volumes, roadway geometry, proximity of the hazard to the traveled way, nature of the hazard, costs for remedial action, accident experience and judgment. If removal, relocation or modification of the hazard is not practical, a barrier should be installed only if engineering judgment indicates it is a reasonable solution. For example, it would probably not be practical to install a barrier to shield an isolated point obstacle, such as a tree, located near the outside edge of the clear zone.

9-3.02 Roadside Hazards

Table 9-2A presents the recommended clear zone distances for various highway conditions. These distances should be free of any fixed objects and non-traversable hazards based on the relative severity between impacting a barrier and impacting the hazard. Roadside hazards include:

1. non-breakaway sign and luminaire supports;
2. concrete bases extending more than 4 in. above the ground;
3. bridge piers and abutments at underpasses;
4. retaining walls;
5. trees with diameter greater than 4 in. (at maturity);
6. rough rock cuts;
7. large boulders;
8. streams or permanent bodies of water;
9. cross culverts with single round pipes with diameters greater than 36 in. or with multiple round pipes each with diameters greater than 30 in.;
10. parallel culverts with single round pipes with diameters greater than 24 in.;

11. protruding concrete headwalls on cross and side culverts, regardless of size of opening;

12. stone fences;

13. utility poles and other wood poles with cross sectional area greater than 50 in.^{2}; and

14. mailboxes.

9-3.03 Embankments (Fill Sections)

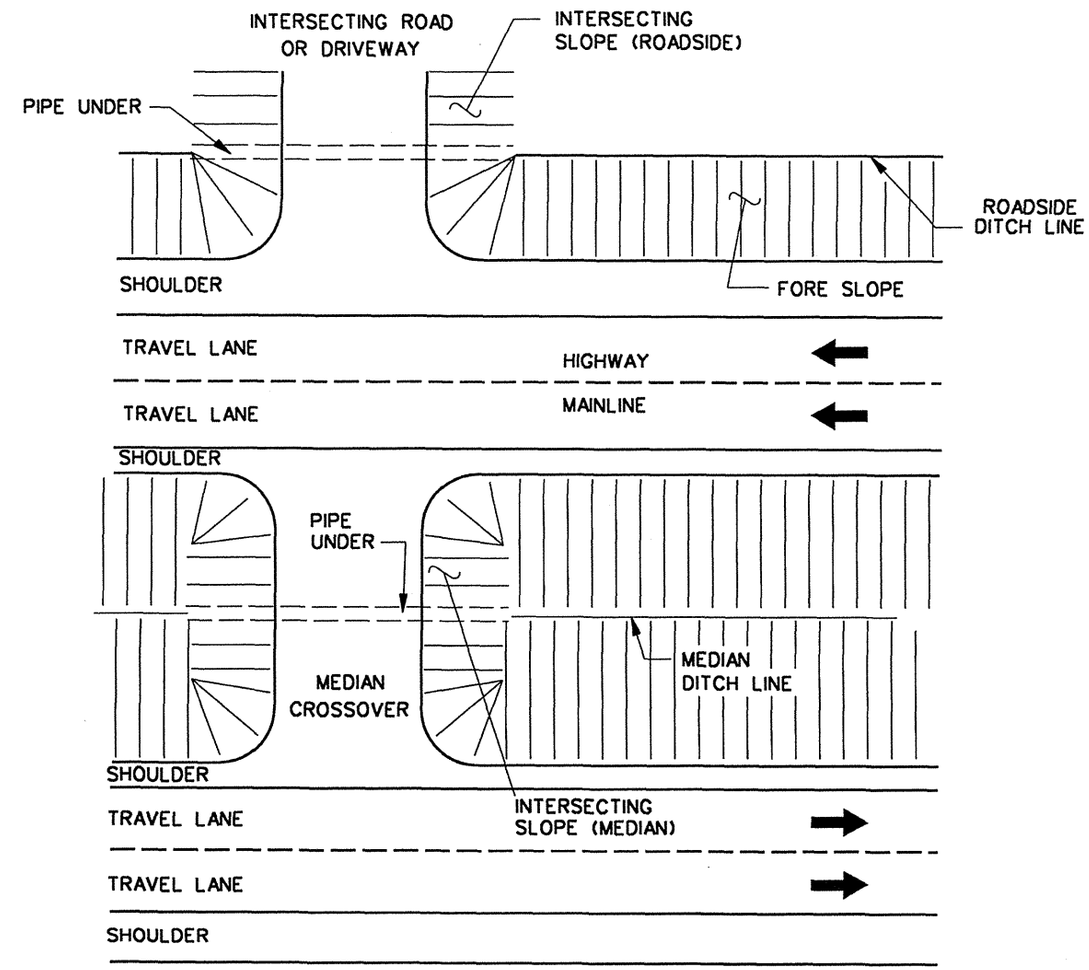
The severity of the roadside condition depends upon the rate of the fill slope. A barrier is warranted on all fill slopes steeper than 3:1. Guardrail is not required on fill slopes 3:1 or flatter, unless there are roadside hazards within the clear zone as calculated from Table 9-2A.

9-3.04 Bridge Rails

Barrier protection is normally warranted on all approach ends to bridge rails or parapets. No roadside barrier is needed on the trailing end of a one-way bridge, unless a barrier is warranted for other reasons (e.g., fill slope steeper than 3:1). See Section 9-6.0 for information on the Department's barrier transitions into bridge rails.

9-3.05 Intersecting Slopes

A highway mainline may intersect a driveway, side road or median crossing. This will present a slope which may be impacted at approximately a 90-degree angle by run-off-the-road vehicles from the mainline. See Figure 9-3A. Even for relatively flat side slopes, this may result in vehicular vaulting; for steeper slopes the vehicle's bumper



INTERSECTING SLOPES
Figure 9-3A

may "catch" in the intersecting slope resulting in an abrupt stop and undesirable occupant reactions.

For these reasons, intersecting slopes should be as flat as practical. The following criteria will apply:

1. Medians. In the median of all multilane highways, the maximum intersecting slope should be 10:1. This includes the sides of any drainage pipes which pass beneath, for example, a median crossover.

2. Rural Facilities (V ≥ 50 mph). For other than the median, the maximum intersecting slope is 6:1 within the clear zone. The desirable slope is 10:1.

3. Rural Facilities (V ≤ 45 mph). The maximum intersecting slope is 3:1 within the clear zone.

4. Urban Facilities (All Design Speeds). The maximum intersecting slope is 3:1 within the clear zone.

Parallel drainage structures may be closely spaced in urban areas because of frequent driveways and intersecting roads. In such locations, it may be desirable to convert the open ditch into a closed drainage design and backfill the areas between adjacent driveways. This treatment will eliminate the ditch section and the intersecting embankments with pipe inlets and outlets.

9-3.06 Traffic Signals/Luminaire Supports/Utility Poles

In general, the designer has limited options when determining acceptable locations for the placement of traffic signal pedestals and poles, controllers, luminaire supports and utility poles, especially in urban areas. Collectively, these are referred to as roadside appurtenances. Considering roadside safety, these appurtenances should be placed as far from the roadway as practical. However, due to visibility requirements, limited mast-arm lengths, limited right-of-way, restrictive geometrics and/or pedestrian requirements, these roadside appurtenances often must be placed relatively close to the traveled way. The designer should consider the following when determining their placement:

1. Urban Curbed Facilities. On urban curbed facilities, the desirable clear zone is 10 ft from the face of the curb to the appurtenance. The minimum offset between the face of the curb and appurtenance should be 1.5 ft.

2. Open Shoulder (Urban and Rural) Facilities; Rural Curbed Facilities. The clear zone criteria in Table 9-2A is desirable for the placement of traffic signals, luminaire supports and utility poles in rural areas. The minimum offset between the edge of the traveled way and the appurtenance is the shoulder width.

3. Channelized Islands. If practical, the designer should avoid placing any roadside appurtenances in channelized islands regardless of the attainable offset.

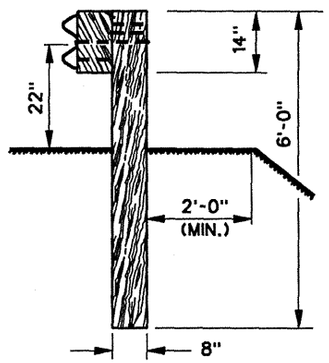
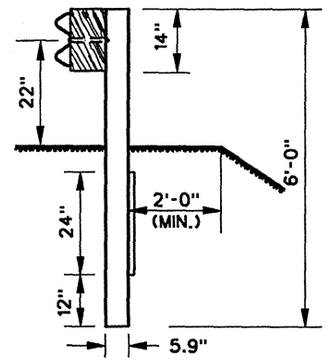
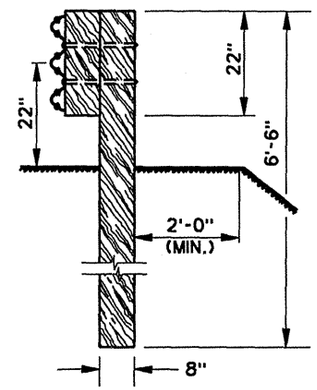
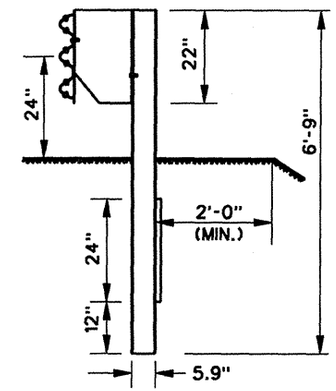
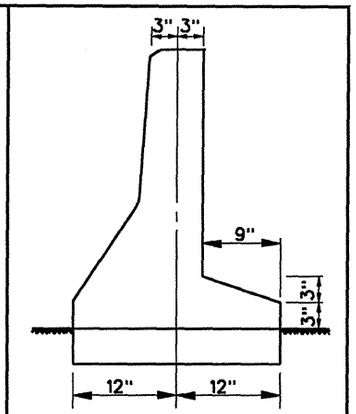
4. Barrier Warrant. Although typically not cost effective, the designer may use a roadside barrier to shield traffic signals, luminaire supports or utility poles where their placement does not meet the criteria in the preceding comments.

9-4.0 ROADSIDE BARRIER TYPES

As of August 1998, FHWA has mandated that all roadside safety hardware used on the National Highway System must meet the performance crash testing criteria in NCHRP 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features*. This applies to roadside barriers, impact attenuators, end treatments, bridge rails and guardrail-to-bridge-rail transitions. On all new construction/reconstruction projects, the roadside safety hardware will be 350-certified whether the project is on or off the NHS.

Table 9-4A presents the types of roadside barriers which are approved for use in Mississippi. The table summarizes the hardware requirements for each system; see

Table 9-4A
ROADSIDE BARRIERS

					
System	"W" Beam Guardrail (Wood Posts)	"W" Beam Guardrail (Steel Posts)	Thrie Beam Guardrail (Wood Posts)	Modified Thrie Beam Guardrail (Steel Posts)	Half-Section Concrete Median Barrier
AASHTO Designation	G4 (2W)	G4 (1S)	None	G9	—
Post Spacing	6.25 ft	6.25 ft	6.25 ft	6.25 ft	N/A
Deflection Distance*	3 ft	3 ft	2.25 ft	2.25 ft	0
Post Type	6″ x 8″Treated Wood Posts	W6 x 9.0 Steel Post	6″ x 8″ Treated Wood Posts	W6 x 9.0 Steel Post	N/A
Beam Type (Galvanized Thickness)	Steel "W" Section, (12 gauge)	Steel "W" Section, (12 gauge)	Steel Thrie Beam (12 gauge)	Steel Thrie Beam, (12 gauge)	N/A
Offset Blockout	6″ x 8″ x 14″ Treated Blockout	6″ x 8″ x 14″ Treated Blockout	6″ x 8″ x 22″ Treated Blockout	W6 x 9.0, 22″ Long Modified Steel Blockout	N/A

** Based on a full-size passenger car at a 60 mph impact speed and 25° angle of impact.*

the Mississippi *Standard Roadway Design Drawings* for the detailed design. The following sections briefly describe each system and its typical usage.

9-4.01 "W" Beam

The "W" beam system with heavy posts is a semi-rigid system. In general, this guardrail is most often the preferred selection. The Department has approved the use of two guardrail "W" beam systems. The AASHTO G4(2W) uses 6 in. x 8 in. timber posts, and the AASHTO G4(1S) uses W6 x 9.0 steel posts. Both systems use a wood blockout. The two systems are usually presented on a competitive bid basis in the contract proposal.

9-4.02 Thrie Beam

The thrie beam guardrail is also a semi-rigid system on heavy posts. The deeper longitudinal member of the thrie beam decreases the probability of an impacting vehicle overriding or underriding the guardrail. In addition, the thrie beam performs somewhat better than the "W" beam when impacted by a bus or large truck. Therefore, typical sites for installing the thrie beam include its use as a transition into rigid obstacles (e.g., bridge parapets) or as a barrier between the traveled way and a fixed object where the deflection distance is limited (e.g., bridge piers).

9-4.03 Half-Section Concrete Median Barrier (CMB)

The half-section CMB should be considered on the roadside around rigid objects where no deflection distance is available. If the rigid object is not continuous (e.g., bridge piers), special back-up posts may be necessary to support the half-section CMB. Fill material is also acceptable. In addition, the transition rail leading into the CMB must be crashworthy, or an impact attenuator should shield the end of the CMB.

9-5.0 ROADSIDE BARRIER LAYOUT

9-5.01 Length of Need

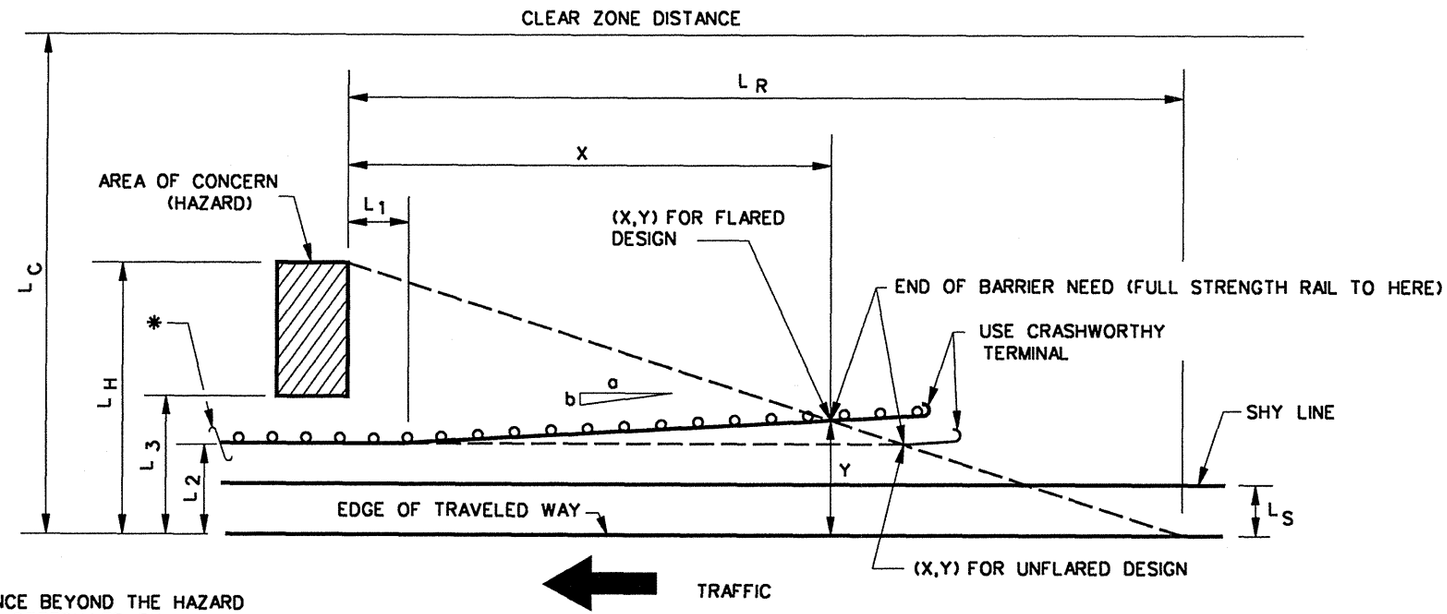
A roadside barrier must be extended a distance before the hazard to safely protect a run-off-the-road vehicle. Otherwise, the vehicle could travel behind the barrier and impact the hazard. The designer should recognize that vehicles depart the road at relatively flat angles. Based on a number of field studies, the average angle of departure is estimated to be 10°. The 80th percentile is estimated to be 15°. These flat angles of departure result in the need to extend the barrier a significant distance in front of the hazards.

Many factors combine to determine the appropriate length of need for a given roadside condition. Figure 9-5A provides the information needed to calculate the barrier length of need. Example 9-5.1 illustrates how to perform the calculations.

The Department has adopted explicit criteria for the length of need application to guardrail transitions into bridge rails. See Section 9-6.0.

9-5.02 Flare Rates

It will sometimes be necessary to laterally transition a run of barrier along the roadside. This may be necessary at the terminal section or along the main section of the barrier to, for example, meet a bridge parapet. The table in Figure 9-5A presents the Department's criteria for maximum flare rates. These are based on the design speed, the location of the barrier with respect to the shy line and the type of roadside barrier. The designer should provide flatter flare rates where practical.



*THE DISTANCE BEYOND THE HAZARD SHOULD BE DETERMINED BY A LENGTH-OF-NEED CALCULATION FOR OPPOSING TRAFFIC, IF APPLICABLE.

Equations to determine end of barrier need:

Flared Design

$$X = \frac{L_H + \frac{b}{a}(L_1) - L_2}{\frac{b}{a} + \frac{L_H}{L_R}}$$

$$Y = L_H - \frac{L_H}{L_R}(X)$$

Unflared Design

$$X = \frac{L_R(L_H - L_2)}{L_H}$$

$$Y = L_2$$

Design Speed (mph)	Design Traffic Volume (ADT)				L_S Shy Line Offset (ft)	Steepest Flare Rates		
	Over 6000	2000-6000	800-2000	Under 800		Within Shy Distance	Outside Shy Distance	
	L_R(ft)	L_R(ft)	L_R(ft)	L_R(ft)			CMB	Guardrail
70	480	440	400	360	10.0	30:1	20:1	15:1
65	440	400	370	330	9.0	28:1	19:1	14:1
60	400	360	330	300	8.0	26:1	17:1	13:1
55	360	330	300	270	7.5	24:1	16:1	12:1
50	320	290	260	240	6.5	21:1	14:1	11:1
40	240	220	200	180	5.0	17:1	11:1	9:1
30	170	160	140	130	3.5	13:1	8:1	7:1

Definition of Terms:

X,Y = coordinates of end of barrier need
b/a = barrier flare (e.g., 15:1)
L_C = recommended clear zone
L_H = distance from edge of traveled way to back of hazard
L_S = shy line offset, or distance at which barrier is no longer perceived as an obstacle by driver
L_R = runout length (see Table)
L_1 = distance from hazard to where barrier flare begins
L_2 = distance from edge of traveled way to barrier
L_3 = distance from edge of traveled way to front of hazard. (L_3 - L_2) must equal or exceed deflection distance.

BARRIER LENGTH OF NEED
Figure 9-5A

Example 9-5.1

Given: Design Speed = 65 mph
ADT = 4000 (20-year projection)
L_H = 25 ft
L_2 = 10 ft
Flare Rate (a/b) = 0 (unflared design)

Problem: Determine the coordinates (X,Y) of the barrier length of need.

Solution: From the table in Figure 9-5A, L_R = 400 ft. Therefore:

$$X = \frac{L_R\,(L_H - L_2)}{L_H}$$

$$X = \frac{400\ (25 - 10)}{25}$$

$$X = 240\ ft$$

$$Y = L_2$$

$$Y = 10\ ft$$

Therefore, the barrier should be extended a distance of 240 ft in advance of the leading edge of the hazard.

Note: If a flare rate of 10:1 had been used in advance of the hazard (L_1 = 0), this would have reduced X to 92 ft. Y would then be 19.25 ft from the edge of traveled way. However, this design could only be used if a 10:1 or flatter slope is present between the barrier and traveled way. See Section 9-5.05.

9-5.03 Lateral Placement

Roadside barriers should be placed as far as practical from the edge of traveled way. These factors should be considered when determining barrier lateral placement:

1. Deflection. The dynamic deflection distance of the barrier, as measured from the back of the post, cannot be violated. See Table 9-4A.

2. Slope. The slope between the traveled way and barrier should not exceed 10:1. See Section 9-5.05.

3. Post Support. At a minimum, 2 ft should be provided between the back of the barrier post and the break (PI) in a fill slope. Longer posts or soil bearing plates may be necessary in weaker soils.

4. Shy Distance. Drivers tend to "shy" away from continuous longitudinal obstacles along the roadside, such as guardrail. Therefore, the minimum lateral barrier offset should be based on the shy distance criteria in the table in Figure 9-5A.

9-5.04 Barrier Gaps

The designer should avoid short gaps between two runs of roadside barrier. The additional end terminal presents extra expense and exposure for impact. Therefore, gaps of less than 200 ft should be connected, unless the break in the barrier is necessary for access (e.g., a driveway or maintenance operations).

9-5.05 Placement on Slopes and Behind Curbs

If a barrier is improperly located on slopes or behind curbs, an errant vehicle could impact the barrier too high or too low with undesirable results. Therefore, these criteria should apply:

1. Curbs. The use of a barrier in combination with a curb should be avoided. Where drainage requires the use of guardrail and a curb in combination, the face of the barrier should be flush with the face of the curb. In this case, the height of the barrier must be measured from the pavement surface.

2. Slopes. A barrier should not be placed on a cut or fill slope steeper than 10:1. This also applies to the area in front of the flared section of barrier, if used.

9-5.06 Terminal Treatments

9-5.06.01 General

The terminal treatment for the approaching end of roadside barriers must meet the current acceptable crash-test performance criteria. The Department has not designated specific terminal types for use. The project contract package will provide appropriate pay items for flared terminals and for unflared terminals, and the project plans will identify where each type is used. In general, Department policy is to use a flared terminal unless site conditions render this impractical; i.e., there is insufficient space for the flare. The contract package will also provide the current list of approved flared and unflared terminals for contractor selection.

The following applies to the terminal treatment for the trailing end of a roadside barrier:

1. Two-Way Roadways. If the trailing end is within the clear zone for the opposing direction of travel, the trailing end terminal must meet the same crashworthy requirements as the approaching end. If outside of the clear zone, it is not necessary to provide a crashworthy terminal on the trailing end. The Type I Anchorage system, as presented in the *Standard Drawings*, may be used at these locations.

2. One-Way Roadways. It is not necessary to provide a crashworthy terminal on the trailing end. The Type I Anchorage system, as presented in the *Standard Drawings*, may be used at these locations.

9-6.0 ROADWAY/BRIDGE RAIL INTERACTION

The Bridge Division is responsible for selecting the type of and design of the bridge rail. This Section discusses the roadway design implications of bridge rails. These design elements must be coordinated with the Bridge Division.

9-6.01 Guardrail Transitions

9-6.01.01 Types

The Mississippi *Standard Roadway Design Drawings* presents those guardrail-to-bridge -rail transitions which are acceptable for use on Department projects.

9-6.01.02 Length of Need

As with any roadside barrier, the guardrail-to-bridge-rail transition must be extended a sufficient distance upstream from the bridge rail to provide proper protection for run-off-the-road vehicles. The information in Figure 9-5A applies to this length of need. To simplify the application, the following will apply to bridge rail approach transitions:

1. Table 9-6A. This Table applies to both sides of an approach transition on a divided highway and to the right side of an approach on a two-way facility.

2. Table 9-6B. This Table applies to the left side of an approach on a two-way facility.

The variables "A," "B," "C" and "D" in the two Tables are defined on the applicable sheets in the Mississippi *Standard Roadway Design Drawings*.

Table 9-6A
LENGTH OF NEED
(Bridge Rail Approaches)

Design Speed (mph)	Traffic Volume (ADT)	Flare Rate (a:b)	Clear Zone ($L_H = L_C$)	Runout Length (L_R)(ft)	Tangent Section (L_1)	Assumed Shoulder Width (L_2)	End Terminal LON Contribution	"A"* (ft)	"B * (ft)	No. of 12.5-ft Sections for "B"
30	<800	30:1	30 ft	130	18′-1¾″	4 ft	25 ft	105.6458	62.5	5
	800-2000			140		6 ft		105.6458	62.5	5
	2000-6000			160		8 ft		105.6458	62.5	5
	>6000			170		8 ft		118.1458	75.0	6
40	<800	30:1	30 ft	180	18′-1¾″	4 ft	25 ft	143.1458	100.0	8
	800-2000			200		6 ft		143.1458	100.0	8
	2000-6000			220		8 ft		143.1458	100.0	8
	>6000			240		8 ft		143.1458	100.0	8
50	<800	30:1	30 ft	240	18′-1¾″	4 ft	25 ft	168.1458	125.0	10
	800-2000			260		6 ft		168.1458	125.0	10
	2000-6000			290		8 ft		168.1458	125.0	10
	>6000			320		8 ft		180.6458	137.5	11
55	<800	30:1	30 ft	270	18′-1¾″	4 ft	25 ft	193.1458	150.0	12
	800-2000			300		6 ft		193.1458	150.0	12
	2000-6000			330		8 ft		193.1458	150.0	12
	>6000			360		8 ft		205.6458	162.5	13
60	<800	30:1	30 ft	300	18′-1¾″	4 ft	25 ft	205.6458	162.5	13
	800-2000			330		6 ft		205.6458	162.5	13
	2000-6000			360		8 ft		205.6458	162.5	13
	>6000			400		10 ft		193.1458	150.0	12
65	<800	30:1	30 ft	330	18′-1¾″	10 ft	25 ft	168.1458	125.0	10
	800-2000			370		10 ft		180.6458	137.5	11
	2000-6000			400		10 ft		193.1458	150.0	12
	>6000			440		10 ft		205.6458	162.5	13
70	<800	30:1	30 ft	360	18′-1¾″	10 ft	25 ft	180.6458	137.5	11
	800-2000			400		10 ft		193.1458	150.0	12
	2000-6000			440		10 ft		205.6458	162.5	13
	>6000			480		10 ft		218.1458	175.0	14

** "A" and "B" in this table apply to both sides of a bridge approach on a divided highway and the right side of a bridge approach on a two-way facility.*

See Figure 9-5A for definition of terms. See Mississippi Standard Roadway Design Drawings *for application.*

Table 9-6B
LENGTH OF NEED
(Bridge Rail Approaches)

Design Speed (mph)	Traffic Volume (ADT)	Flare Rate (a:b)	Clear Zone ($L_H = L_C$)	Runout Length (L_R) (ft)	Tangent Section (L_1)	Assumed Lane Width [1]	Assumed Shoulder Width [1]	End Terminal LON Contribution	"C"* (ft)	"D"* (ft)	No. of 12.5-ft Sections for "D"
30	<800	30:1	30 ft	130	18′-1¾″	10 ft	4 ft	25 ft	68.1458	25.0	2
	800-2000			140		11 ft	6 ft		55.6458	12.5	1
	2000-6000			160		11 ft	8 ft		55.6458	12.5	1
	>6000			170		12 ft	8 ft		55.6458	12.5	1
40	<800	30:1	30 ft	180	18′-1¾″	11 ft	4 ft	25 ft	80.6458	37.5	3
	800-2000			200		11 ft	6 ft		80.6458	37.5	3
	2000-6000			220		11 ft	8 ft		80.6458	37.5	3
	>6000			240		12 ft	8 ft		68.1458	25.0	2
50	<800	30:1	30 ft	240	18′-1¾″	11 ft	4 ft	25 ft	105.6458	62.5	5
	800-2000			260		11 ft	6 ft		93.1458	50.0	4
	2000-6000			290		12 ft	8 ft		80.6458	37.5	3
	>6000			320		12 ft	8 ft		93.1458	50.0	4
55	<800	30:1	30 ft	270	18′-1¾″	12 ft	4 ft	25 ft	105.6458	62.5	5
	800-2000			300		12 ft	6 ft		105.6458	62.5	5
	2000-6000			330		12 ft	8 ft		93.1458	50.0	4
	>6000			360		12 ft	8 ft		93.1458	50.0	4
60	<800	30:1	30 ft	300	18′-1¾″	12 ft	4 ft	25 ft	118.1458	75.0	6
	800-2000			330		12 ft	6 ft		105.6458	62.5	5
	2000-6000			360		12 ft	8 ft		93.1458	50.0	4
	>6000			400		12 ft	10 ft		80.6458	37.5	3
65	<800	30:1	30 ft	330	18′-1¾″	12 ft	6 ft	25 ft	105.6458	62.5	5
	800-2000			370		12 ft	6 ft		118.1458	75.0	6
	2000-6000			400		12 ft	8 ft		105.6458	62.5	5
	>6000			440		12 ft	10 ft		93.1458	50.0	4
70	<800	30:1	30 ft	360	18′-1¾″	12 ft	6 ft	25 ft	118.1458	75.0	6
	800-2000			400		12 ft	6 ft		118.1458	75.0	6
	2000-6000			440		12 ft	8 ft		105.6458	62.5	5
	>6000			480		12 ft	10 ft		93.1458	50.0	4

* *"C" and "D" in this table apply to the left side of a bridge approach on a two-lane, two-way facility.*

[1] *L_2 = Lane width + Shoulder width. See Figure 9-5A for definition of terms. See Mississippi* Standard Roadway Design Drawings *for application.*

9-6.02 Sidewalks

The use of sidewalks approaching and/or crossing a bridge has several roadway and bridge rail design implications. These are discussed in the following Sections.

9-6.02.01 Bridge Rail Location

Where a sidewalk is present on a bridge, this will impact the location of the bridge rail. The potential problem is that, when a vehicle strikes a curb, it may become airborne. Depending upon the lateral offset of the bridge rail and the impact conditions, the vehicle may impact the rail while airborne and, thus, may interfere with the proper vehicle/bridge rail interaction.

The following will apply to the location of a bridge rail in combination with a sidewalk:

1. $V \leq 45$ mph. The bridge rail may be located at either the back of the sidewalk or between the roadway and the sidewalk. Use the 42-in. concrete median barrier bridge rail and the standard guardrail-to-bridge-rail transition.

2. $V \geq 50$ mph. It is not acceptable to place the bridge rail at the back of the sidewalk; it must be placed between the traveled way and the sidewalk. A pedestrian or bicycle rail will then be used at the back of the sidewalk. Where the vehicular bridge rail is placed on the inside of the sidewalk, the sidewalk should not be raised; i.e., the roadway surface and sidewalk surface should be at the same elevation. See Section 9-6.02.02 for the design of this arrangement.

9-6.02.02 Design Details

Where the bridge rail is located between the traveled way and sidewalk, Figure 9-6A presents a cross section and plan view of the recommended design.

9-6.03 Pedestrian Rails

If a sidewalk is placed on a bridge, it may be warranted to provide a bridge rail to separate the vehicular traffic from pedestrians and then use a pedestrian rail on the outside edge of the sidewalk. The following will apply:

1. $V \geq 50$ mph. As discussed in Section 9-6.02, it is not acceptable to place a bridge rail at the back of a sidewalk where the design speed is greater than or equal to 50 mph. Therefore, a separate pedestrian rail is required on all bridges with sidewalks on these facilities.

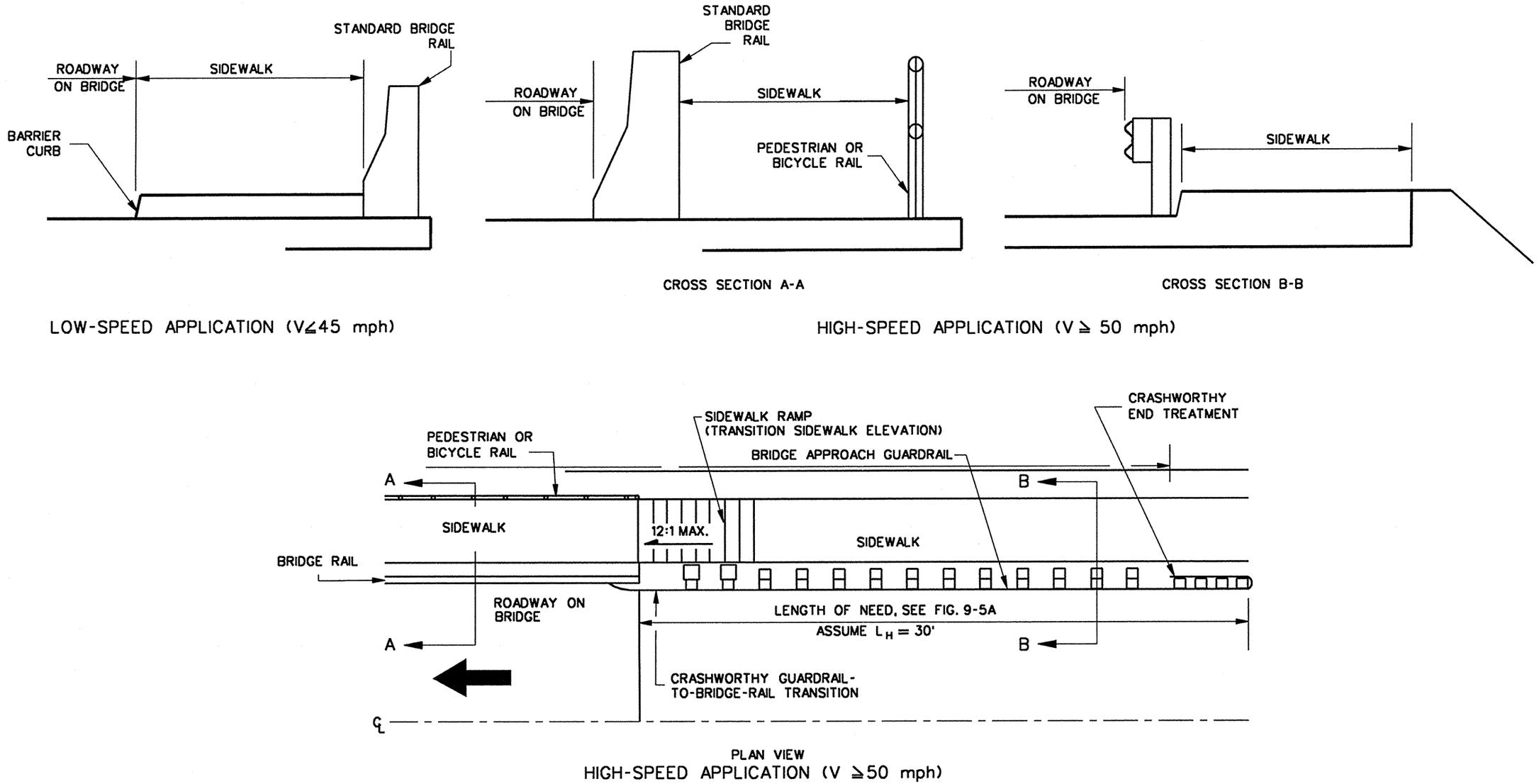
2. $V \leq 45$ mph. On facilities with sidewalks on bridges and where $V \leq 45$ mph, the need for special protection of pedestrians by use of a combination vehicular bridge rail/pedestrian rail will be considered on a case-by-case basis. The following factors will be evaluated:

   a. design speed;
   b. pedestrian volumes;
   c. vehicular traffic volumes;
   d. accident history;
   e. geometric impacts (e.g., sight distance);
   f. practicality of providing proper end treatments;
   g. construction costs; and
   h. local preference.

The AASHTO *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* discusses the geometric configuration and structural loading for pedestrian rails.

9-6.04 Bicycle Rails

If bicyclists are permitted to use a bridge, it may be warranted to provide a bicycle rail across the bridge. The following will apply:



BRIDGE RAIL/SIDEWALK DESIGNS
Figure 9-6A

`1. Bicycle Paths. These are defined as a bikeway physically separated from motorized vehicular traffic by an open space or barrier and either within the highway right-of-way or within an independent right-of-way. Bridges which are part of a bicycle path will require a bicycle rail.

2. Highway Facilities. On highway facilities where bicycles are legally allowed to use the roadway, it may be warranted to provide a bridge rail to separate the vehicular traffic from the bicyclists and then use a bicycle rail on the outside edge of the bridge. The need for a combination vehicular bridge rail/bicycle rail to protect bicyclists will be considered on a case-by-case basis. The following factors will be evaluated:

   a. design speed;
   b. bicycle volumes;
   c. vehicular traffic volumes;
   d. accident history;
   e. geometric impacts (e.g., sight distance);
   f. practicality of providing proper end treatments;
   g. construction costs; and
   h. local preference.

The AASHTO *LRFD Bridge Design Specifications* discuss the geometric configuration and structural loading for bicycle rails.

9-7.0 MEDIAN BARRIERS

9-7.01 Warrants

The following summarizes the Department's criteria.

9-7.01.01 Freeways

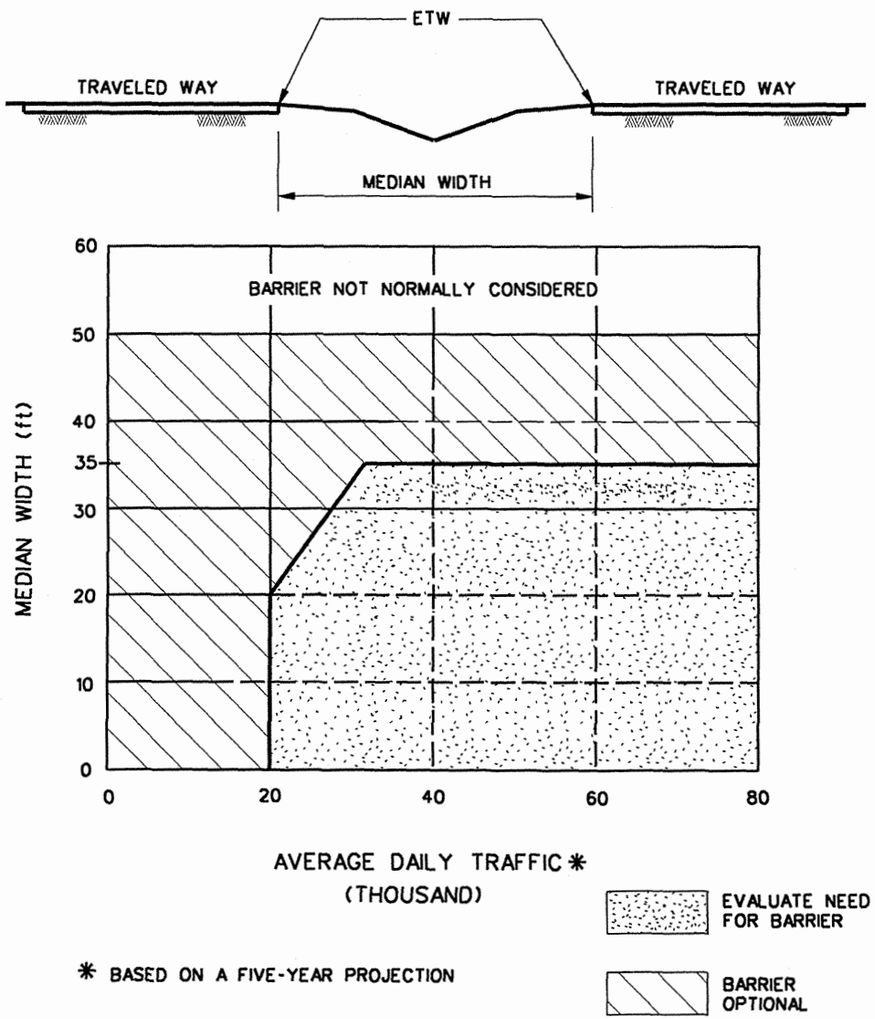
Figure 9-7A presents the warrants for a median barrier based on median width and traffic volumes. Note that the traffic volumes are based on a 5-year projection. In the areas shown as optional, the decision to use a median barrier will be based on construction and maintenance costs and crossover accident experience. A median barrier may also be warranted on wider medians if a significant number of crossover accidents have occurred.

9-7.01.02 Non-Freeways

On other highways, some judgment must be used to determine median barrier warrants. On highways without full access control, the median barrier must terminate at each at-grade intersection, which is undesirable. In addition, lower speeds may reduce the likelihood of a crossover accident. On non-freeway highways, the designer should evaluate the accident history, traffic volumes and speeds, median width, alignment, sight distance and construction costs to determine the need for a median barrier. Figure 9-7A may also be used as guidance.

9-7.02 Types

Table 9-7A presents the types of median barriers which are approved for use in Mississippi. The table summarizes the hardware requirements for each system; see the Mississippi *Standard Roadway Design Drawings* for more detailed information. The following sections briefly describe each system and its typical usage.



MEDIAN BARRIER WARRANTS
Figure 9-7A

9-7.02.01 "W" Beam

The ""W" beam median barrier with heavy posts is a semi-rigid system. Its performance is similar to the "W" beam guardrail system. This median barrier is most applicable to medians with intermediate width. A special application is for the separation of adjacent on/off ramps at interchanges.

The Department has approved the use of two "W" beam median barriers. The AASHTO MB (4W) uses 6″ x 8″ timber posts, and the AASHTO MB (4S) uses W6 x 9.0 steel posts. Both systems use a wood blockout. The two systems are usually presented on a competitive bid basis in the contract proposal.

9-7.02.02 Thrie Beam

The thrie beam median barrier on heavy posts is also a semi-rigid system. The posts may be timber or steel. Its performance is similar to the thrie beam guardrail system. Candidate sites for using the thrie beam median barrier are similar to those for the thrie beam guardrail.

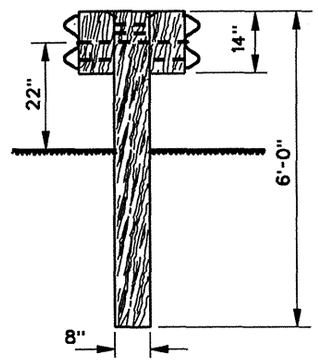
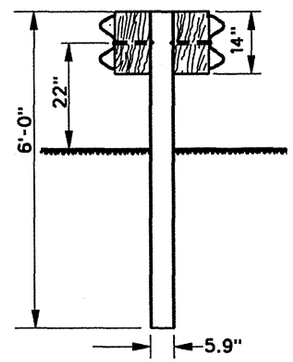
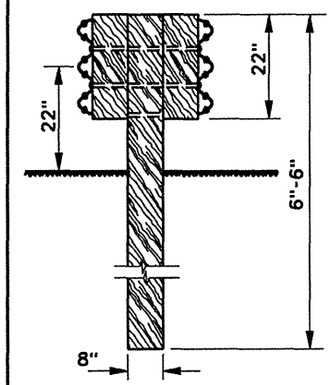
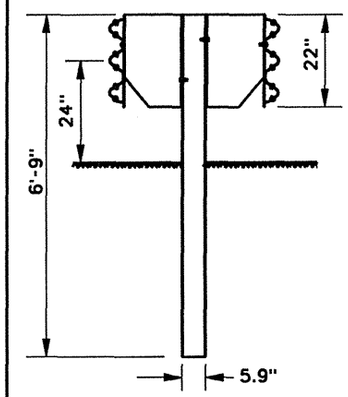
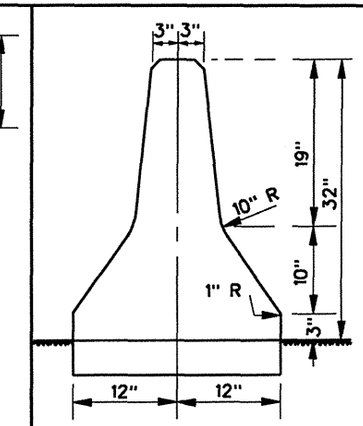
9-7.02.03 Concrete Median Barrier

The concrete median barrier (CMB) is a rigid system which does not deflect upon impact. A variation of the CMB is a half-section of the barrier system. These may be necessary where the median barrier must divide to go around a fixed object in the median (e.g., bridge piers).

The standard 32-in. CMB (Table 9-7A) may not successfully redirect heavy vehicles if the impact speed and angle are high. If the median barrier is penetrated by a heavy vehicle, this can lead to catastrophic results. Therefore, on some freeways, it may be warranted to install the 42-in. high CMB. The barrier maintains the standard safety shape of the 32-in. CMB, but it is extended an additional 10 in. However, this results in restricted vision for authorized vehicles (e.g., police) who wish to view the

Table 9-7A

MEDIAN BARRIERS

					
System	"W" Beam Median Barrier (Wood Posts)	"W" Beam Median Barrier (Steel Posts)	Thrie Beam Median Barrier (Wood Posts)	Modified Thrie Beam Median Barrier (Steel Posts)	Concrete Median Barrier
AASHTO Designation	MB4W	MB4S	None	MB9	MB5
Post Spacing	6.25 ft	6.25 ft	6.25 ft	6.25 ft	N/A
Deflection Distance*	3 ft	3 ft	2.25 ft	2.25 ft	0
Post Type	6″ x 8″ Treated Wood Post	W 6 x 9.0 Steel Post	6″ x 8″ Treated Wood Post	W6 x 9.0 Steel Post	N/A
Beam Type (Galvanized Thickness)	Steel "W" Section, (12 gauge)	Steel "W" Section, (12 gauge)	Steel Thrie Beam (12 gauge)	Steel Thrie Beam (12 gauge)	N/A
Offset Blockout	6″ x 8″ x 14″ Treated Blockout	6″ x 8″ 14″ Treated Blockout	6″ x 8″ x 22″ Treated Blockout	W6 x 9.0, 22″ Long Modified Steel Blockout	N/A

* *Based on a full-size passenger car at a 60 mph impact speed and 25° angle of impact.*

opposing lanes. If there has been an adverse history of heavy-vehicle crossover accidents, the tall CMB should be considered.

9-7.03 Median Barrier Selection

The Department has not adopted specific criteria for the selection of median barrier systems. This involves a subjective evaluation of the many trade-offs between systems. The designer should evaluate each of the following factors when selecting a median barrier:

1. Median Width. The median width will significantly affect the probability of impact (i.e., the number of hits) and the likely angles of impact. The former will influence maintenance costs; the latter influences safety. The greater the offset to the barrier, the higher the likely angle of impact. Specifically for the CMB, offsets of more than 12 ft should be avoided. Therefore, considering both maintenance and safety, this favors the use of the CMB on median widths up to about 25 ft and either the "W" beam or thrie beam system for wider medians.

2. Traffic Volumes. The higher the traffic volume, the higher the likely number of impacts on the median barrier. From a maintenance perspective, this favors the CMB; from a safety perspective, this favors the metal beam systems.

3. Heavy Vehicle Traffic. The CMB, especially the 42-in. high CMB, is more likely to restrain and redirect heavy vehicles (trucks and buses) than the metal beam systems. Therefore, where there is a higher volume of heavy vehicles, this favors the CMB even on medians wider than 25 ft. Between the two metal beam systems, the thrie beam performs somewhat better when impacted by heavy vehicles.

4. Costs. The initial cost of the CMB will exceed, perhaps by a wide margin, the initial cost of the two metal beam median barriers. However, the maintenance costs per impact on the CMB will probably be far less. Considering maintenance costs, then, favors the CMB in narrow medians and/or on high-volume highways.

5. Maintenance Operations. Two factors are important. First, maintenance response time will influence safety. The longer that a damaged section of median barrier is present, the greater the likelihood of a second impact. This observation favors the use of the CMB which normally sustains far less damage when impacted. Second, the maintenance operation of repairing damaged barrier can interrupt traffic operations. It is particularly undesirable to close a traffic lane to repair a barrier. This observation favors the use of the CMB in narrow medians and/or on high-volume highways.

9-7.04 Median Barrier Layout

Much of the information presented in Section 9-5.0 on roadside barrier layout also applies to median barriers. For example, Figure 9-5A presents the criteria for maximum flare rates which also apply to median barriers. The following sections present criteria specifically for the design of median barriers.

9-7.04.01 Placement on Slopes

A median barrier should not be placed where the roadside slope leading up to the barrier exceeds 10:1. For a CMB, this slope will probably be the shoulder slope, and this will not usually present a problem. However, depressed medians may present difficulties. Existing median slopes greater than 10:1 should be flattened, if practical.

9-7.04.02 Divided Median Barriers

It may be necessary to intermittently divide a median barrier. The slope criteria in Section 9-7.04.01 or a fixed object in the median may require a divided barrier. The median barrier may be divided by one of these methods:

1. A fixed object may be encased by a CMB.

2. A half-section CMB may be used on both sides to shield a fixed object.

3. A metal beam median barrier can be split into two separate runs of roadside barrier passing on either side of the median hazard (fixed object or slope).

If a median barrier is split, the design should adhere to the acceptable flare rates presented in Figure 9-5A.

9-7.04.03 Barrier-Mounted Obstacles

If trucks or buses impact the CMB, their high center of gravity may result in a vehicular roll angle which possibly will allow the truck or bus to impact obstacles on top of the CMB (e.g., luminaire supports). If practical, these devices should be moved to the outside of the roadway, or the designer should provide additional distance between the barrier and obstacle.

9-7.04.04 Terminal Treatments

Because of its adaptability to narrow obstacles, the QuadGuard is typically used at exposed ends of the CMB. It is also acceptable to use the REACT-350. See Section 9-8.0 for more information. An exception is where the end of the CMB is flared to terminate outside of the roadside clear zone.

9-8.0 IMPACT ATTENUATORS

Impact attenuators are protective systems that prevent errant vehicles from impacting hazards by either decelerating the vehicle to a stop after a frontal impact or by redirecting it away from the hazard after a side impact. They are adaptable to many roadside hazard locations where longitudinal barriers cannot practically be used. In addition, impact attenuators may prove to be a better selection than barriers because of their smaller target area.

9-8.01 Warrants

Warrants for impact attenuators are the same as barrier warrants. Once a hazard is identified, the designer should first attempt to remove, relocate or make the hazard breakaway. If the foregoing is impractical, then an impact attenuator should be considered. Impact attenuators are most often installed to shield fixed-point hazards. Examples include exit gore areas (particularly on structures), bridge piers, non-breakaway sign supports and median barrier ends.

9-8.02 Types

The following impact attenuators have been approved for use in Mississippi. The designer should note that each of these attenuators is patented. Other attenuators are available which meet the current FHWA performance criteria, and they may be used on a case-by-case basis.

9-8.02.01 QuadGuard Family

The QuadGuard family uses crushable, energy-absorbing cartridges surrounded by a steel framework to dissipate vehicular energy. A back-up structure and base pad are required. The QuadGuard's primary advantage is its adaptability to narrow obstacles where encroachments beyond the width of the obstacles cannot be tolerated. An

example is the exposed end of a concrete median barrier in a narrow median. It is available in widths from 2 ft to 7.5 ft. In addition, a portable Construction Zone QuadGuard has been developed which can be used without being anchored to a rigid back-up structure. See Section 9-9.0.

9-8.02.02 REACT-350

The REACT-350 is an array of plastic cylinders which dissipate vehicular energy upon impact. It is a redirective, non-gating system which can be used to shield a wide range of obstacle widths. The REACT-350 has a high percentage of reusability after impact.

9-8.02.03 Inertial System (Sand Barrels)

These systems are an array of sand-filled plastic modules. The sand barrels can be intermixed and arranged to accommodate various design speeds and obstacle widths. At some sites, it may be the only practical impact attenuator for very wide hazards (e.g., a freeway gore area which leads to a steep fill slope). The system requires no back-up support.

9-8.03 Impact Attenuator Selection

Table 9-8A lists the advantages and disadvantages of the impact attenuators used in Mississippi. The Department has not adopted specific criteria for the selection of an impact attenuator. This involves an evaluation of several factors, which are briefly discussed:

1. Site Conditions. The selected impact attenuator must be compatible with the specific site conditions of the hazard. These include factors such as the width of the hazard, the need for redirective capability, the adaptability of a back-up structure (if needed) and the space available for an attenuator.

2. Frequency of Impacts. The anticipated frequency of impacts on the attenuator is important. First, the greater the number of impacts, then the greater the demand on maintenance. This favors the REACT-350 and QuadGuard over the inertial system. Second, the REACT-350 and QuadGuard systems can better sustain a second impact if damaged than can the inertial system. Therefore, if there are likely to be many impacts at the site, the estimated exposure time of a damaged attenuator is a factor.

3. Costs. Typically, the initial cost of the inertial system is less than that of the REACT-350 or QuadGuard. However, the maintenance cost per hit is usually higher for the inertial system.

9-8.04 Impact Attenuator Design

Once an impact attenuator has been selected, the designer must ensure that its design is compatible with the traffic and physical conditions at the site. For the patented attenuators, the designer should coordinate with the manufacturer for all design features.

9-8.04.01 Deceleration

For impact attenuators, acceptable vehicular deceleration is determined by the criteria which has been adopted by FHWA. The manufacturer is responsible for ensuring that the system meets the current FHWA impact performance criteria.

An initial impact speed must be selected to design an attenuator. Table 9-8B presents the Department's criteria.

Table 9-8A

IMPACT ATTENUATOR TYPES
(Advantages and Disadvantages)

SYSTEM	ADVANTAGES	DISADVANTAGES
REACT-350	1. Little or no debris after hit. 2. Ease of maintenance after hit. 3. Some attenuation capacity after a hit. 4. Relatively low maintenance cost to repair after hit. 5. Coffin corner protection and side hit redirection. 6. Lightweight and therefore more suitable to structures.	1. High initial cost. 2. Considerable site preparation (e.g., back-up structure). 3. Width of protection limited by width of units available. Standard widths range from 3 ft to 10 ft.
QuadGuard	1.-6. Same as REACT-350. 7. Best for very narrow obstacles.	1. High initial cost. 2. Considerable site preparation (e.g., back-up structure). 3. Width of protection limited by width of units available. Standard widths from 2 ft to 7.5 ft.
Inertial	1. Relatively low initial cost. 2. Ease of installation. 3. Little or no site preparation required. 4. Versatile; can be used to cover a large area.	1. Considerable debris after a unit is hit requiring clean up. 2. Relatively high maintenance cost to repair after hits (damaged barrels must be replaced). 3. Generally no residual attenuation capacity after a major hit. 4. No side redirection capability. 5. Considerable inventory of parts and space for replacements required. 6. Modules may “walk” when placed on structures.

Table 9-8B

IMPACT SPEED FOR IMPACT ATTENUATORS

Highway Design Speed (mph)	Impact Speed (mph)	
	Freeways	Non-Freeways
$V \geq 60$	60	60
$45 < V < 60$	60	Design Speed
$V \leq 45$	—	45

9-8.04.02 Side Impacts

The impact attenuator design should allow for safe side impacts. All systems used in Mississippi, except the inertial systems, are designed to redirect impacting vehicles on the side.

9-8.04.03 Placement

Several factors should be considered in the placement of an impact attenuator:

1. Level terrain. The attenuator should be placed on a level surface or on a cross slope not to exceed 5 percent.

2. Curbs. No curbs should be present at proposed new installations. At existing attenuator sites, any existing curbs should be removed.

3. Surface. A paved, bituminous or concrete pad should be provided under the attenuator.

4. Elevated Structures. Inertial systems should not be placed on elevated structures.

5. Orientation. The proper orientation angle will depend upon the design speed, roadway alignment and lateral offset distance to the cushion. For most roadside conditions, a maximum angle of approximately 10°, as measured between the highway and attenuator longitudinal centerlines, may be appropriate.

9-9.0 TRAFFIC CONTROL CRITERIA

9-9.01 Permanent Traffic Control

The Department has adopted the *Manual of Uniform Traffic Control Devices* (MUTCD), and all projects must conform to its criteria. The following sections briefly elaborate on the Department's criteria and policies for permanent traffic control designs.

9-9.01.01 Criteria for No-Passing Zones

Pavement markings are used to delineate no-passing zones on all two-lane, two-way highways which warrant centerlines. The traffic control criteria for marking no-passing zones differ from the geometric design criteria for determining passing sight distance (Section 2-6.0). The differences result from different operational assumptions for the two criteria.

The Department has established 1000 ft of sight distance as its threshold for warranting pavement markings for no-passing zones; i.e., where the available sight distance falls below this amount at a horizontal or vertical curve, no-passing markings will be provided. This distance applies to all design speeds.

A 3.5-ft height of eye and object are used for measurement. The beginning of the no-passing zone is that point at which the sight distance first becomes less than 1000 ft. The end of the zone is that point at which the sight distance becomes equal to 1000 ft. No-passing zone markings should also be considered for other roadway features where passing should be prohibited (e.g., approaching a stop condition).

9-9.01.02 Pavement Markings

The selection of the type of pavement marking material on concrete surfaces will be determined based on current MDOT policy. This includes concrete pavements, bridge decks and bridge end pavements. For asphalt-surfaced pavements, the following will apply to the selection of the type of pavement marking material:

1. Center, No-Passing and Lane Lines. Plastic markings will be used as follows:

   a. on all Interstate highways;

   b. on all other highways with current ADT of 2000 or more; and

   c. on all bridges and bridge end pavements of highways with current ADT of 2000 or more.

   Paint will be used on all other asphalt-surfaced highways.

2. Edge Stripes. These will be provided on all pavements including D.B.S.T. Plastic marking will be used as follows:

   a. on all bridges and bridge end pavements of highways with current ADT of 2000 or more; and

   b. on all asphalt-surfaced highways with current ADT of 2000 or more.

   Paint will be used on all other highways.

Table 9-9A

UNIT OF MEASURE FOR PAVEMENT MARKINGS

TYPE OF PAVEMENT MARKING	UNIT OF MEASURE
Center Lines (1) Lane Lines (1) Pavement Edge Stripes (1)	Mile
No-Passing Lines Stop Bars (3) Detailed Traffic Stripes (2) (3)	Feet
Messages Arrows	Square Feet
Raised Pavement Markers	Each

Notes:

(1) Provide separate estimates for each type of line (skip or continuous) and for each color (typically white or yellow). The measurement of traffic stripes will be from end to end of each individual stripe. In the case of skip lines, the measurement will include nominal gap intervals and painted sections.

(2) Detailed traffic stripes are measured from end to end of individual stripes, excluding nominal gap intervals, without regard to color of individual paint stripes; however, plastic detailed stripe is separated by color.

(3) Pay items are based on a striping width of 6 in. Where a striping width of other than 6 in. is specified, the measured length will be adjusted in the ratio of the specified width to 6 in.

3. Continuous Two-Way Left-Turn Lanes (CTWLT Lanes). Plastic will be used.

4. Railroad Advance Markings. Plastic will be used.

5. Raised Pavement Markers. These should supplement the standard markings on all projects.

6. Other Special Markings. Markings such as channelization lines, crosswalks, stop bars, arrows, legends, left-turn bays, etc., will be the same type material as the centerline of the main roadway.

Contract plans for roadway paving should include provisions for all required pavement markings including painted stripes, plastic pavement markings, reflective markers and pavement messages. The following applies to the presentation of pavement marking information in the contract plans:

1. Estimating Quantities. Quantities of pavement marking will be estimated in terms of the pay item unit of measure called for by the specifications. These are presented in Table 9-9A.

2. Plan Sheets. Sufficient details of pavement markings should be shown in the plans, either by referencing Department standard drawings or by special detailed sheets. Typical striping details for several special applications are indicated for:

   a. simple, non-channelized intersections (Figure 9-9A);

   b. intersection of 2-lane with 4-lane roadways (Figure 9-9B);

   c. 5-lane with a CTWLT lane (Figure 9-9C);

   d. transition from a 4-lane undivided facility to a 5-lane facility with a CTWLT lane, and transition from a 5-lane facility with a CTWLT lane to a 4-lane divided facility (Figure 9-9D);

   e. 4-lane to 2-lane transitions at intersections (Figure 9-9E);

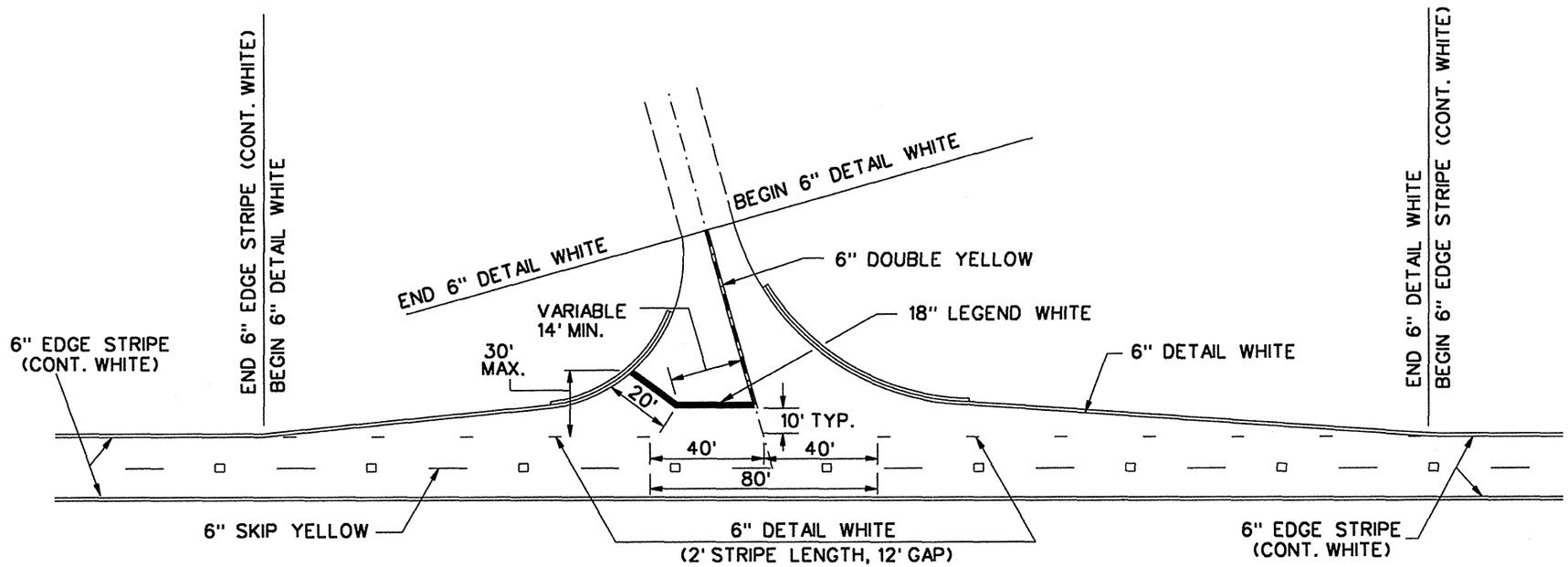
   f. desirable and minimum transitions between 2-lane and 4-lane roadways (Figure 9-9F);

   g. channelized with introduced left-turn lanes (Figure 9-9G);

   h. simple, channelized (no designated left-turn lanes) (Figure 9-9H); and

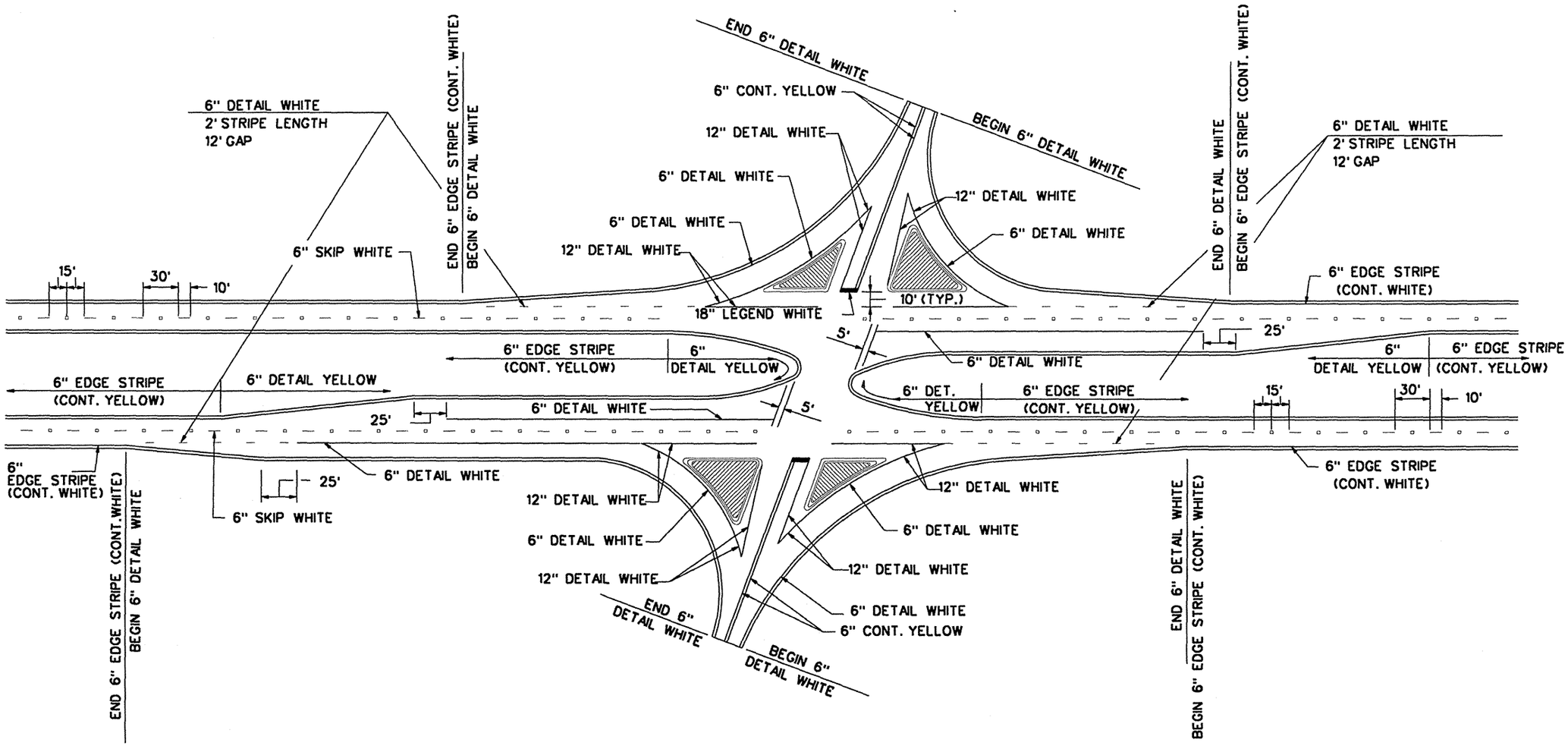
   i. channelized with introduced left-turn lanes and auxiliary lanes (Figure 9-9H).

See the *Mississippi Roadway Design Standard Drawings* for more detailed information on the use of pavement markings.



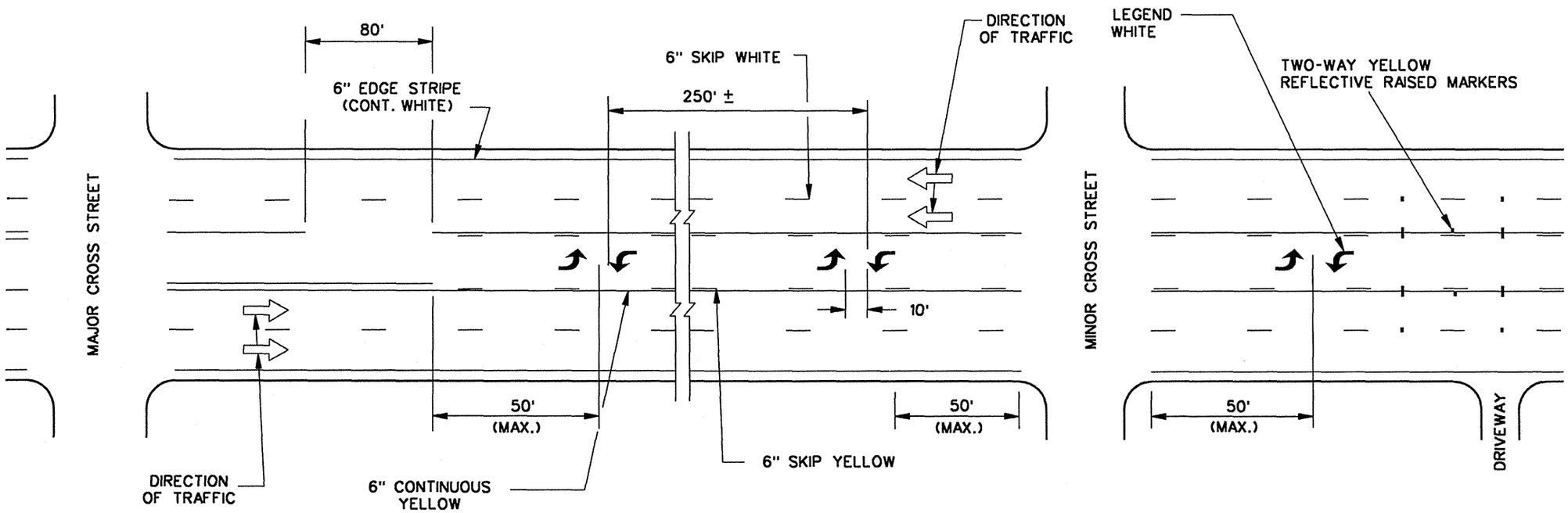
TYPICAL PAVEMENT MARKINGS
(Simple Intersection)

Figure 9-9A



TYPICAL PAVEMENT MARKINGS
(Intersection of 2-Lane with 4-Lane Roadways)

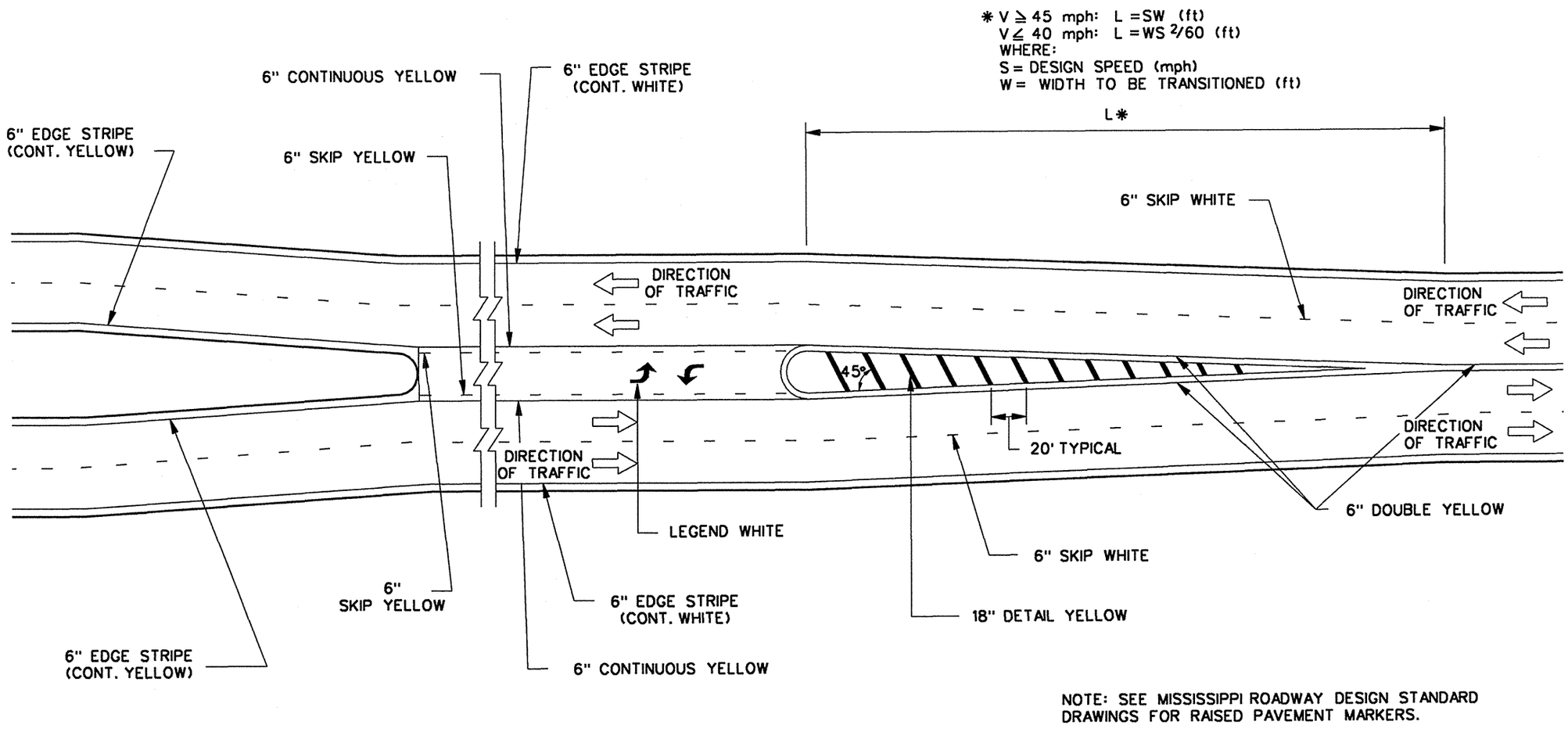
Figure 9-9B



NOTE: SEE MISSISSIPPI ROADWAY DESIGN STANDARD DRAWINGS FOR RAISED PAVEMENT MARKERS.

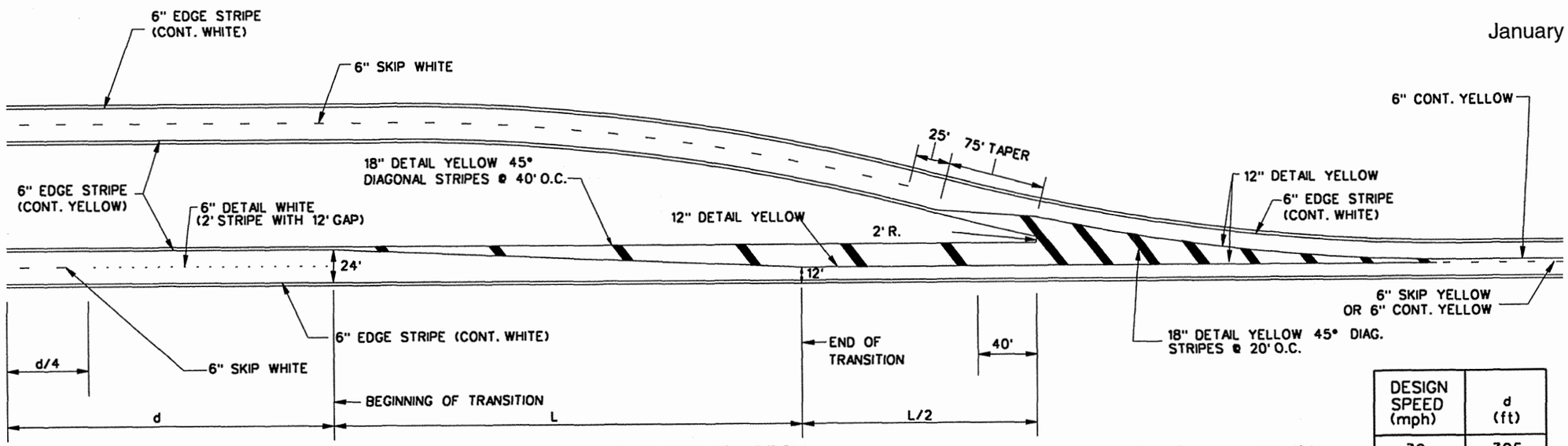
TYPICAL PAVEMENT MARKINGS FOR 5-LANE, CTWLT LANE

Figure 9-9C



TYPICAL PAVEMENT MARKINGS
(Transitions From 4-Lane Divided to 5-Lane CTWLT Lane to 4-Lane Undivided Facilities)

Figure 9-9D



(DESIRABLE TRANSITION)

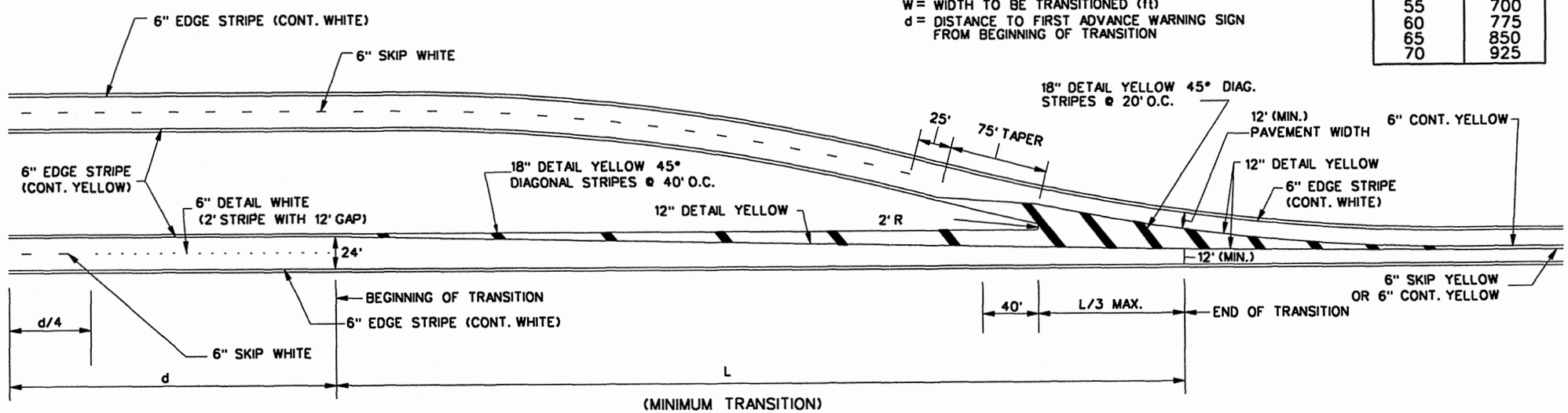
L = LENGTH OF TRANSITION = V ≧ 45 mph: L = SW (ft); V ≦ 40 mph: L = WS²/60 (ft)

S = DESIGN SPEED (mph)

W = WIDTH TO BE TRANSITIONED (ft)

d = DISTANCE TO FIRST ADVANCE WARNING SIGN FROM BEGINNING OF TRANSITION

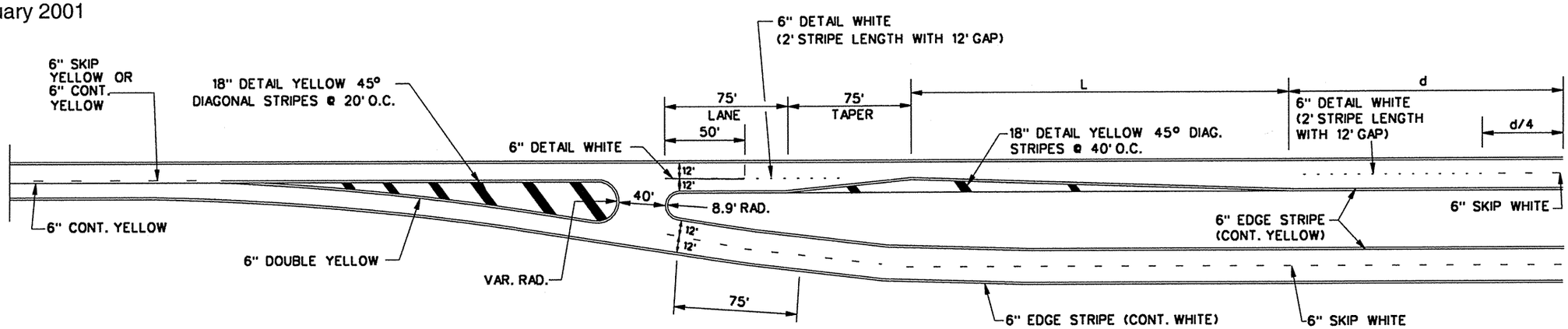
DESIGN SPEED (mph)	d (ft)
30	325
40	475
50	625
55	700
60	775
65	850
70	925



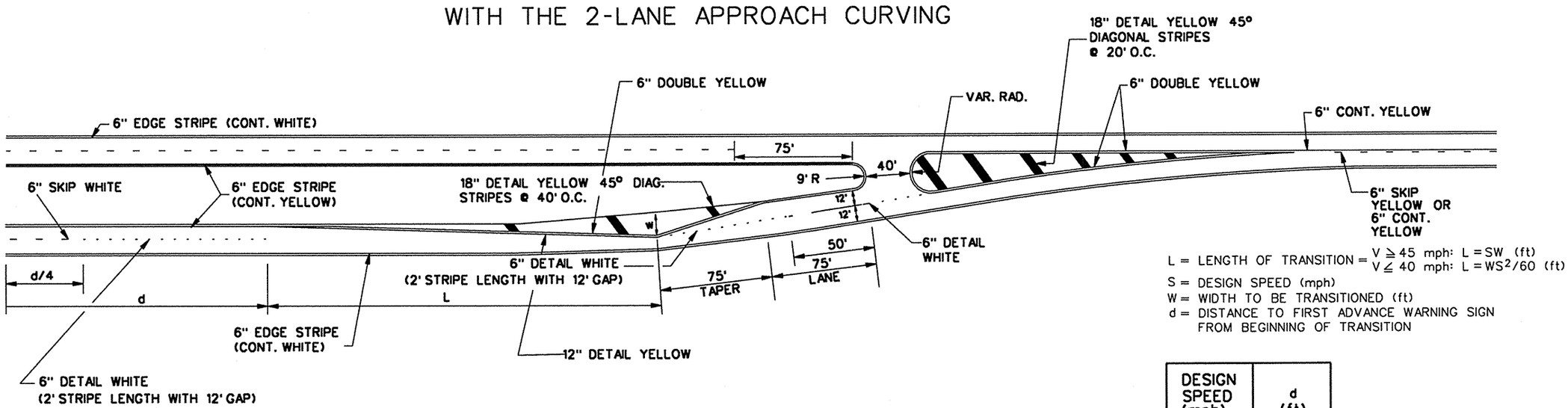
(MINIMUM TRANSITION)

TYPICAL PAVEMENT MARKINGS
(4-Lane to 2-Lane Transition)

Figure 9-9E



WITH THE 2-LANE APPROACH CURVING



WITH THE 2-LANE APPROACH STRAIGHT

L = LENGTH OF TRANSITION = $V \geq 45$ mph: $L = SW$ (ft); $V \leq 40$ mph: $L = WS^2/60$ (ft)

S = DESIGN SPEED (mph)

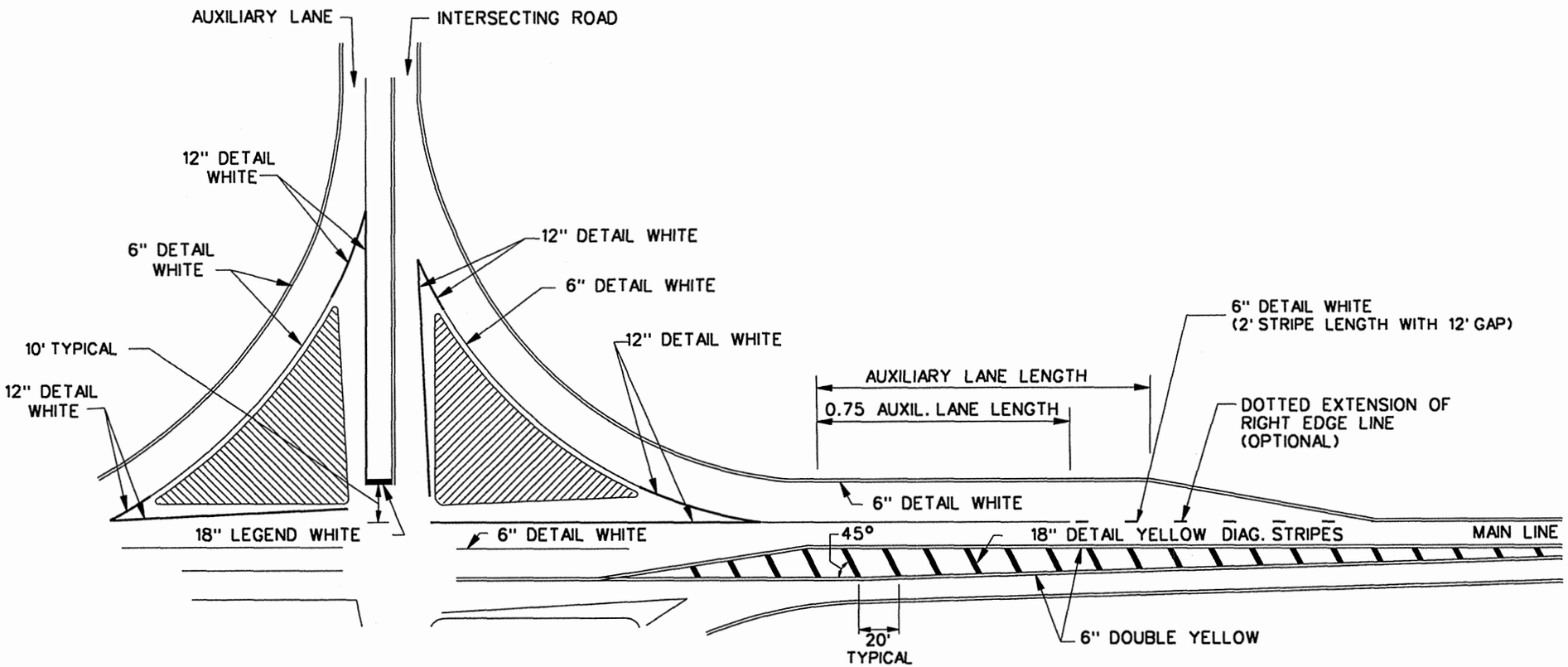
W = WIDTH TO BE TRANSITIONED (ft)

d = DISTANCE TO FIRST ADVANCE WARNING SIGN FROM BEGINNING OF TRANSITION

DESIGN SPEED (mph)	d (ft)
30	325
40	475
50	625
55	700
60	775
65	850
70	925

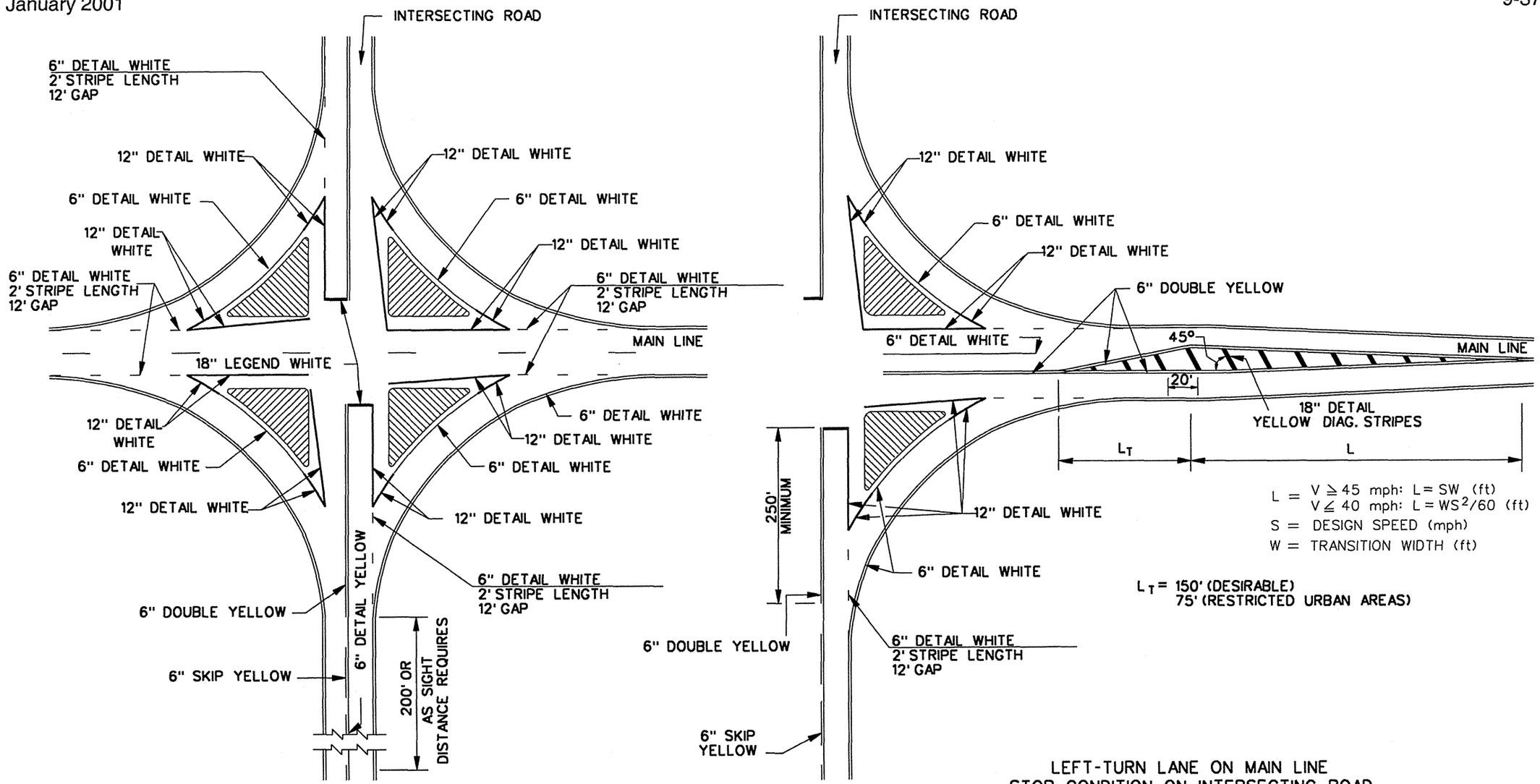
TYPICAL PAVEMENT MARKINGS
(2-Lane to 4-Lane Transition at Interchanges)

Figure 9-9F



TYPICAL PAVEMENT MARKINGS
(Channelized Intersection with Auxiliary Lanes)

Figure 9-9G



TYPICAL PAVEMENT MARKINGS
(Channelized Intersections)

Figure 9-9H

9-9.01.03 Signing

Signing is required to be in place on all projects before the roadway is opened to traffic. Permanent signing may or may not be included with the roadway contract. All signing must conform to the MUTCD.

The Signing Specialist is responsible for the design of signing and preparation of signing plan sheets. Regular plan sheets usually will be developed with a scale of 1″ = 200′. For interchanges and rest areas, a scale of 1″ = 100′is preferred.

The design procedure includes:

1. placing desired standard signs and guide signs at appropriate locations on the plan sheets;
2. determining actual sizes of all guide signs;
3. conducting field inspections and making necessary changes;
4. determining length and size of all sign supports;
5. making detailed drawings of individual directional signs;
6. identifying requirements for delineators, object markers and distance reference markers; and
7. computing quantities for all applicable pay items and preparing a summary of quantities.

For State Aid projects, permanent signs are contract items and are paid for by the appropriate pay item.

9-9.01.04 Traffic Signals

Traffic signals are considered for installation at intersections where warranted as presented in the MUTCD. The 20-year projection used for geometric design of the roadway can be used for anticipating signal installations; however, normally only a 5-year projection is used to determine if signals should be installed at non-signalized intersections.

If an investigation reveals the possible need for a signal installation, the Traffic Engineering Division should be requested to make a detailed analysis of the location to determine if a signal should be installed.

If there are existing signals, the designer should notify the Traffic Engineering Division, which will decide whether the existing signals should be reworked under the proposed construction contract, left in place and adjusted by State forces to match the new construction, or removed.

The geometric design of intersections should be based on the existence of a traffic signal, unless the traffic volume warrants clearly indicate otherwise.

The designer can gain assistance from the Traffic Engineering Division in developing the geometric requirements for a signalized intersection. Where a signal will be installed under the construction contract, plans will be developed by the Traffic Engineering Division for the signal installation, and these plans will be inserted into the construction plans. The designer should coordinate his plan development with the Traffic Engineering Division's signal plan development to allow for proper sheet numbering, pay item listing, plan completion dates, field inspection, geometric design, etc.

9-9.01.05 Lighting

The AASHTO publication *An Informational Guide for Roadway Lighting* presents generally accepted warranting conditions for installing fixed highway lighting. These criteria are applicable in Mississippi. Additional information is available in NCHRP Report No. 152 *Warrants for Highway Lighting* and the FHWA *Roadway Lighting Handbook.*

Department policy specifically warrants highway lighting at the following locations:

1. interchanges between Interstate routes;

2. complete interchange lighting where the total current ADT ramp traffic entering and leaving the freeway within the interchange area exceeds 10,000 for urban conditions, 8000 for suburban conditions or 5000 for rural conditions;

3. sections of through divided highways with median widths of less than 16 ft;

4. all sections of through highway with six or more through travel lanes;

5. sections of highway approaching illuminated interchange areas for sufficient distances before the interchange to safely transition drivers from an unlighted to lighted area; and

6. all rest areas, hospitality stations and weigh stations including entrance and exit ramps and the site area.

In addition, attention should be directed to those locations experiencing a large number of night time accidents. If the ratio of night to day accidents is considerably higher than the Statewide average for similar locations, lighting may reduce the night time accident rate.

The Lighting Specialist has the basic responsibility for designing the lighting layout, preparing the contract plans and recommending specifications. Normally, the lighting contract will be separate from the roadway construction contract. There is a need, however, for a considerable amount of coordination with the Roadway Design Division, Bridge Design Division, Geotechnical Branch and Construction Division.

A lighting layout should be transmitted to the Bridge Division for bridges on which future lighting will be installed, so that brackets for messenger cable, lighting assembly mounts and anchor bolts will be provided as part of the bridge structure.

9-9.02 Temporary Traffic Control in Construction Zones

9-9.02.01 General

Highway construction will disrupt the normal traffic operations; therefore, the designer must devote special attention to traffic control in construction zones during the design of every project. The Traffic Control Plan (TCP) should minimize the operational and safety problems through the work zone.

The designer is responsible for preparing the TCP for the maintenance and protection of traffic in construction zones. On complex projects, he/she may need to coordinate the development of the TCP with the Traffic Engineering Division.

9-9.02.02 Traffic Control Plans (TCP)

A TCP is a plan for the safe and comfortable movement of traffic through a highway or street work zone. It may range in scope from a TCP which describes every detail of traffic accommodation to a simple reference to the MUTCD and the Department's *Standard Drawings.* The scope of the TCP will depend on the complexity and duration of the construction project.

The plans, specifications and estimates (PS&E) for each project will include a TCP. The TCP should address the following, as applicable:

1. signing;
2. application and removal of pavement markings;
3. delineation and channelization;
4. lane closures;
5. detours and crossovers;
6. placement of all traffic control devices;
7. location and types of safety appurtenances;
8. means of maintaining access to and from existing interchange ramps;
9. flagging;
10. work scheduling restrictions, if applicable;
11. storage of equipment and materials;
12. traffic regulations, if applicable;
13. the time for which a specific traffic control feature (e.g., a flashing arrow board) will be in use (throughout construction period, only during closure of left lane, etc.); and
14. surveillance and inspection requirements.

For additional details, refer to the latest edition of the MUTCD. In addition, the Department has developed several standard sheets which may be applicable to traffic control in a construction zone.

For construction work which may involve a significant disruption to normal traffic operations, the Department should coordinate the Traffic Control Plan development, including the incident management plan (where applicable), with all affected local interests.

9-9.02.03 Geometric Design

The geometric design of the traffic control plan should minimize operational and safety problems. This will require careful consideration of many elements. The following will apply:

1. Design Speed. Significant speed reductions through construction zones are undesirable and may lead to poor operating conditions. Regulatory or warning speed signs are generally ineffective with the exception, perhaps, of signs at horizontal curves. The desirable design speed will be no more than 10 mph below the posted speed limit for the approaching roadway. However, on low-volume facilities, it may be justified to use a design speed which is more than 10 mph below the approaching posted speed limit.

   Once the design speed is selected through the construction zone, this will determine the applicable criteria for those design elements based on speed (e.g., horizontal curves).

2. Sight Distance. Changes in the geometric design of the approaching highway are often necessary through construction zones. Therefore, the available sight distance to the approaching motorist is especially important. Unfortunately, the location of many design features may be dictated by the construction operations. However, some elements may have an optional location. For example, lane closures and other transitions should be located

so that an approaching vehicle has decision sight distance to the closure or transition. (See Table 2-6A).

Where inadequate sight distance is available to a construction zone, this should be considered when determining the advance signing to the site.

3. Lane/Shoulder Width. Desirably, there will be no reduction in the cross section width through the construction zone. However, this will not always be practical. The designer should minimize any reductions but, in some cases, it may be necessary to provide a narrow lane with no shoulders. This is considered acceptable but not desirable.

4. Lane Closures/Other Transitions. As discussed in Comment #2, decision sight distance should be available to any closure or other roadway transition. In addition, the designer should ensure that the rate of taper conforms to the MUTCD criteria. These are:

$$L = SW \qquad (S \geq 45 \text{ mph})$$

$$L = \frac{WS^2}{60} \qquad (S \leq 40 \text{ mph})$$

where:
L = minimum length of transition, ft
S = posted speed limit before construction, mph
W = width of offset, ft

Sections 9-9.02.07 through 9-9.02.09 provide geometric design criteria specifically for detours/diversions, temporary crossovers and bridge replacement projects.

9-9.02.04 Roadside Safety

Positive protection may be warranted in construction zones. This decision will be made on a project-by-project and site-by-site basis. The following factors should be considered:

1. duration of construction activity;
2. traffic volumes;
3. nature of hazard (e.g., edge of traveled way drop-offs);
4. proximity between traffic and construction workers and construction equipment;
5. adverse geometrics which may increase the likelihood of run-off-the-road vehicles;
6. two-way traffic on one roadway of a divided highway (see Section 9-9.02.06);
7. using a barrier in the taper of a lane closure or lane transition; and/or
8. the length of the work zone.

Roadside safety appurtenances in construction zones include the following:

1. portable concrete median barrier (PCMB) (see *Standard Drawings*);
2. temporary guardrail (e.g., for shielding bridge ends during construction) (see *Standard Drawings*);
3. construction zone QuadGuard;

4. inertial impact attenuator; and

5. the Triton Barrier.

The design and installation details for these safety appurtenances should be the same as for permanent installations, if practical. The *Standard Drawings* and the *Roadside Design Guide* should be followed for all construction zone safety appurtenances.

9-9.02.05 Capacity

Capacity problems through construction zones can create considerable driver frustration and poor operations. Therefore, the designer should estimate the predicted level of service during peak traffic hours. The design should avoid lengthy back-ups and delays. The lateral distance from the traveled way to obstacles will be a significant capacity adjustment. A capacity analysis could lead to significant modifications in the traffic control scheme. For example, lane closures may be prohibited during peak traffic hours, or the sequence of construction could be altered to improve traffic operations.

9-9.02.06 Two-Way Traffic on Divided Highways

Often, the most efficient construction scheme on divided highways is to close one roadway of the highway to traffic and provide 2-way operation on the other roadway. This allows construction work to proceed unimpeded by traffic. However, this is a severe violation of driver expectancy which can lead to head-on collisions. Therefore, this scheme should not be used until other methods of traffic control have been evaluated.

Where 2-way traffic is used on a divided highway, some means of separation between the two traffic flows is necessary, except where:

1. drivers can see the transition back to the normal 1-way operation, or

2. unusual conditions exist, such as very narrow bridges, which make separation undesirable.

The separation can be provided by drums, cones, the PCMB or other appropriate devices. To determine which type of separation should be used, a project-by-project analysis will be necessary. The analysis should consider traffic volumes, duration of separation, vehicular speeds, length of 2-way operation, geometrics (sight distance, lane widths, horizontal alignment, etc.) and construction costs. In particular, the transitions to and from the 2-way operation must be carefully designed.

See Section 9-9.02.08 for the design of temporary crossovers, which are typically needed to convert a divided highway to temporary two-way operation.

9-9.02.07 Detours/Diversions

Detours or diversions around a construction site effectively remove the traffic from the work area. These may be constructed exclusively for the project or may use existing roads to re-route the traffic. Because they have obvious safety and operational advantages, detours and diversions should be used if practical.

Sections 9-9.02.08 and 9-9.02.09 provide design criteria explicitly for temporary crossovers and bridge replacement projects, which are special types of detours/ diversions. The following criteria apply in general to detours and diversions:

1. Design Speed. The design speed of the detour or diversion will be determined on a case-by-case basis. It should preferably not be more than 15 mph below the approaching highway design speed. However, this is frequently not practical for low-volume roads or where local roads are used for the detour.

2. Maximum Degree of Curvature. Table 9-9B presents the Department's maximum degree of curvature for detours and diversions on all facilities. These are based on the Department's 3R criteria (Figure 11-2A) assuming retention of the typical cross slope; i.e., e = -0.02.

3. Width. The cross section of the detour or diversion should be as wide as practical. Table 9-9C presents minimum width criteria for two-lane, two-way facilities used for detours/diversions.

9-9.02.08 Temporary Crossovers

Temporary crossovers are typically provided to transition a divided highway to two-way operation on one roadway. Because of their special application, the following applies to the design of temporary crossovers:

1. Signing. TCP-13 in the *Roadway Design Standard Drawings* provides the signing design for temporary crossovers.

2. Design Speed. The design speed for the crossover will be no more than 10 mph below the posted speed limit of the approaching highway.

3. Maximum Degree of Curvature. Where used on tangent sections on mainline, D_{max} must be 1°30′. Where used on horizontal curves on mainline, D_{max} may be less than 1°30′, but the combination of D, design speed and superelevation rate must satisfy the criteria in Figure 11-2A.

9-9.02.09 Bridge Replacement Projects

Where detours/diversions are used for bridge replacement projects, the following additional criteria apply to their design:

Table 9-9B

MAXIMUM DEGREE OF CURVATURE FOR DETOURS/DIVERSIONS

Design Speed	Superelevation Rate	D_{max}
30 mph	-0.02	13°15′
35 mph	-0.02	9°30′
40 mph	-0.02	7°00′
45 mph	-0.02	5°15′
50 mph	-0.02	4°00′
55 mph	-0.02	3°00′
60 mph	-0.02	2°15′
65 mph	-0.02	1°45′
70 mph	-0.02	1°15′

Table 9-9C

WIDTHS OF DETOURS/DIVERSIONS
(Two-Lane, Two-Way Facilities)

DHV	Roadway Width		Bridge Width
	Lanes	Shoulders	
≤ 200	12 ft	3 ft	24 ft
> 200	12 ft	5 ft	24 ft

1. Guardrail-to-Bridge Rail Transitions. The minimum length of these transitions will be 37.5 ft. See the *Standard Drawings* for design details of the transition.

2. Advisory Speed Signs. Advisory speed signs will be required which reduce the speed limit to 35 mph in 10-mph increments.

9-9.02.10 Temporary Pavement Markings

Temporary pavement markings may be required in construction zones to clearly delineate the intended path of travel for motorists. Once the decision has been made to use temporary pavement markings, the following presents the Department's policy for selecting the type of tape:

1. Type 1. This tape should be used for:

   a. complicated lane closures,
   b. detour switches from 4-lane to 2-lane operation, and
   c. detour "run-arounds."

   Figure 9-9I illustrates these three applications of Type 1 tape and provides additional information for its use.

2. Type 2. This tape should be used for all other locations where temporary pavement markings are warranted.

3. Temporary Raised Pavement Markers. These are required if the construction zone will be open to traffic for more than three months.

9-10.0 REFERENCES

1. *Roadside Design Guide*, AASHTO, 1988, 1996.

2. *A Roadside Design Procedure*, James Hatton, Federal Highway Administration, January, 1974.

3. NCHRP 350 *Recommended Procedures for the Safety Performance of Highway Features*, Transportation Research Board, 1993.

4. *Design Program, Saving Lives by Design*, Energy Absorption Systems, Inc.

5. *Manual on Uniform Traffic Control Devices*, FHWA.

Notes:

1. *Temporary pavement markings shall be placed on all construction in accordance with the standards shown here. The only exception to this shall be for projects of two days or less.*

2. *Removable pavement markings shall be installed on the finished pavement surface, on reversible crossovers and at other hazardous locations as designated on the plans.*

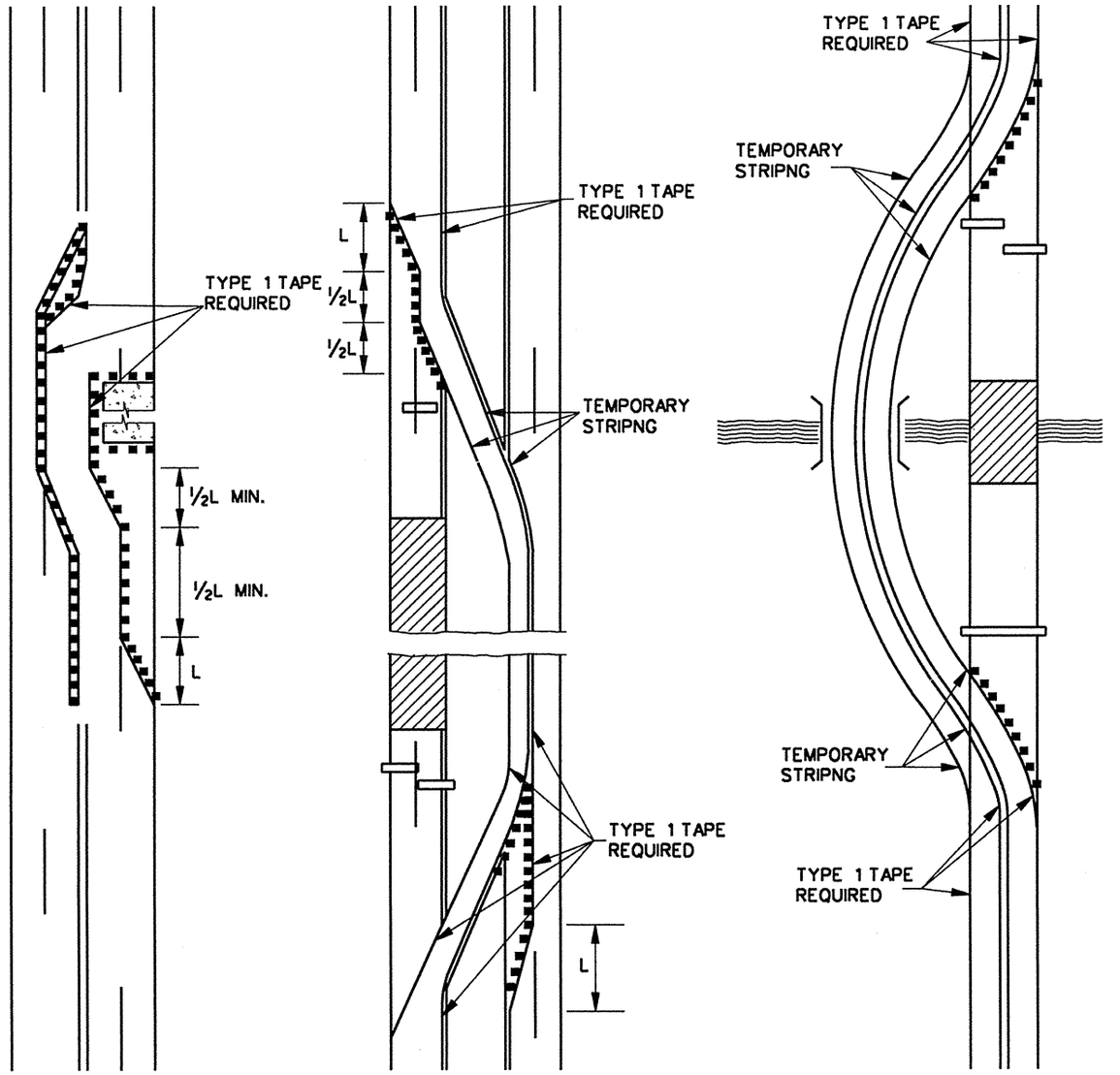
3. *Non-removable pavement markings shall be installed at locations which will be further disturbed or which will receive an additional surface course.*

4. *Removable pavement markings shall be preformed retroreflective tape that is readily removed when necessary in large pieces without the use of heat, chemicals, grinding or blasting and without objectionable residual marks. Removable pavement markings shall be placed on all areas designated as "TYPE 1" as shown on the plans.*

5. *Temporary pavement markings shall be in place before the area is open to traffic. Where removal of temporary markings is required, the markings shall be removed immediately prior to the placement of permanent pavement markings.*

6. *All markings shall be applied in such a manner as to ensure neat, accurate and uniform lines.*

7. *Pavement markings no longer applicable which might create confusion in the mind of the motorist shall be removed or obliterated before the roadway is open to traffic.*



REQUIRED LOCATIONS FOR TEMPORARY TYPE 1 TAPE

Figure 9-9I

Chapter 10

Right-of-Way and Fencing

MISSISSIPPI DESIGN MANUAL

Chapter 10

RIGHT-OF-WAY AND FENCING

Section		Page
10-1.0	RIGHT-OF-WAY	10-1
10-1.01	General Procedures	10-1
10-1.02	Right-of-Way Widths	10-1
10-1.03	Right-of-Way Configurations	10-1
10-1.04	Beginning or Ending of Project.	10-1
10-1.05	Temporary Easements	10-2
10-1.06	Access Control	10-2
10-2.0	FENCING	10-2
10-2.01	Types	10-2
10-2.01.01	Woven Wire (Rural Areas)	10-2
10-2.01.02	Chain Link (Urban or Built-up Areas)	10-2
10-2.02	Estimating Quantities	10-3
10-2.03	Fencing Plans	10-4

Chapter 10

RIGHT-OF-WAY AND FENCING

This chapter discusses design criteria and procedures for right-of-way requirements and fencing. Where possible, the design criteria presented in other chapters will be referenced.

10-1.0 RIGHT-OF-WAY

10-1.01 General Procedures

When a survey is received in the Roadway Design Division, the property maps are forwarded to the Right-of-Way Division. Pertinent right-of-way information from the property maps should be recorded on the plan-profile sheets.

Following the field inspection(s) and after the designer has completed all revisions that will affect property owners, the final right-of-way plans are submitted to the Right-of-Way Division. The Right-of-Way Division then furnishes the District with a set of plans to be used for field staking of the right-of-way. Any subsequent revisions by the Design Division should be noted, and revised prints should be forwarded to the Right-of-Way Division.

10-1.02 Right-of-Way Widths

The right-of-way width is the sum of all cross section elements including lane widths, median width, side slopes and border widths. The right-of-way border width is defined as the distance between the right-of-way lines inward to the construction limits of the project. The criteria for right-of-way width are presented in the Tables of Geometric Design Criteria in Chapters 2, 11, 13 and 14. Construction limits represent the toe-of-fill slopes or top-of-cut slopes and are illustrated on the plan sheet with short (0.2-in) broken lines with dots and skips.

10-1.03 Right-of-Way Configurations

Right-of-way limits should be illustrated with as few breaks and changes in width as practical. Considerations should also be given to the ease of writing right-of-way descriptions. The following general criteria should apply:

1. Where there is a need for a width change close to the PC or PT of a horizontal curve, the breaks should be made to coincide with the curve points.

2. Where width changes are required to both right and left sides of the facility, the break points should coincide with the same centerline stationing.

3. Breaks in width should not occur in drainage channels, roads or driveways where installation of right-of-way markers would be impractical.

10-1.04 Beginning or Ending of Project

When the project's beginning or ending right-of-way width falls within the limits of an individual property ownership, consideration should be given to acquiring the remaining right-of-way width through the ownership for future connecting projects. This will eliminate the need for further negotiations with the property owner.

10-1.05 Temporary Easements

A temporary easement may be obtained when it is not necessary to obtain permanent possession of the land. However, if future maintenance is anticipated, permanent right-of-way should be provided. The acquisition of a temporary easement may be required:

1. to obtain proper grade on private driveways,
2. for inlet and outlet ditches at drainage structures,
3. for channel alterations where future maintenance is not anticipated, or
4. on urban projects to provide a construction area for installation of storm drains and driveways.

10-1.06 Access Control

Access control is the condition where the public authority fully or partially controls the right of property owners to have access to and from the public highway. Section 2-4.0 discusses the definitions of the basic types of access control and their application.

10-2.0 FENCING

Fencing is required on all Interstate highways. Other Type 1 full-controlled-access highways may be fenced where warranted and approved, in writing, by the Chief Engineer. The designer should reference the AASHTO publication *An Informational Guide On Fencing Controlled Access Highways* for additional information.

10-2.01 Types

The two basic types of fences used on freeways by the Department are the woven wire and the chain link fence. The designer should determine the individual characteristics such as the type of posts and height of chain link fence. Details of the types of fences, posts and gates and examples of typical installations are provided in the *Standard Roadway Drawings.* General design criteria for fencing are discussed in the following sections.

10-2.01.01 Woven Wire (Rural Areas)

1. Normally used with timber posts.
2. Use concrete posts within limits of interchange areas (between the extremities of exit or entrance ramps).
3. Always use timber posts adjacent to rest areas to preserve natural appearance.
4. On multilane highways with wide medians where structures are installed for the passage of animals, use a comparable fence across the median on each side of the path to allow passage without permitting access to the roadway.

10-2.01.02 Chain Link (Urban or Built-up Areas)

1. Normally use 5-ft high chain link.
2. Use 6-ft high chain link where pedestrian traffic is likely to be heavy.
3. At junkyard locations, use chain link with lattice of a height to sufficiently screen the junkyard from the highway.

10-2.02 Estimating Quantities

Estimates of fencing quantities should be made according to the *Standard Specifications* and will include the following pay items:

1. length (in feet) of fence of the type and size to be installed, exclusive of openings;

2. the number of posts by type of material, by type of post (e.g., line post, brace post, gate post) and by length;

3. the number of gates of specified type and size; and

4. length (in feet) of single-strand barbed wire when not included in the standard installation.

The method of estimating fencing quantities is as follows:

1. Total Length of Fence. The total length of fence (in feet) is a linear measurement, exclusive of openings, and is measured directly from the plan and profile sheets.

2. Total Number of Brace Posts. The total number of brace posts is estimated as follows:

   a. 3 brace posts are required at every fence turn,
   b. 1 brace post is required at every fence gate,
   c. 2 brace posts are required every 250 ft in curves, and
   d. 2 brace posts are required every 500 ft along tangents.

3. Type of Brace Posts. To estimate the number of each brace post type, apply the following percent breakdown to the total number of brace posts determined in item 2 above:

   a. 75% for 8-ft brace posts,
   b. 15% for 10-ft brace posts, and
   c. 10% for 12-ft brace posts.

4. Total Number of Line Posts. The total number of line posts is determined as follows:

   ((Total Fence Length)/12 ft) - Total Number of Brace Posts

5. Type of Line Posts. To estimate the number of each line post type, apply the following percent breakdown to the total number of line posts determined in Item 4 above:

   a. 75% for 7-ft line posts,
   b. 15% for 9-ft line posts, and
   c. 10% for 10-ft line posts.

6. Number of Steel Posts at Crossings. To determine the number of steel posts required at each crossing (e.g., stream), apply the following criteria:

   a. Type I crossings require no steel posts,
   b. Type II crossings require 3 steel posts, and
   c. Type III crossings require 2 steel posts.

7. Number of Gate Posts. One gate post is required for each gate.

8. Length of Barbed Wire at Crossings. To determine the length of barbed wire required at crossings, apply the following criteria:

   a. Type I crossings require 350 ft of barbed wire,
   b. Type II crossings require 200 ft of barbed wire, and
   c. Type III crossings require 150 ft of barbed wire.

Estimated quantities should be provided on each plan sheet and the total of all fencing items summarized on the Summary of Quantities sheet.

10-2.03 Fencing Plans

Fencing work may be included with the roadway construction project or may be performed under a separate fencing contract. Normally, a fencing contract will be let to include the limits of several roadway construction projects.

When fencing is included with other construction work, the fencing requirements are provided on the plan portion of the plan-profile sheets. For separate fencing, contract plan sheets are drawn to a scale of 1″ = 200′, and interchange layouts to a scale of 1″ = 100′. They provide only a plan view with items pertinent to the fencing. These items include right-of-way widths, "no-access" limits, drainage structures, ditch crossings, bridges, etc.

Chapter 11

Geometric Design of Existing Highways (3R Projects)

MISSISSIPPI DESIGN MANUAL

Chapter 11

GEOMETRIC DESIGN OF EXISTING HIGHWAYS (3R Projects)

Section		Page
11-1.0	INTRODUCTION	11-1
11-2.0	3R PROJECTS (Non-Freeways)	11-1
11-2.01	General	11-1
11-2.01.01	Background	11-1
11-2.01.02	Objectives	11-3
11-2.01.03	Approach	11-3
11-2.01.04	3R Project Evaluation	11-4
11-2.02	Design Controls	11-6
11-2.02.01	Design Speed	11-6
11-2.02.02	Highway Capacity	11-6
11-2.03	Cross Sections	11-6
11-2.03.01	Roadway Widths	11-6
11-2.03.02	Median Width	11-7
11-2.03.03	Bridges	11-7
11-2.03.04	Right-of-Way	11-8
11-2.04	Horizontal Alignment	11-8
11-2.05	Vertical Alignment	11-9
11-2.05.01	Grades	11-9
11-2.05.02	Crest Vertical Curves	11-9
11-2.05.03	Sag Vertical Curves	11-10
11-2.06	Intersections At-Grade	11-12
11-2.06.01	Intersection Sight Distance	11-12
11-2.06.02	Auxiliary Turning Lanes	11-12
11-2.06.03	Driveway Design	11-12
11-2.07	Special Design Elements	11-13
11-2.08	Roadside Safety	11-13
11-2.08.01	Clear Zones	11-13
11-2.08.02	Embankments (Fill Sections)	11-14
11-2.08.03	Intersecting Slopes	11-14
11-2.08.04	Roadside Ditches	11-16
11-2.08.05	Safety Appurtenances	11-16
11-2.09	3R Tables of Geometric Design Criteria	11-17
11-2.10	Exceptions to Geometric Design Criteria	11-17
11-3.0	3R PROJECTS (Freeways)	11-22
11-3.01	General	11-22
11-3.01.01	Background	11-22
11-3.01.02	Objectives	11-22
11-3.01.03	Approach	11-22
11-3.01.04	3R Project Evaluation	11-23
11-3.02	Geometric Design of 3R Freeway Projects	11-23
11-3.02.01	Design for Original Construction	11-23
11-3.02.02	Design Speed	11-23
11-3.02.03	Traffic Volumes	11-24
11-3.02.04	Vertical Clearances	11-24
11-3.02.05	Bridges	11-24
11-3.02.06	Roadside Safety	11-25
11-3.02.07	Interchanges	11-26
11-3.02.08	Emergency Crossovers	11-27
11-3.02.09	3R Table of Geometric Design Criteria	11-27
11-3.02.10	Exceptions to Geometric Design Criteria	11-27
11-4.0	SPOT IMPROVEMENTS (Non-Freeways)	11-27
11-4.01	Objectives	11-27
11-4.02	Approach	11-30
11-4.03	Geometric Design of Highway Bridge Replacement/ Rehabilitation Program (HBRRP) Projects	11-30

Chapter 11

GEOMETRIC DESIGN OF EXISTING HIGHWAYS (3R Projects)

11-1.0 INTRODUCTION

A significant percentage of the Department's current and future highway program will involve work on existing highways. The Department's responsibility is to realize the greatest overall benefit from the available funds. Therefore, the geometric design of projects on existing highways must be viewed from a different perspective than the design of new construction/reconstruction projects. 3R (resurfacing, restoration and rehabilitation) projects are often initiated for reasons other than geometric design deficiencies (e.g., pavement deterioration), and they must often be designed within restrictive right-of-way, financial and environmental constraints. Therefore, the design criteria for new construction are often not attainable without major and, frequently, unacceptable adverse impacts. At the same time, however, the Department must take the opportunity to make cost-effective, practical improvements to the geometric design of existing highways and streets.

For these reasons, the Department has adopted revised limits for geometric design criteria for projects on existing highways which are, in many cases, lower than the values for new construction/reconstruction. These criteria are based on a sound, engineering assessment of the underlying principles behind geometric design and on how the criteria for new construction/reconstruction can be legitimately modified to apply to existing highways without sacrificing highway safety.

Chapter 11 presents the Department's criteria for rural 3R non-freeway projects on the State highway system; for 3R freeway projects; and for spot improvements (non-freeways). Section 13-4.0 presents 3R criteria for State-Aid projects, and Section 14-3.0 presents 3R criteria for urban non-freeway facilities. The 3R criteria are intended to find the balance among many competing and conflicting objectives. These include the objective of improving Mississippi's existing highways; the objective of minimizing the adverse impacts of highway construction to existing highways; and the objective of improving the greatest number of miles within the available funds.

11-2.0 3R PROJECTS (Non-Freeways)

11-2.01 General

11-2.01.01 Background

Federal 3R Regulations

On June 10, 1982, FHWA issued its Final Rule entitled *Design Standards for Highways; Resurfacing, Restoration and Rehabilitation of Streets and Highways Other Than Freeways.* This rule modified 23CFR Part 625 to adopt a flexible approach to the geometric design of 3R projects. Part 625 was modified again on March 31, 1983 to explicitly state that one objective of 3R projects is to enhance highway safety. In the 1982 rule FHWA determined that it was not practical to adopt 3R design criteria for nationwide application. Instead, each State can develop its own criteria and/or procedures for the design of 3R projects, subject to FHWA approval. This allows each State to tailor its design criteria for the 3R program according to the prevailing conditions within that State. This approach is in contrast to the application of criteria for new construction and reconstruction, for which the AASHTO *A Policy on*

Geometric Design of Highways and Streets provides nationwide criteria for application.

Special Report 214

In 1987, the Transportation Research Board published Special Report 214 *Designing Safer Roads; Practices for Resurfacing, Restoration and Rehabilitation.* This study was mandated by the Surface Transportation Assistance Act of 1982. The objective of the TRB study was to examine the safety cost-effectiveness of highway geometric design criteria and to recommend minimum design criteria for 3R projects on non-freeways. The final TRB report (SR214):

1. reviewed the existing 3R design practices of 15 State highway agencies and several local highway agencies;

2. examined the relationship between highway accident potential and geometric design elements, based on the existing research literature and on special research projects commissioned as part of the TRB study;

3. examined the relationship between the extent of geometric design improvements and the cost of 3R projects;

4. discussed the issue of cost-effectiveness relative to geometric design improvements on 3R projects;

5. reviewed the literature on tort liability and geometric design;

6. presented a safety-conscious design process; and

7. presented specific numerical criteria for the geometric design of 3R projects for the following elements:

   a. travel lane and shoulder widths,
   b. horizontal curvature and superelevation,
   c. vertical curvature,
   d. bridge width,
   e. side slopes, and
   f. pavement cross slopes.

The SR214 information has been incorporated, where considered appropriate for Mississippi, into the Department's criteria and procedures for 3R projects. The designer should reference SR214 for more discussion on 3R projects.

FHWA Technical Advisory T5040.28

Pursuant to its adoption of SR214, FHWA issued on October 17, 1988, Technical Advisory T5040.28 "Developing Geometric Design Criteria and Processes for Non-Freeway RRR Projects." The purpose of the Advisory is to provide guidance on developing or modifying criteria for the design of Federal-aid, non-freeway 3R projects. The Advisory:

1. discusses the procedures for developing 3R criteria;

2. discusses the factors which should be evaluated in a safety-conscious design process;

3. discusses the application of design exceptions for the FHWA controlling design criteria on 3R projects; and

4. presents specific criteria for the design of 3R projects based on SR214.

The information from the Technical Advisory has been incorporated, where considered appropriate for Mississippi, into the Department's criteria and procedures for 3R projects.

11-2.01.02 Objectives

From an overall perspective, the 3R program is intended to improve the greatest number of highway miles within the available funds for highway projects. "Improve" is meant to apply to all aspects which determine a facility's serviceability, including:

1. the structural integrity of the pavement, bridges and culverts;

2. the drainage design of the facility to, among other objectives, minimize ponding on the highway, to protect the pavement structure from failure, and to prevent roadway flooding during the design-year storm;

3. from a highway capacity perspective, the level of service provided for the traffic flow;

4. the adequacy of access to abutting properties;

5. the geometric design of the highway to safely accommodate expected vehicular speeds and traffic volumes;

6. the roadside safety design to reduce, within some reasonable boundary, the adverse impacts of run-off-the-road vehicles; and

7. the traffic control devices to provide the driver with critical information and to meet driver expectancies.

These objectives are competing for the limited funds available for 3R projects on existing highways. The Department's responsibility is to realize the greatest overall benefit from the available funds. Therefore, on individual projects, some compromises may be necessary to achieve the goals of the overall highway program. Specifically for geometric design and roadside safety, the compromise is between what is desirable (new construction/reconstruction criteria) and what is practical for the specific conditions of each highway project.

Considering the above discussion, the Department has adopted and FHWA has approved its approach to the geometric design of 3R non-freeway projects. The overall objective of the Department's criteria is to fulfill the requirements of the FHWA regulation and Technical Advisory which govern the 3R program. These objectives may be summarized as follows:

1. 3R projects are intended to extend the service life of the existing facility and to return its features to a condition of structural or functional adequacy.

2. 3R projects are intended to enhance highway safety.

3. 3R projects are intended to incorporate cost-effective, practical improvements to the geometric design of the existing facility.

11-2.01.03 Approach

The Department's approach to the geometric design of 3R projects is to adopt, where justifiable, a revised set of numerical criteria. The design criteria throughout the other *Manual* chapters provide the frame of reference for the 3R criteria. The following summarizes the approach which has been adopted:

1. Design Speed. The tables in Section 11-2.09 present the 3R design speeds for rural arterials and rural collectors on the State highway system. Note that these speeds are lower than those for new construction/reconstruction projects, subject to the posted/regulatory speed limit.

2. Speed-Related Criteria. Many geometric design values are calculated directly from the design speed (e.g., vertical curves, horizontal degree of curvature). The 3R design speed is used to determine these speed-related criteria. For many speed-related elements, Chapter 11 presents an acceptable threshold value for the element which is considerably below the 3R design speed. For example, if the calculated design speed of an existing crest vertical curve is within 15 mph of the 3R project design speed and there is not an adverse

accident history, the existing crest vertical curve may be retained in the project design without a design exception.

3. Cross Section Widths. The criteria in Chapter 2 have been evaluated relative to the typical constraints of 3R projects. Where justifiable, the values of the cross section width criteria have been reduced. See Section 11-2.03 for more discussion on cross section widths.

4. Other Design Criteria. The Department's *Design Manual* contains many other details on proper geometric design techniques. These criteria are obviously applicable to new construction and reconstruction. For 3R projects, these criteria have been evaluated and a judgment has been made on their proper application to 3R projects. Unless stated otherwise in this Chapter, the criteria in other chapters apply to 3R projects and should be incorporated if practical.

5. NHS Projects. For 3R projects on NHS facilities, it is not acceptable to propose a design value which is less than the value for the existing facility. For example, the proposed roadway width must equal or exceed the existing roadway width.

11-2.01.04 3R Project Evaluation

The designer should evaluate available data when determining the geometric design of 3R projects. The necessary evaluations presented for 3R projects are based on the FHWA Technical Advisory T5040.28 "Developing Geometric Design Criteria and Processes for Non-freeway RRR Projects." These are:

1. Conduct Field Review. The designer will normally schedule a thorough field review of the proposed 3R project. The Department's Location and Design Committee typically attends the field review. The Committee consists of representatives from the Roadway Design Division, the Construction Division, the District, the Environmental/Location Division and (if appropriate) FHWA. One objective of the field review will be to identify potential safety hazards and potential safety improvements to the facility.

2. Document Existing Geometrics. The Location and Design Committee will normally review the as-built plans and combine this with the field review to determine the existing geometrics within the project limits. This review includes lane and shoulder widths, horizontal and vertical alignment, intersection geometrics and the roadside safety design.

3. Accident Experience. The accident data within the limits of the 3R project will be evaluated. Accident data is available from the Planning Division. The following accident data analyses should be conducted:

   a. Accident Rate versus Statewide Average (for that type facility). This will provide an overall indication of safety problems within the 3R project limits.

   b. Accident Analysis by Type. This will indicate if certain types of accidents are a particular problem. For example, a disproportionate number of head-on and/or sideswipe accidents may indicate inadequate roadway width. A disproportionate number of fixed-object accidents may indicate an inadequate roadside clear zone.

   c. Accident Analysis by Location. Accidents may cluster about certain locations, such as a horizontal curve or intersection. In particular, the analysis should check to see if any sites on the Department's list of high-accident locations, as identified by the Department's accident data system, fall within the proposed project limits.

4. Speed Studies. It may be appropriate to review existing speed studies in the vicinity of the project and, if necessary, conduct a speed study to assist in determining the design speed of the 3R project. In addition, it may be desirable to conduct spot speed studies at specific locations (e.g., in advance

of a horizontal or vertical curve) to assist in the determination of geometric design improvements.

5. Traffic Volumes. The Location and Design Committee will examine the current and predicted traffic volumes within the limits of the 3R project. This may influence the decisions on the extent of geometric improvement. Section 11-2.02 discusses the Department's criteria for highway capacity analyses on 3R projects.

6. Early Coordination for Right-of-Way Acquisition. Significant R/W acquisitions are typically outside the scope of 3R projects. However, the field, accident and/or speed studies may indicate the need for selective safety improvements which will require R/W purchases. Therefore, the Location and Design Committee should, as early as feasible, determine improvements which will be incorporated into the project design and initiate the R/W acquisition process.

7. Pavement Condition. 3R projects are often programmed because of a significant deterioration of the pavement structure. The extent of deterioration will influence the decision on whether a project should be designed using the 3R design criteria or whether it should be designed using reconstruction criteria. In addition, all 3R projects will include a pavement surface which meets the Department's skid resistance requirements.

   Whenever the proposed pavement improvement is major, it may be practical to include significant geometric improvements (e.g., lane and shoulder widening) in the project design. However, the proper level of geometric improvement is often determined by many factors other than the extent of pavement improvement. These include available right-of-way, traffic volumes, accident experience and available funds for the project. Therefore, it may be appropriate for the 3R project to include, for example, full-depth pavement reconstruction and minimal geometric improvement, if deemed proper to meet the safety and operational objectives of the 3R program.

8. Geometric Design of Adjacent Highway Sections. The Location and Design Committee will examine the geometric features and operating speeds of highway sections adjacent to the 3R project. This includes investigating whether or not any highway improvements are in the planning stages. The 3R project should provide design continuity with the adjacent sections. This involves a consideration of factors such as driver expectancy, geometric design consistency and proper transitions between sections of different geometric designs.

9. Physical Constraints. The physical constraints within the limits of the 3R project will often determine what geometric improvements are practical and cost effective. These include topography, adjacent development, available right-of-way, utilities and environmental constraints (e.g., wetlands).

10. Traffic Control Devices. All signing and pavement markings on 3R projects must meet the criteria of the *Manual on Uniform Traffic Control Devices* (MUTCD). The Traffic Engineering Division is responsible for selecting and locating the traffic control devices on the project. The Location and Design Committee will work with the Traffic Engineering Division to identify possible geometric and safety deficiencies which will remain in place (i.e., no improvement will be made). These include:

    a. narrow bridges,
    b. horizontal and vertical curves which do not meet the 3R criteria, and
    c. roadside hazards within the clear zone.

11. Economics. 3R projects are intended to protect the existing economic investment and to derive the maximum economic benefit from the Department's existing highway system. Therefore, economic factors (i.e., the cost of improvement versus the anticipated benefit) are a major consideration in determining which geometric design improvements are practical and reasonable.

12. Potential Impacts of Various Types of Improvements. 3R projects may impact the social, economic and environmental nature of the surrounding land and development. In particular, the existing right-of-way may severely restrict the practical extent of geometric improvements.

Once the project evaluation is completed, the Location and Design Committee will produce a report which will recommend the project scope of work for the proposed improvement (i.e., 3R or reconstruction).

11-2.02 Design Controls

11-2.02.01 Design Speed

Reference: Section 2-2.0

Tables 11-2C and 11-2D present the 3R criteria for design speed. In addition, the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility.

11-2.02.02 Highway Capacity

Reference: Section 2-3.0

Three major factors determine the results of a capacity analysis. Their specific application to 3R projects is discussed below:

1. Level of Service (LOS). Tables 11-2C and 11-2D provide the LOS criteria for 3R projects.

2. Design Volume. Desirably, all elements of the facility will meet the LOS criteria for a DHV and/or ADT determined for ten years beyond the expected construction completion date. At a minimum, the highway facility should accommodate the LOS for the DHV and/or ADT based on current traffic volumes.

3. Capacity Analysis. The analytical techniques in the *Highway Capacity Manual* will be used to conduct the capacity analysis.

The designer should obtain from the Planning Division the traffic data necessary to determine the level of improvement. At a minimum, this will include current ADT, DHV, percent of trucks and buses, turning movements at intersections and any known future traffic impact.

11-2.03 Cross Sections

Reference: Section 2-6.03

11-2.03.01 Roadway Widths

Chapter 2 presents the Department's criteria for roadway widths for new construction and reconstruction. Tables 11-2C and 11-2D present the lane and shoulder widths for rural 3R projects on State highways. In general, these widths have been established considering the minimum acceptable width for the element from an operational and safety perspective; considering what will be available for a practical improvement on a "typical" 3R project; and considering that, in general, it is better to improve more miles to a lower level than to improve fewer miles to a higher level. All of these considerations are consistent with the overall objectives of the Department's 3R program.

The roadway width of the existing highway should be evaluated against the criteria in the 3R tables. The Department's policy on the application of these widths in the tables is as follows:

1. Existing Width Does Not Meet the 3R Width. The designer should consider widening the element. If the decision is made to widen the lane or shoulder

width, the designer should provide a width which at least meets the 3R criteria. This will be sufficient for the majority of 3R projects. However, if practical, it may be appropriate to widen the roadway width to meet new construction/ reconstruction criteria.

2. Existing Width Does Meet the 3R Width. In most cases, it will not be cost effective to widen the roadway. However, if practical, it may be appropriate to widen the width to meet new construction/reconstruction criteria.

11-2.03.02 Median Width

An existing multilane, divided highway (non-freeway) may be improved as a 3R project. If so, the existing median width will be acceptable.

If an existing 2-lane highway is being converted to a divided highway (i.e., a new 2-lane roadway will be constructed), the median width should be according to the Department's criteria for new construction.

11-2.03.03 Bridges

Scope of Work

Several bridges may be within the limits of the 3R project. The scope of work may be any of the following:

1. Bridge Replacement. Depending upon the extent of the structural deficiencies, if any, it may be economical to replace the entire bridge (i.e., superstructure, substructure and foundation).

2. Bridge Reconstruction. If the existing superstructure is structurally deficient but the substructure/foundation is structurally sound, the superstructure may be reconstructed or replaced as part of the 3R project.

3. Bridge Deck Rehabilitation. If the existing bridge deck is structurally deficient, it may be rehabilitated as part of the 3R project. In addition, where the bridge deck is structurally sound but its width is inadequate (i.e., the bridge is functionally deficient), the bridge deck may be rehabilitated solely to widen the bridge deck. Bridge deck widening may then require work to the superstructure and/or substructure.

4. Existing Bridge to Remain In Place. If an existing bridge is structurally sound and if it meets the Department's design loading capacity, it is unlikely to be cost effective to improve the geometrics of the bridge. These are considered existing bridges to remain in place. However, if the geometric deficiencies are severe and/or if there has been an adverse accident experience at the bridge, it may be warranted to rehabilitate the bridge deck to widen the bridge or to make other improvements.

   In some cases, only the bridge substructure (e.g., abutments, piers) and/or foundation (e.g., footings, piles) may require rehabilitative work. For the purpose of applying the 3R geometric design criteria, these may be considered existing bridges to remain in place.

5. Bridge Rails/Transitions. For reconstructed bridges or rehabilitated bridge decks, the existing bridge rails and approaching guardrail-to-bridge-rail transitions must be upgraded to meet current Department criteria. For existing bridges to remain in place within the limits of a 3R project, the Bridge Division will evaluate the adequacy of the existing bridge rail to determine if it should be upgraded. The roadway designer will evaluate the adequacy of the existing approaching bridge rail transition for needed upgrading. See Chapter 9 for more information on guardrail-to-bridge-rail transitions.

Bridge Width

The following will apply to the evaluation and improvement to the width of bridges within the limits of a 3R project:

1. Bridge Replacement. For this scope of work, the full approach roadway width according to new construction criteria will be carried across the bridge. However, bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to economic, traffic and safety studies.

2. Bridge Reconstruction/Bridge Deck Rehabilitation. For these scopes of work, the full approach 3R roadway width will be carried across the bridge. However, bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to economic, traffic and safety studies.

3. Existing Bridge to Remain in Place. The existing width of bridges proposed to remain in place will be evaluated against the criteria in Tables 11-2C and 11-2D. If the existing width does not meet these criteria, the designer must either widen the bridge as part of the 3R project or seek a design exception. However, bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to economic, traffic and safety studies.

   All bridges which are narrower than the approach roadway width (including shoulders) should be evaluated for special narrow bridge treatments. At a minimum, the signing and pavement markings must meet the criteria of the MUTCD. In addition, NCHRP 203 *Safety at Narrow Bridge Sites* provides criteria specifically for narrow bridges (e.g., special pavement markings). The designer, in coordination with the Traffic Engineering Division, should evaluate the value of these additional treatments at the bridge site.

Horizontal/Vertical Alignment

Except for bridge replacements, it is unlikely to be cost effective to improve the existing horizontal or vertical alignment for a bridge within the limits of a 3R project.

11-2.03.04 Right-of-Way

The acquisition of significant amounts of right-of-way is usually outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. Wherever practical, additional right-of-way should be secured to allow cost-effective geometric and roadside safety improvements.

11-2.04 Horizontal Alignment

Reference: Chapter 3

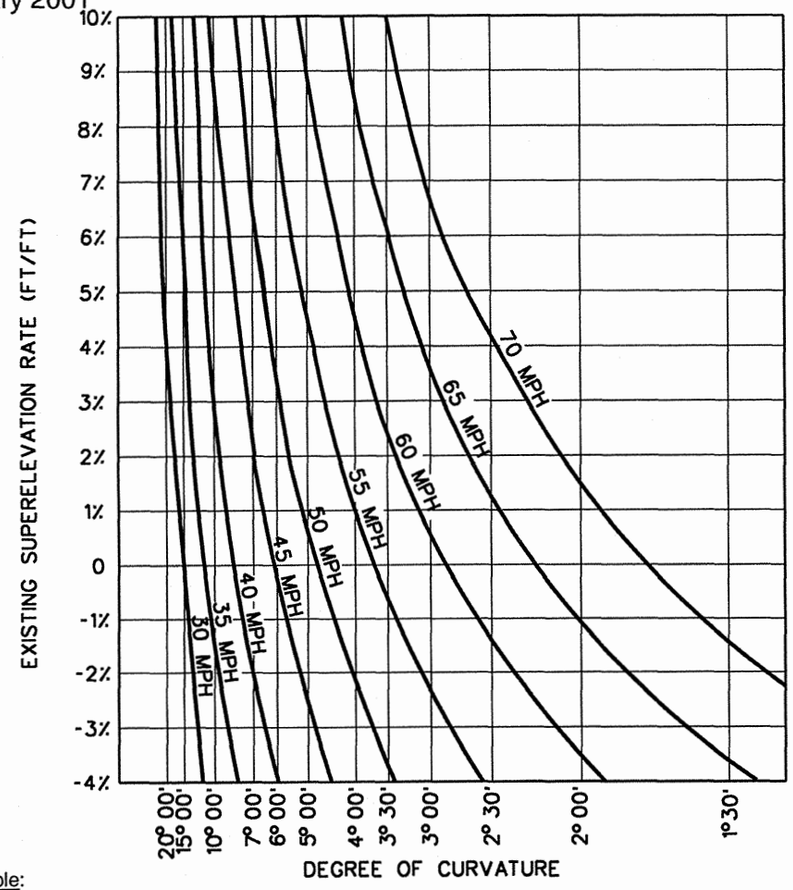
Chapter 3 discusses horizontal alignment criteria. These criteria will apply to 3R projects, except where discussed in the following paragraphs.

Figure 11-2A will be used to determine the calculated design speed of an existing horizontal curve. This may require field measurements to attain the as-built information. This design speed should be compared to the design speed of the 3R project as determined from Tables 11-2C and 11-2D.

Figure 11-2A can only be used to decide if corrective action should be considered. Once the decision has been made to improve the curve, the designer should use the criteria in Chapter 3 to determine the proper combination of degree of curvature and superelevation to meet the 3R design speed.

Figure 11-2A can also be used to determine the appropriate advisory speed for signing curves which are below the 3R design speed but will remain in place.

If the existing curve satisfies the 3R design speed, the designer will not normally need to check other details of the horizontal curve (e.g., warrants for spirals, superelevation runoff length, distribution of superelevation between tangent and curve).



Example:

The design speed for the 3R project will be 55 mph. An existing curve within the project limits has the following data:

D = 4°
e = 3.0%

Using the figure, the existing curve is adequate for a design speed of 55 mph. Therefore, no improvement is necessary. Note that if Table 3-4A is used, the necessary superelevation rate is 8.3%.

> These radii will be used to determine if an existing horizontal curve is acceptable. See Table 3-4A if any improvements are made to the curve.

CRITERIA FOR RETAINING EXISTING HORIZONTAL CURVES
(Rural 3R Projects (All Design Speeds) and Urban 3R Projects (V ≥ 50 mph))

Figure 11-2A

11-2.05 Vertical Alignment

Reference: Chapter 4

11-2.05.01 Grades

Reference: Section 4-3.0

Tables 11-2C and 11-2D present the Department's criteria for maximum and minimum grades on 3R projects. The maximum grades are 2% steeper than those for new construction and reconstruction projects.

11-2.05.02 Crest Vertical Curves

Reference: Section 4-4.02

Section 4-4.02 presents the Department's criteria for the design of crest vertical curves for new construction and reconstruction projects. This information will be used to determine the calculated design speed of an existing crest vertical curve, which will then be compared to the 3R design speed. The following summarizes the 3R design criteria for crest vertical curves:

1. ADT < 1500. In the absence of an adverse accident history, all existing crest vertical curves are acceptable without further evaluation regardless of the design speed of the vertical curve.

2. ADT > 1500. In the absence of an adverse accident history, all existing crest vertical curves with a design speed within 15 mph of the 3R design speed are acceptable without further evaluation. The designer should evaluate the reconstruction of the crest vertical curve if its calculated design speed is more than 15 mph less than the 3R design speed and if the crest hides from view

major hazards such as intersections, sharp horizontal curves or narrow bridges.

3. Angle Points. It is acceptable to retain an existing "angle" point (i.e., no vertical curve) of 0.5 percent or less.

If the existing crest vertical curve satisfies these criteria, the designer will not normally need to check other details of the vertical curve (e.g., minimum length of vertical curve).

If the decision is made to flatten the crest vertical curve, the designer will desirably design the reconstructed curve to meet the criteria for new construction/reconstruction in Section 4-4.02. The K-values in Section 4-4.02 will provide for minimum stopping sight distance to a 6-in. height of object. However, at a minimum, it is acceptable to use the K-values in Table 11-2A to reconstruct the crest vertical curve on a 3R project. These values provide minimum stopping sight distance to an 18-in. height of object, which is approximately the height of the taillights on a passenger car.

Table 11-2A
K-VALUES FOR RECONSTRUCTED CREST VERTICAL CURVES
(Minimum for 3R Projects)

Design Speed (mph)	Minimum SSD Rounded for Design (ft)	K-Values Rounded for Design $K = S^2/1917$
30	185	18
40	275	40
50	380	76
55	435	99
60	505	133
65	550	158
70	615	198

$$L = \frac{AS^2}{1917} = KA$$

where:

S = the sight distance for the design speed (ft)
L = the length of vertical curve (ft)
A = the algebraic difference between grades (%)
K = the horizontal distance required (in ft) to produce a 1% change in gradient

Example 11-2.1

Given: An existing crest vertical curve within the limits of a 3R project has the following data:

A = algebraic difference = 6%
L = length of curve = 300 ft
V = 3R design speed = 55 mph
ADT = 2000 vpd
There is an at-grade intersection just beyond the crest.

Problem: Evaluate the adequacy of the existing crest vertical curve.

Solution: Use the following steps:

1. Determine the design speed of the existing crest.

   Use the equation in Table 4-4A to calculate S for the existing crest:

$$S^2 = \frac{1329\ L}{A} = \frac{(1329)\ (300)}{6} = 66{,}450$$

$$S = 258\ ft$$

2. Determine if the designer should evaluate reconstructing the crest.

   The available sight distance of 258 ft is adequate for a vehicular speed of approximately 38 mph. This is more than 15 mph less than the 3R project design speed of 55 mph. In addition, the ADT >1500 and the crest hides from view an intersection. Therefore, the designer should evaluate reconstructing the crest vertical curve.

3. Determine course of action.

   The designer will select from the following options:

   a. Reconstruct to satisfy new construction/reconstruction criteria. From Table 4-4A, K = 143 for 55 mph and L will equal 143 x 6 = 858 ft.

   b. Reconstruct to satisfy the criteria in Table 11-2A. K = 99 for 55 mph and L will equal 99 x 6 = 594 ft.

   c. Leave in place and seek a design exception. See Section 2-8.0.

   The selection will be based on construction cost differences, environment and right-of-way impacts, accident history, traffic volumes, etc.

11-2.05.03 Sag Vertical Curves

Reference: Section 4-4.03

Section 4-4.03 presents the Department's criteria for the design of sag vertical curves for new construction and reconstruction. These criteria are based on designing the sag to allow the vehicle's headlights to illuminate the pavement for a distance equal to the stopping sight distance for the design speed. For 3R projects, the following will apply:

1. Evaluation. The comfort criteria represent the minimum criteria for the retention of an existing sag vertical curve. Figure 11-2B presents the comfort criteria. If an existing sag does not meet these criteria, then the designer should consider flattening the sag vertical curve.

2. Corrective Action. If the decision is made to flatten the sag, the design should meet the criteria for headlight sight distance in Section 4-4.03. At a minimum, it is acceptable for the re-designed sag to meet the comfort criteria in Figure 11-2B. Proper illumination of the sag vertical curve may be considered as a mitigating measure.

3. Angle Points. It is acceptable to retain an existing "angle" point (i.e., no vertical curve) of 0.5 percent or less.

If the existing sag vertical curve satisfies the above criteria, the designer will not normally need to check other details of the vertical curve (e.g., minimum length of vertical curve).

11-2.06 Intersections At-Grade

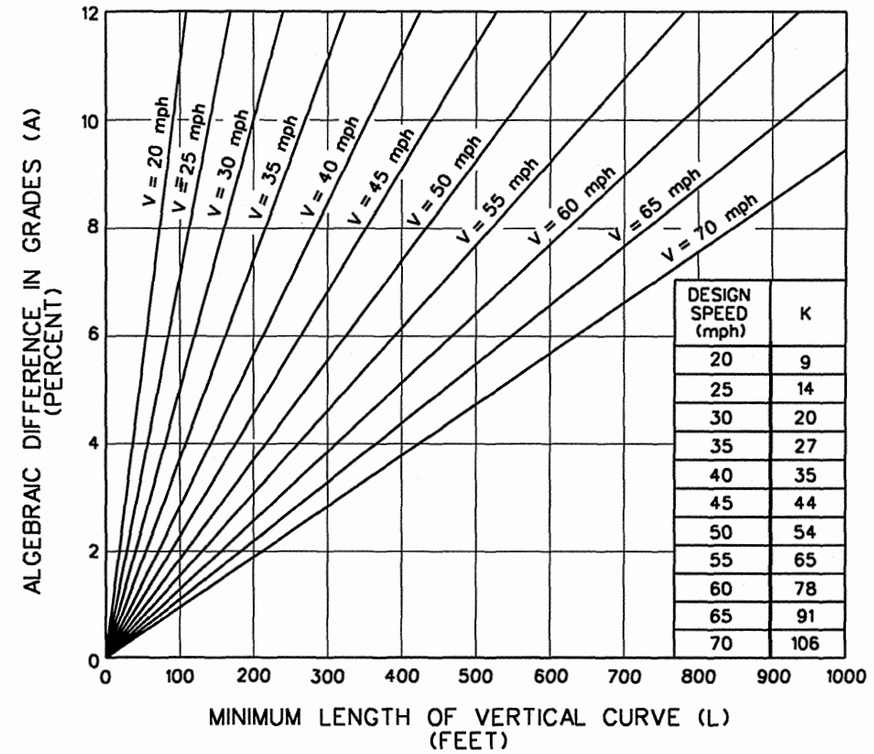
Reference: Chapter 6

Chapter 6 provides criteria for the detailed design of intersections at-grade. Where practical, these criteria apply to 3R projects and should be implemented. The following sections indicate areas where modifications to the intersection criteria may be appropriate for 3R projects.

11-2.06.01 Intersection Sight Distance

Reference: Section 6-6.0

The criteria in Section 6-6.0 on intersection sight distance will apply to 3R projects. However, note that it is often impractical to meet the sight flare criteria on 3R projects.



DESIGN SPEED (mph)	K
20	9
25	14
30	20
35	27
40	35
45	44
50	54
55	65
60	78
65	91
70	106

$$L = \frac{AV^2}{46.5}$$

$$L = KA$$

Where:
L = Length of vertical curve, ft
A = Algebraic difference between grades, %
K = Horizontal distance required to effect a 1% change in gradient, ft

LENGTH OF SAG VERTICAL CURVES
(3R Projects)

Figure 11-2B

11-2.06.02 Auxiliary Turning Lanes

Reference: Section 6-4.0

Section 6-4.0 presents warrants for right- and left-turn lanes. These criteria apply to 3R projects. Section 6-4.0 also presents design details for auxiliary turning lanes, and these should be met for 3R projects.

11-2.06.03 Driveway Design

Reference: Section 6-9.0

The criteria in Section 6-9.0 on the design of rural driveways will apply to 3R projects.

11-2.07 Special Design Elements

Reference: Chapter 8

Chapter 8 provides the Department's criteria and design details for many special design elements (e.g., bikeways). The designer should review this Chapter to determine if these criteria apply to the 3R project. For example, Section 8-2.0 presents accessibility criteria for disabled individuals. These criteria apply directly to all 3R projects.

11-2.08 Roadside Safety

Reference: Chapter 9

The roadside safety criteria in Chapter 9 have been developed explicitly for new construction and reconstruction. This includes criteria for clear zones and barrier layout details (e.g., flare rates, length of need). The criteria in Chapter 9 should apply, as practical, to the roadside safety design of 3R projects. The following sections indicate where these criteria may be modified.

11-2.08.01 Clear Zones

Reference: Section 9-2.0

Attempting to achieve a roadside clear zone on a 3R project can cause significant problems. The roadside environment along existing highways is typically cluttered with any number of natural and man-made obstacles. To remove or relocate these obstacles can present formidable problems and public opposition, and it can be very costly. On the other hand, the designer cannot ignore the consequences to a run-off-the-road vehicle. Therefore, the designer must exercise considerable judgment when determining the appropriate clear zone on the 3R project. The designer should consider the following:

1. Accident Data. The designer should review the accident data to estimate the extent of the roadside safety problem. In particular, there may be sites where clusters of run-off-the-road accidents have occurred.

2. Utilities. Utility poles are a common roadside obstacle on 3R projects. Relocation is mandatory when the utility poles physically interfere with construction. Relocations for safety benefits must be evaluated on a project-by-project basis. Poles should be located as near as practical to the right-of-way line. In restricted right-of-way areas, every effort should be made to provide the clear zone used in design. Consider the use of armless single-pole construction with vertical configuration of wires and cables and/or other special construction, as may be appropriate. In urban areas, the designer should also consider burying the utilities underground where relocation is impractical.

3. Application. The designer may consider a selective application of the roadside clear zone criteria. Along some sections of highway, it may be practical to pro-

vide the desirable clear zone while, along other sections, it may be impractical. In addition, some obstacles will be more hazardous than others. Judgment will be necessary for the application of the clear zone criteria.

4. Public. Public acceptance of widened clear zones can be a significant issue, especially where the removal of trees is being considered. The designer must judge the community impact and subjectively factor this into the decision-making process.

5. Safety Appurtenances. Installing barriers or impact attenuators is an alternative to providing a wider clear zone. Section 9-3.0 presents warrants for barriers. However, this can lead to lengthy runs of barrier along the roadside. The designer should realize that the barrier warrants are based on the relative severity between hazard and barrier; they do not address the question of whether or not a barrier installation is cost effective. Therefore, on 3R projects, the designer must judge whether or not a barrier should be installed to shield a hazard within the clear zone. See Section 11-2.08.05 for more discussion.

Section 9-2.0 presents the Department's criteria for roadside clear zones on new construction and reconstruction projects. On 3R projects, the most desirable objective will be to provide a clear zone equal to the criteria in Section 9-2.0. The designer should make every reasonable effort to achieve this objective. Where this is not practical, the clear zone criteria in Table 11-2B may be used for 3R projects. Note that these clear zone distances apply to facilities without curbs and on fill sections. As indicated in Table 11-2B, the designer will reference the applicable discussion in Section 9-2.0 to determine the clear zone on non-recoverable fill slopes and on curbed facilities. For ditch sections, use the following procedure:

1. Check Fore Slope. Use Table 11-2B to determine the clear zone based on the ditch fore slope.

2. Check Location of the Toe of Back Slope. Based on the distance from Step #1, determine if the toe of the back slope is within the clear zone. If the toe is at or beyond the clear zone, then the designer usually need only consider roadside hazards within the clear zone on the fore slope or within the ditch. If the toe is within the clear zone, the designer should evaluate the practicality of relocating the toe of back slope. If the toe of back slope will remain within the clear zone, Step #3 will apply.

3. Determine Clear Zone on Back Slope (Earth Cuts). If the toe of the back slope is within the clear zone distance from Step #1 above, provide a clear zone on the back slope. This clear zone will be a distance beyond the toe of back slope as follows:

   a. Calculate the percentage of the clear zone (from Step #1) available to the toe of the back slope.

   b. Subtract this percentage from 100% and multiply the results by the clear zone for the back slope in Table 11-2B(b).

   c. Add the available clear zone to the toe of the back slope to the value determined in Step #3b. This yields the required clear zone from the edge of traveled way to a point on the back slope.

For auxiliary lanes such as climbing lanes, passing lanes, etc., the clear zone will be determined assuming the absence of the auxiliary lane and will be measured from the edge of the traveled way.

11-2.08.02 Embankments (Fill Sections)

Reference: Section 9-3.03

If practical, any fill slope steeper than 3:1 should be flattened to a 3:1 or flatter slope. If this is not practical, guardrail will normally be installed where the slope is steeper than 3:1 and the height of the embankment is more than 10 ft.

Table 11-2B

RECOMMENDED CLEAR ZONE DISTANCES (In Feet)
(3R Non-Freeway Projects)

(a) Fill Slopes/Uncurbed Facilities

Design Speed (mph)	Design ADT	Recoverable (Figure 9-2A(a))		Non-recoverable (Figure 9-2A(b))
		6:1 or Flatter	5:1 to 4:1	
35 or Less	Under 750 750 - 1500 1501 - 6000 Over 6000	3.5 - 5 5 - 6 6 - 7 7 - 8	3.5 - 5 6 - 7 7 - 8 8 - 9	SEE PROCEDURE IN SECTION 9-2.02.**
45-50	Under 750 750 - 1500 1501 - 6000 Over 6000	5 - 6 7 - 8 8 - 9 10 - 11	6 - 7 8 - 10 10 - 13 12 - 14	
55	Under 750 750 - 1500 1501 - 6000 Over 6000	6 - 7 8 - 9 10 - 11 11 - 12	7 - 9 10 - 12 12 - 15 13 - 16*	
60	Under 750 750 - 1500 1501 - 6000 Over 6000	8 - 9 10 - 12 13 - 15 15 - 16*	10 - 12 13 - 16* 16 - 20* 18 - 22*	
70	Under 750 750 - 1500 1501 - 6000 Over 6000	9 - 10 12 - 13 14 - 16* 15 - 17*	10 - 13 14 - 18* 17 - 21* 19 - 23*	

* *The clear zone distance may be limited to 15 ft for practicality and to provide a consistent roadway template.*

** *Use the clear zone values from Table 11-2B(a) in the procedure in Section 9-2.02.Notes:*

1. *All distances are measured from the edge of traveled way.*
2. *The "Design ADT" will be the total ADT for both directions of travel for the design year used for the project. This applies to both divided and undivided facilities.*

(b) Back Slopes

Design Speed (mph)	Design ADT	3:1	5:1 to 4:1	6:1 or Flatter
35 or Less	Under 750 750 - 1500 1501 - 6000 Over 6000	3.5 - 5 5 - 6 6 - 7 7 - 8	3.5 - 5 5 - 6 6 - 7 7 - 8	3.5 - 5 5 - 6 6 - 7 7 - 8
45-50	Under 750 750 - 1500 1501 - 6000 Over 6000	4 - 5 5 - 6 6 - 7 7 - 8	4 - 5 5 - 7 7 - 8 9 - 10	5 - 6 7 - 8 8 - 9 10 -11
55	Under 750 750 - 1500 1501 - 6000 Over 6000	4 - 5 5 - 6 7 - 8 8 - 9	5 - 6 7 - 8 8 - 9 10 - 11	5 - 6 8 - 9 10 - 11 11 - 12
60	Under 750 750 - 1500 1501 - 6000 Over 6000	5 - 6 6 - 7 7 - 9 10 - 11	6 - 7 8 - 9 9 - 11 12 - 13	7 - 8 10 - 11 12 - 13 13 - 14
70	Under 750 750 - 1500 1501 - 6000 Over 6000	5 - 6 6 - 8 8 - 10 11 - 12	7 - 8 9 - 10 11 - 12 13 - 15	7 - 8 10 - 11 13 - 14 14 - 15

11-2.08.03 Intersecting Slopes

Reference: Section 9-3.05

On 3R non-freeway projects, intersecting slopes should meet the following criteria (either for retention of existing slopes or for flattening existing slopes):

1. Medians. For existing medians on multilane highways, the maximum intersecting slope should be 10:1. This includes the sides of any drainage pipes which pass beneath, for example, a median crossover.

2. Facilities with Type 2 Access (Partial Control) ($V \geq 50$ mph). For intersecting slopes other than in the median, the desirable slope is 10:1; the maximum slope is 6:1. This includes the sides of any drainage pipes which pass beneath, for example, an intersecting highway.

3. All Other Facilities. In the absence of an adverse accident history related to intersecting slopes, all existing intersecting slopes may be retained. In addition, an intersecting slope may require a drainage structure beneath the intersecting road, driveway or median crossover which is parallel to the traffic on the mainline. Any parallel drainage structures within the clear zone should desirably match the side slope and be safely treated. Although many of these structures are small and present a minimal target, the addition of pipes and bars perpendicular to traffic can reduce wheel snagging in the culvert openings. Single pipes with diameters greater than 24 in. and which are within the clear zone should include a grate across the opening. For multiple-pipe installations, the use of grates for even smaller pipes should be considered.

11-2.08.04 Roadside Ditches

In the absence of an adverse accident history related to roadside ditches parallel to the roadway, all existing roadside ditch configurations may be retained.

11-2.08.05 Safety Appurtenances

Reference: Chapter 9 (Various)

During the design of a 3R project, all existing safety appurtenances should be examined to determine if they meet the latest safety performance and design criteria. This includes guardrail, median barriers, impact attenuators, sign supports, luminaire supports, etc. Typically, all safety appurtenances will be upgraded to meet the most recent criteria. This also includes the Department's criteria for the layout of guardrail, median barriers and impact attenuators.

Barrier warrants on 3R projects can be especially difficult to resolve. Basically, the evaluation process will be:

1. Determine if barrier is warranted. However, also see Comment #5 in Section 11-2.08.01.

2. If an existing run of barrier is located where none is warranted, remove the barrier.

3. If barrier is warranted, consider removing or relocating the hazard; reducing the hazard (e.g., flattening a slope); or making it breakaway.

4. If the hazard cannot be eliminated and a barrier is considered cost effective, then install a barrier. For existing runs of barrier, ensure that they meet the applicable current performance and design criteria, including:

   a. operational acceptability (hardware, height, etc.),
   b. dynamic deflection criteria,
   c. length of need,
   d. flare rate,
   e. lateral placement,
   f. placement on slopes and behind curbs,

g. end treatments, and
h. transitions.

The designer may exercise some judgment and apply cost-effective considerations when evaluating existing installations. For example, an existing barrier installation may satisfy all current Department criteria (crashworthiness, end treatment, flare rate) except that its length may be one standard guardrail section (12.5 ft) short of that required by the criteria in Section 9-5.0. In this case, it likely would not be worth the expense to remove the existing end treatment, extend the barrier by one standard length and install a new end treatment.

A common problem on 3R projects will be barrier height because of a pavement overlay or rehabilitation. Each existing barrier run which will remain must be considered individually. As a general rule, the designer should seriously consider raising the barrier where its height after construction will not be within 3 in. of the required height for a new installation.

Another potential problem for 3R projects is the lateral placement of roadside barriers. Where guardrail is placed to shield a fill slope, there will desirably be 2 ft between the back of the barrier post and the break (PI) in the fill slope. On 3R projects this may not be practical, and it may be necessary to place the post at the break in the slope. Where the fill slope is 3:1 or flatter, this does not materially affect the performance of the barrier. However, where the slope is steeper than 3:1, a post 7 ft in length should be used.

11-2.09 3R Tables of Geometric Design Criteria

Tables 11-2C and 11-2D present the Department's 3R criteria for the geometric design of rural arterials and rural collectors on State highways.

11-2.10 Exceptions to Geometric Design Criteria

Reference: Section 2-8.0

The discussion in Section 2-8.0 on exceptions applies equally to the geometric design of 3R projects. The only difference, obviously, is that the designer will be evaluating the proposed design against the criteria presented in Section 11-2.0.

Table 11-2C

GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (3R Projects)

	DESIGN ELEMENT		Design ADT T	Manual Section	2-Lane 0 - 2000 T ≥ 10%	2-Lane 0 - 2000 T < 10%	2-Lane > 2000 T ≥ 10%	2-Lane > 2000 T < 10%	Multilane (1)
Design Controls	Design Forecast Year			11-2.02	Desirable: 10 Years Minimum: Current				Desirable: 10 Years Minimum: Current
	✻Design Speed (2)			11-2.02	≤ 55 mph				65 mph
	Control of Access			2-4.0	Control by Regulation (Type 3)				Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)
	Level of Service			11-2.02	Desirable: B Minimum: D				Desirable: B Minimum: D
Cross Section Elements	✻Lane Width			11-2.03	V≤45: 11′ V≥50: 12′	11′	12′		12′
	Outside Shoulder Width	✻Usable		11-2.03	V≤45: 2′ V≥50: 3′		6′	5′	4′
		Surfaced			(3)				(3)
	Median Shoulder Width	✻Usable		11-2.03	N/A				4′
		Surfaced			N/A				(3)
	✻Cross Slope	Travel Lane		2-6.03	2%				2%
		Shoulder			4%				4%
	Auxiliary Lanes	Lane Width		2-6.03	Desirable: 12′ Minimum: 11′				Desirable: 12′ Minimum: 11′
		Usable Shoulder Width			2′				4′
	Median Width			11-2.03	N/A				Existing
	✻Reconstructed/ Rehabilitated Bridges (5)	Structural Capacity		11-2.03	HS-20				HS-20
		Minimum Width (4)			V≤45: 26′ V≥50: 30′	V≤45: 26′ V≥50: 28′	36′	34′	Approach Roadway Width
	✻Existing Bridges to Remain in Place	Structural Capacity		11-2.03	HS-20				HS-20
		Minimum Width (5)			Approach Traveled Way + 2′		Approach Traveled Way +4′		Approach Traveled Way + 4′ (Each Roadway)
	Minimum Right-of-Way Width			11-2.03	(6)				(6)
	Desirable R/W Border Width (Beyond toe/top of fill/cut slope)				(6)				(6)
	Roadside Clear Zone	Guardrail		11-2.08	Usable Shoulder Width				Usable Shoulder Width
		Obstruction			See Note (7)				See Note (7)
	Slope Schedule	Cut: Foreslope (within clear zone)		2-6.03	4:1				4:1
		Cut: Depth of Ditch			1′				4′
		Cut: Backslope			3:1				3:1
		Fill: Height ≤ 10′			Desirable: 6:1 Maximum: 2:1				Desirable: 1:6 Maximum: 1:2
		Fill: Height > 10′			Desirable: 6:1 Maximum: 3:1				Desirable: 1:6 Maximum: 1:3
Alignment Elements	DESIGN SPEED				55 mph		60 mph		65 mph
	✻Minimum Stopping Sight Distance			2-6.02	435′		505′		550′
	✻Maximum Degree of Curvature			11-2.04	See Section 11-2.04				
	✻Superelevation Rate			11-2.04	See Section 11-2.04				
	✻Horizontal Sight Distance			3-5.0	(8)				
	✻Maximum Grades (9)	Level		11-2.05	5.5%		5%		5%
		Rolling			6.5%		6%		6%
	Minimum Grades			4-3.02	Desirable: 0.4% Minimum: 0.0%				
	✻Vertical Curve (K-values)			11-2.05	See Section 11-2.05				
	✻Minimum Vertical Clearance (Arterial Under)	New/Reconstructed Bridges (10)		2-6.03	16.5′				
		Existing Bridges			16′				
		Sign Truss (10)			17.5′				
	Vertical Clearance (Arterial Over Railroad)			2-6.03	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS
(3R Projects)

Footnotes to Table 11-2C

1. Multilane Facilities. The criteria in the table for 3R multilane facilities will apply to:

   a. an existing multilane facility, or
   b. an existing 2-lane facility which will become a divided highway.

   In the case of "b," the new roadway will be designed according to the Department's new construction criteria. See Table 2-7C.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Shoulder Surface Type. Generally, shoulders will not be surfaced except as approved in special conditions.

4. Bridge Width (New Bridges). For new bridges within the limits of a 3R project, the minimum width will be the full approach roadway width according to the Department's criteria for new construction. However, bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

5. Bridge Width (Reconstructed Bridges/Rehabilitated Bridges/Existing Bridges to Remain in Place). Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Right-of-Way Width and Border Width. The acquisition of significant amounts of right-of-way is typically outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. However, the designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

7. Roadside Clear Zone. See Section 11-2.08 for clear zones. All values are measured from the edge of traveled way.

8. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

9. Maximum Grades. Grades 1 percent steeper may be used on 1-way downgrades.

10. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

Table 11-2D
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(3R Projects)

	DESIGN ELEMENT		Design ADT T	Manual Section	2-Lane 0 - 1500 T ≥ 10%	2-Lane 0 - 1500 T < 10%	2-Lane 1501 - 2000 T ≥ 10%	2-Lane 1501 - 2000 T < 10%	2-Lane > 2000 T ≥ 10%	2-Lane > 2000 T < 10%	Multilane (1)
Design Controls	Design Forecast Year			11-2.02	Desirable: 10 Years Minimum: Current						Desirable: 10 Years Minimum: Current
	✳Design Speed (2)			11-2.02	≤ 55 mph						65 mph
	Control of Access			2-4.0	Control by Regulation (Type 3)						Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)
	Level of Service			11-2.02	Desirable: B Minimum: D						Desirable: B Minimum: D
Cross Section Elements	✳Lane Width			11-2.03	11′		V≤45: 11′ V≥50: 12′	11′	12′		12′
	Outside Shoulder Width	✳Usable		11-2.03	V ≤ 45: 2′ V ≥ 50: 3′		V≤45: 2′ V≥50: 3′		6′	5′	4′
		Surfaced			(3)						(3)
	Median Shoulder Width	✳Usable		11-2.03	N/A						4′
		Surfaced			N/A						(3)
	✳Cross Slope	Travel Lane		2-6.03	2%						2%
		Shoulder			4%						4%
	Auxiliary Lanes	Lane Width		2-6.03	Desirable: 12′ Minimum: 11′						Desirable: 12′ Minimum: 11′
		Usable Shoulder Width			2′						4′
	Median Width			11-2.03	N/A						Existing
	✳Reconstructed/ Rehabilitated Bridges	Structural Capacity		11-2.03	HS-20						HS-20
		Minimum Width (4)			V≤45: 26′ V≥50: 28′		V≤45: 26′ V≥50: 30′	V≤45: 26′ V≥50: 28′	36′	34′	Approach Roadway Width
	✳Existing Bridges to Remain in Place	Structural Capacity		11-2.03	HS-20						HS-20
		Minimum Width (5)			Approach Traveled Way + 2′ OR 22′, whichever is less		Approach Traveled Way +2′ OR 24′, whichever is less		Approach Traveled Way + 4′ OR 28′, whichever is less		Approach Traveled Way +4′ (Each Roadway) OR 28′, whichever is less
	Minimum Right-of-Way Width			11-2.03	(6)						(6)
	Desirable R/W Border Width (Beyond toe/top of fill/cut slope)				(6)						(6)
	Roadside Clear Zone	Guardrail		11-2.08	Usable Shoulder Width						Usable Shoulder Width
		Obstruction			See Note (7)						See Note (7)
	Slope Schedule	Cut: Foreslope (within clear zone)		2-6.03	V ≤ 45: 3:1 V ≥ 50: 4:1						4:1
		Cut: Depth of Ditch			V ≤ 45: 2′ V ≥ 50: 3′						___′
		Cut: Backslope			2:1						3:1
		Fill: Height ≤ 10′			2:1						Desirable: 6:1 Maximum: 2:1
		Fill: Height > 10′			3:1						Desirable: 6:1 Maximum: 3:1
Alignment Elements	DESIGN Speed				45 mph		50 mph		55 mph	60 mph	65 mph
	✳Minimum Stopping Sight Distance			2-6.02	325′		380′		435′	505′	550′
	✳Maximum Degree of Curvature			11-2.04	See Section 11-2.04						
	✳Superelevation Rate			11-2.04	See Section 11-2.04						
	✳Horizontal Sight Distance			3-5.0	(8)						
	✳Maximum Grades	Level		11-2.05	8.5%		8%		7.5%	7%	6.5%
		Rolling			9.5%		9%		8.5%	8%	7.5%
	Minimum Grades			4-3.02	Desirable: 0.4% Minimum: 0.0%						
	✳Vertical Curve (K-values)			11-2.05	See Section 11-2.05						
	✳Minimum Vertical Clearance (Collector Under)	New/Reconstructed Bridges (9)		2-6.03	16.5′						
		Existing Bridges			14.5′						
	Vertical Clearance (Collector Over Railroad)			2-6.03	23′						

✳Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(3R Projects)

Footnotes to Table 11-2D

1. Multilane Facilities. The criteria in the table for 3R multilane facilities will apply to:

   a. an existing multilane facility, or
   b. an existing 2-lane facility which will become a divided highway.

   In the case of "b," the new roadway will be designed according to the Department's new construction criteria. See Table 2-7D.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Shoulder Surface Type. Generally, shoulders will not be surfaced except as approved in special conditions.

4. Bridge Width (New Bridges). For new bridges within the limits of a 3R project, the minimum width will be the full approach roadway width according to the Department's criteria for new construction. However, bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

5. Bridge Width (Reconstructed Bridges/Rehabilitated Bridges/Existing Bridges to Remain in Place). Bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Right-of-Way Width and Border Width. The acquisition of significant amounts of right-of-way is typically outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. However, the designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

7. Roadside Clear Zone. See Section 11-2.08 for clear zones. All values are measured from the edge of traveled way.

8. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

9. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

11-3.0 3R PROJECTS (Freeways)

11-3.01 General

11-3.01.01 Background

The Department began construction of its freeway system in the 1950's and, today, the Mississippi system is nearing completion. The freeway system has introduced a level of mobility and safety for the traveling public which was unattainable without its special features, such as full control of access, wide roadway widths and high design speeds. In the past few decades, this system has carried traffic volumes in far greater proportion than its mileage within the State.

The freeway system requires periodic repair and upgrading which exceeds the limits of normal maintenance. In general, these capital improvements are referred to as 3R freeway projects (resurfacing, restoration and rehabilitation). As with non-freeway 3R projects, it is often impractical to fully apply new construction/reconstruction criteria to 3R projects without some qualifications. Therefore, the geometric design of 3R freeway projects requires special design considerations which are discussed in the following sections. If the project scope of work is reconstruction, Chapter 11 does not apply.

11-3.01.02 Objectives

The objective of a 3R freeway project is, within practical limits, to return the freeway to its original level of serviceability or to improve its serviceability to meet current and future demands. This objective applies to all aspects of the freeway's serviceability include:

1. structural adequacy,
2. drainage,
3. level of service for the traffic volume,
4. geometric design,
5. roadside safety, and
6. traffic control.

11-3.01.03 Approach

3R freeway projects are most often initiated to make a specific improvement to the freeway. Therefore, the Department's approach to the geometric design of 3R freeway projects is to selectively evaluate and improve the existing geometrics. Table 11-3A presents the Department's geometric design criteria for 3R projects. The 3R approach is summarized as follows:

1. Nature of Improvement. Identify the specific improvement intended for the 3R project. For example, improvements might include:

    a. pavement, resurfacing, restoration and/or rehabilitation, including shoulders (see Comment #5);

    b. improve roadway delineation;

    c. add auxiliary lanes;

    d. upgrade roadside safety;

    e. increase the length of one or more acceleration and/or deceleration lanes at an interchange;

    f. widen an existing bridge as part of a bridge deck rehabilitation project;

    g. eliminate a weaving area at an interchange; and/or

    h. improve the pavement and roadside drainage.

2. Numerical Criteria. Table 11-3A presents the Department's numerical criteria for the design of 3R projects. In particular, the footnotes to the table provide essential information on 3R projects. In addition, the information in the remainder of Section 11-3.0 provides criteria on other geometric design elements on 3R projects.

3. Secondary Impacts. Identify and evaluate any secondary impacts which may be precipitated by the primary freeway improvement. For example:

   a. The installation of a CMB may restrict horizontal sight distance.

   b. A pavement overlay may require the adjustment of barrier height.

4. Other Improvements. Identify any other severe geometric design deficiencies within the project limits which can be practically corrected without exceeding the intended project scope of work. The designer will exercise his/her judgment when determining any other improvements which may be justified.

5. Pavement Improvement. Whenever the proposed pavement improvement is major, it may be practical to include significant geometric improvements (e.g., lane and shoulder widening) in the project design. However, the proper level of geometric improvement is often determined by many factors other than the extent of pavement improvement. These include available right-of-way, traffic volumes, accident experience and available funds for the project. Therefore, it may be appropriate for the 3R project to include, for example, full-depth pavement reconstruction and minimal geometric improvement, if deemed proper to meet the safety and operational objectives of the 3R program.

11-3.01.04 3R Project Evaluation

Section 11-2.01 discusses factors which should be evaluated in the design of 3R non-freeway projects (e.g., accident records). These also apply to the evaluation of 3R freeway projects.

11-3.02 Geometric Design of 3R Freeway Projects

The following discusses the flexibility the designer has in the geometric design of a 3R freeway project.

11-3.02.01 Design for Original Construction

A specific geometric design element on an existing freeway (e.g., a crest vertical curve) may have met the applicable design criteria at the time of original construction but does not meet the Department's current criteria. In these cases, the design criteria used for horizontal alignment, vertical alignment and widths of median, traveled way and shoulders for 3R freeway projects may be the AASHTO freeway design criteria that were in effect at the time of original construction or inclusion into the Interstate system.

11-3.02.02 Design Speed

Reference: Section 2-2.0

In most cases, the 3R design speed will be as used at the time of original construction. In addition, the design speed should equal or exceed the anticipated posted speed limit after construction.

11-3.02.03 Traffic Volumes

Reference: Section 2-3.0

Some design elements on 3R freeway projects will require the selection of the DHV (e.g., level of service) or ADT (e.g., roadside clear zones). Desirably, the freeway will be designed to meet the geometric design criteria for traffic volumes determined for 20 years beyond the expected completion date. At a minimum, the designer will use 10 years beyond the completion date.

11-3.02.04 Vertical Clearances

The minimum vertical clearance for freeways passing beneath a structure should be 16 ft over the entire roadway width, including auxiliary lanes and shoulders.

11-3.02.05 Bridges

Reference: Section 2-6.03

Scope of Work

Several bridges may be within the limits of the 3R project. The scope of work may be any of the following:

1. Bridge Replacement. Depending upon the extent of the structural deficiencies, if any, it may be economical to replace the entire bridge (i.e., superstructure, substructure and foundation).

2. Bridge Reconstruction. If the existing superstructure is structurally deficient but the substructure/foundation is structurally sound, the superstructure may be reconstructed or replaced as part of the 3R project.

3. Bridge Deck Rehabilitation. If the existing bridge deck is structurally deficient, it may be rehabilitated as part of the 3R project. In addition, where the bridge deck is structurally sound but its width is inadequate (i.e., the bridge is functionally deficient), the bridge deck may be rehabilitated solely to widen the bridge deck. Bridge deck widening may then require work to the superstructure and/or substructure.

4. Existing Bridge to Remain In Place. If an existing bridge is structurally sound and if it meets the Department's design loading capacity, it is unlikely to be cost effective to improve the geometrics of the bridge. These are considered existing bridges to remain in place. However, if the geometric deficiencies are severe and/or if there has been an adverse accident experience at the bridge, it may be warranted to rehabilitate the bridge deck to widen the bridge or to make other improvements.

   In some cases, only the bridge substructure (e.g., abutments, piers) and/or foundation (e.g., footings, piles) may require rehabilitative work. For the purpose of applying the 3R geometric design criteria, these may be considered existing bridges to remain in place.

5. Bridge Rails/Transitions. For reconstructed bridges or rehabilitated bridge decks, the existing bridge rails and approaching guardrail-to-bridge-rail transitions must be upgraded to meet current Department criteria. For existing bridges to remain in place within the limits of a 3R project, the Bridge Division will evaluate the adequacy of the existing bridge rail to determine if it should be upgraded. The roadway designer will evaluate the adequacy of the existing approaching bridge rail transition for needed upgrading. See Chapter 9 for more information on guardrail-to-bridge-rail transitions.

Bridge Width

The following will apply to the evaluation and improvement to the width of bridges within the limits of a 3R project:

1. Bridge Replacement/Bridge Reconstruction/Bridge Deck Rehabilitation. For these scopes of work, the criteria in Table 11-3A will apply to the bridge width.

2. Existing Bridges to Remain in Place. The existing width of bridges proposed to remain in place will be evaluated against the criteria in Table 11-3A. If the existing width does not meet these criteria, the designer must either widen the bridge as part of the 3R project or seek a design exception.

   All bridges which are narrower than the approach roadway width (including shoulders) should be evaluated for special narrow bridge treatments. At a minimum, the signing and pavement markings must meet the criteria of the MUTCD.

Horizontal/Vertical Alignment

Except for bridge replacements, it is unlikely to be cost effective to improve the existing horizontal or vertical alignment for a bridge within the limits of a 3R project.

11-3.02.06 Roadside Safety

Reference: Chapter 9

One of the objectives of a 3R freeway project may be to upgrade roadside safety along the freeway. Barrier rail warrants and design can present difficult problems (e.g., guardrail height), as discussed in the following sections.

Cross Drainage Structures

The following will apply:

1. Beyond 30 ft. Cross drainage structures located beyond 30 ft will not require treatment.

2. Within 30 ft. Cross drainage structures within 30 ft will be treated as follows:

   a. If the opening is less than 36 in., replace any headwalls with a flared end section.

   b. If the opening is greater than 36 in. the options are to extend the drainage structure beyond 30 ft, install guardrail or do nothing. If the do-nothing option is selected, a design exception will be required.

Medians

The following will apply:

1. Median Width 64 ft and Wider. Obstacles (e.g., bridge piers, drainage structures) that are located in the center of the median will not require treatment.

2. Median Width Less than 64 ft. Any obstacles within 30 ft of the edge of traveled way should be treated. If the existing median side slopes are 10:1 or flatter, guardrail should be placed as close to the obstacle as practical, subject to the dynamic deflection of the system. If the side slopes are steeper than 10:1, the following options are available:

   a. Regrade the median slope to 10:1 and place guardrail close to the obstacle. This is the more desirable option; however, drainage considerations may render this option impractical.

   b. Place guardrail with the face of the guardrail at the shoulder line.

3. Ditch Plugs. Where the median width is 64 ft or less and median or cross drainage pipes are used in conjunction with ditch plugs, the following will apply:

   a. Reconstruct ditch plugs and inlets to current standards.

   b. For pipe sizes up to 36 in., replace any existing headwalls with a flared end section.

   c. For pipe sizes greater than 36 in., construct a drainage inlet which conforms to the side slope.

4. Intersecting Slope. This refers to slopes that are transverse to the direction of traffic which exist, for example, at an emergency crossover. If necessary, intersecting slopes should be regraded to a 10:1 or flatter slope within the clear zone.

Exit and Entrance Gores

The gore area is normally considered to be both the paved triangular area between the through lane and the exit or entrance lane, plus the graded area which may extend several hundred feet downstream or upstream from the gore nose. The following should be considered when evaluating the safety of the gore:

1. Roadside Obstacles. If practical, the area beyond the exit gore nose should be free of signs and luminaire supports for approximately 300 ft. The designer should ensure that any existing breakaway supports remain functional. In addition, all curbs and exposed sign footings should be removed from both entrance and exit gores.

2. Slopes. The graded area beyond or before the gore nose should be as flat as practical. If the elevation between the exit or entrance ramp and the mainline increases rapidly, this may not be practical. These areas may be non-traversable and the gore design should shield these areas from the motorist. At some sites, the vertical divergence of the ramp and mainline will warrant protection for both roadways beyond or before the gore.

3. Paved Gore. The paved triangular gore area between the through lane and exit or entrance lane should be safely traversable. If practical, this area should have a uniform slope at a rate which is intermediate between the slope of the through lane and the slope of the exit or entrance lane. However, if a drainage inlet is placed within the paved gore, this may not be practical and may, in fact, require two breaks in the cross slope. In this case, the break in pavement cross slope should not exceed 8 percent at any point.

Safety Appurtenances

Section 11-2.08 presents several observations on safety appurtenances on 3R non-freeway projects. These apply equally to 3R freeway projects.

11-3.02.07 Interchanges

Reference: Section 6-10.0

A 3R freeway project may include proposed work on a freeway interchange. The work may be to rehabilitate the entire interchange or to make only selective improvements to the interchange geometrics. For example, the designer should evaluate the geometric design of any acceleration/deceleration lanes within the project limits to determine if any improvements are warranted.

11-3.02.08 Emergency Crossovers

One or more emergency crossovers may be present within the limits of a 3R freeway project. In the absence of any operational or safety problems, the existing locations of these crossovers may be retained.

11-3.02.09 3R Table of Geometric Design Criteria

Table 11-3A presents the Department's criteria for the geometric design of 3R freeway projects.

11-3.02.10 Exceptions to Geometric Design Criteria

The discussion in Section 2-8.0 on design exceptions applies equally to the geometric design of 3R freeway projects. However, it will only apply to the geometric design of the specific freeway improvement. As an example, if the 3R project is primarily a pavement overlay, it will be necessary to meet the Department's roadway width criteria (lane and shoulders) and the travel lane and shoulder cross slope criteria. It will not be necessary to secure a design exception for a horizontal or vertical curve which does not meet the Department's criteria for new construction/reconstruction.

11-4.0 SPOT IMPROVEMENTS (Non-Freeways)

11-4.01 Objectives

Spot improvements are intended to correct an identified deficiency at an isolated location on non-freeways. This project scope of work is consistent with the Department's responsibility to provide a safe driving environment for the motoring public which is free of unexpected demands on the driver. Experience has demonstrated the benefits of improving limited roadway sections or spot locations with recognized geometric deficiencies to at least a level consistent with the adjacent highway sections. This will provide drivers with a facility that is consistent with the principles of driver expectancy.

The deficiency which the spot improvement will correct may be related to structural, geometric, safety, drainage or traffic control problems. These projects are not intended to provide a general upgrading of the highway, as are projects categorized as new construction, reconstruction or 3R. For these reasons, a flexible approach is necessary to determine the appropriate geometric design criteria which will apply to the spot improvement.

Spot improvement projects may also be affected by special criteria which may apply to a particular funding category. Two examples are:

1. Safety Projects. These projects are intended to provide cost-effective improvements to sites identified as having a disproportionate number of accidents or accident rate. Typical projects are intersection improvements, flattening a horizontal curve, installing roadside barriers or installing traffic control devices. Most often, projects will only be funded when the B/C ratio is estimated to be above 1.0. The Traffic Engineering Division is responsible for conducting a preliminary evaluation of the site and recommending improvements. When roadway work is involved, the Roadway Design Division is responsible for preparing the detailed project design.

2. Highway Bridge Replacement & Rehabilitation Program (HBRRP). The HBRRP is intended to correct structural and functional deficiencies on a priority basis. The priorities are determined by a Statewide bridge inspection program which leads to a sufficiency rating for each bridge. The rating is based on a weighted formula which reflects both structural and geometric deficiencies (e.g., inadequate roadway width, poor alignment or inadequate bridge rail or transitions). However, the structural deficiencies usually have the greater influence on the sufficiency rating. Therefore, this should be reflected in the consideration of any geometric improvements. Geometric design criteria for HBRRP projects are discussed separately in Section 11-4.03.

Table 11-3A
GEOMETRIC DESIGN CRITERIA FOR FREEWAYS
(3R Projects)

	DESIGN ELEMENT			Manual Section	Rural		Urban	
Design Controls	Design Forecast Year			11-3.02	Desirable: 20 Years Minimum: 10 Years		Desirable: 20 Years Minimum: 10 Years	
	✻Design Speed			11-3.02	(1)		(1)	
	Control of Access			2-4.0	Full (Type 1)		Full (Type 1)	
	Level of Service			2-3.04	B		Desirable: B Minimum: C	
Cross Section Elements	✻Lane Width			2-6.03	12′		12′	
	Outside Shoulder Width	✻Usable (2a)		2-6.03	12′		12′	
		✻Surfaced (2b)			10′		10′	
	Median Shoulder Width(3)	✻Usable		2-6.03	8′		8′	
		✻Surfaced			4′		4′	
	✻Cross Slope	Travel Lane		2-6.03	2%		2%	
		Shoulder			4%		4%	
	Auxiliary Lanes	Lane Width		2-6.03	12′		12′	
		Shoulder Width (4)			Surfaced: 10′ Usable 12′		Surfaced: 10′ Usable 12′	
	Median Width			2-6.03	Existing		Existing	
	✻New and Reconstructed Bridges	Structural Capacity		2-6.03	HS-20		HS-20	
		Minimum Width			Traveled Way + 12′ (Outside Shoulder) + 6′ (Median Shoulder)		Traveled Way + 12′ (Outside Shoulder) + 6′ (Median Shoulder)	
	✻Existing Bridges to Remain in Place	Structural Capacity		2-6.03	HS-20		HS-20	
		Minimum Width (5)			Traveled Way + 10′ (Outside Shoulder) + 4′ (Inside Shoulder)		Traveled Way + 10′ (Outside Shoulder) + 4′ (Inside Shoulder)	
	Minimum Right-of-Way Width			2-6.03; 10-1.02	(6)		(6)	
	Desirable R/W Border Width (Beyond toe/top of fill/cut slope)				Existing		Existing	
	Roadside Clear Zone	Guardrail		9-4.03; 9-2.0	Usable Shoulder Width		Usable Shoulder Width	
		Obstruction (7)			30′		30′	
	Slope Schedule	Cut	Foreslope (within clear zone)	2-6.03	Existing		Existing	
			Depth of Ditch		Existing		Existing	
			Backslope		Existing		Existing	
		Fill	Height ≤ 10′		Desirable: 6:1 Maximum: 2:1		Desirable: 6:1 Maximum: 2:1	
			Height > 10′ (8)		3:1		3:1	
Alignment Elements	DESIGN SPEED				60 mph	65 mph		70 mph
	✻Minimum Stopping Sight Distance			2-6.02	Existing	Existing		Existing
	✻Maximum Degree of Curvature			3-3.0	Existing	Existing		Existing
	✻Superelevation Rate			3-4.01	Existing (9)			
	✻Horizontal Sight Distance			3-5.0	Existing			
	✻Maximum Grades	Level		4-3.01	Existing	Existing		Existing
		Rolling			Existing	Existing		Existing
	Minimum Grades			4-3.02	Existing			
	✻Vertical Curve (K-values)			4-4.0	Existing	Existing		Existing
	✻Minimum Vertical Clearance (Freeway Under)	New/Reconstructed Bridges (10a)		2-6.03	16.5′			
		Existing Bridges (10b)			16′			
		Sign Truss (10a)			17.5′			
	Vertical Clearance (Freeway Over Railroad)			2-6.03	23′			

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR FREEWAYS
(3R Projects)

Footnotes to Table 11-3A

1. Design Speed. At a minimum, this will be as selected at the time of the original construction or the posted speed limit, whichever is greater.

2. Outside Shoulder Width. The following will apply:

   a. An existing usable shoulder width of 10 ft may be retained.

   b. Where the truck DDHV exceeds 250, consider providing a 12-ft surfaced shoulder. A surfaced shoulder is not required on non-Interstate freeways.

3. Median Shoulder Width. A surfaced shoulder is not required on non-Interstate freeways. For three or more lanes in one direction, the following will apply:

   a. Depressed Median. Provide a 12-ft usable shoulder width of which 10 ft will be surfaced. However, an existing usable shoulder width of 10 ft may be retained. Where the truck volume exceeds 250 DDHV, a 12-ft surfaced shoulder should be considered.

   b. Concrete Median Barrier. Where the two roadways are separated by a CMB, the desirable surfaced median shoulder width is 12 ft; the minimum surfaced median shoulder width is 10 ft. Where the truck traffic exceeds 250 DDHV, the 12-ft width merits stronger consideration.

4. Auxiliary Lane Shoulder Width. An existing usable shoulder width of 10 ft may be retained. A surfaced shoulder is not required on non-Interstate freeways.

5. Bridge Width. For existing bridges to remain in place which are considered major structures (i.e., lengths greater than 200 ft), the minimum clear width is the width of the traveled way plus 3.5 ft each side.

6. Right-of-Way Width. The acquisition of significant amounts of right-of-way is typically outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. However, the designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

7. Roadside Clear Zone. The recommended clear zones are for design speeds of 60 mph or 70 mph, for 6:1 fill slopes or foreslopes and for an ADT > 6000. See Section 9-2.0 for clear zones based on other roadside conditions. All values are measured from the edge of traveled way or auxiliary lane, if present.

8. Fill Slopes (Height > 10 ft). All existing fill slopes steeper than 3:1 will either be flattened or guardrail will be provided.

9. Superelevation Rate. Normally, the existing superelevation rate will be adequate. However, the designer should refer to Table 3-4A to determine if any improvements should be considered.

10. Minimum Vertical Clearances. The following will apply:

    a. Table values include an additional 6-in. allowance for future resurfacing.

    b. The minimum vertical clearance should be 16 ft. However, in highly developed urban areas, where attaining the 16-ft clearance would result in unreasonable costs, the minimum clearance can be 14.5 ft if there is an alternative freeway facility with the minimum 16-ft clearance.

11-4.02 Approach

The Department has adopted a flexible approach to the geometric design of spot improvement projects. The following summarizes the approach:

1. Numerical Criteria. The designer should consider the level of improvement which will most likely be used to upgrade the highway in the future. If this is deemed to be reconstruction, then the criteria in Chapters 2, 3, 4, 6 and 9 will provide the frame of reference for the spot improvement. If a 3R project is considered the most likely level of improvement, then the criteria in Chapter 11 will apply.

2. Design Speed. The design speed of the adjacent sections should be used for the spot improvement; however, a speed less than the posted speed should not be used. The selection of the applicable design speed will be left to the judgment of the designer. Some factors that may be considered include:

   a. the results from a speed study by the Traffic Engineering Division, if conducted;

   b. the design speed for new construction or a 3R project; and

   c. the posted/regulatory speed limit (this will be a minimum).

3. Application. The designer should apply the selected criteria specifically to the geometric improvement related to the objective of the spot improvement project (e.g., install barrier, flatten a horizontal curve, add a left-turn lane). In addition, the designer should evaluate other geometric design deficiencies within the project limits, especially any secondary impacts precipitated by the primary improvement. The designer should consider improving any severe deficiencies, even if not related to the specific objective of the spot improvement. The designer will exercise his/her judgment when determining any other improvements which may be justified.

4. Exceptions. The discussion in Section 2-8.0 on design exceptions applies equally to the geometric design of spot improvement projects. However, it will only apply to the geometric design of the specific geometric design improvement, and it will also apply to any secondary impacts which may result from the improvement.

11-4.03 Geometric Design of Highway Bridge Replacement/Rehabilitation Program (HBRRP) Projects

The spot improvement approach discussed in Section 11-4.02 also applies to HBRRP projects. The following offers additional factors to consider:

1. Horizontal and Vertical Alignment. Many existing bridges have alignments which do not meet the Department's current criteria. For bridge replacement projects, the designer should evaluate the practicality of realining the bridge to meet the applicable alignment criteria (new construction/reconstruction or 3R). For bridge rehabilitation projects, it is unlikely to be cost effective to realine the bridge to correct any alignment deficiencies.

2. Width (Bridge Replacement). The bridge width will be determined by the criteria for new construction/reconstruction based on a 20-year traffic projection.

3. Width (Bridge Reconstruction/Bridge Deck Rehabilitation). The bridge width should equal or exceed the width as determined from the Department's criteria for the most likely level of future highway improvement on the approaches (reconstruction or 3R). This width will be determined by the tables in Chapters 2 or 11. The traffic volume projections will also be based on the Department's criteria for selecting the design year for the scope of work (3R or reconstruction).

4. Narrow Bridges. All bridges which are narrower than the approach roadway width (including shoulders) should be evaluated for special narrow bridge treatments. At a minimum, the signing and pavement markings must meet the

criteria of the MUTCD. In addition, NCHRP 203 *Safety at Narrow Bridge Sites* provides criteria specifically for narrow bridges (e.g., special pavement markings). The designer, in coordination with the Traffic Engineering Division, should evaluate the value of these additional treatments at the bridge site.

5. Bridge Rails. All existing bridge rails and the approach transitions will be evaluated to determine if they meet the Department's current criteria. The Bridge Division will be responsible for assessing the bridge rails and for making any recommendations for improvements. The roadway designer will evaluate the adequacy of the approach guardrail-to-bridge-rail transitions.

Chapter 12

Pavement Design

MISSISSIPPI DESIGN MANUAL

Chapter 12

PAVEMENT DESIGN

Section		Page
12-1.0	GENERAL RESPONSIBILITIES AND PROCEDURES	12-1
12-2.0	PAVEMENT STRUCTURE COMPONENTS	12-1
12-2.01	Pavement	12-2
12-2.02	Base Course	12-2
12-2.03	Subbase Course	12-2
12-2.04	Design Soil	12-2
12-3.0	PAVEMENT STRUCTURE DESIGN	12-3
12-4.0	TYPICAL PAVEMENT STRUCTURE DETAILS	12-3
12-4.01	Layers of Hot Mix Asphalt Pavement	12-3
12-4.02	Stage Construction	12-5
12-4.03	Special Designs	12-5
12-4.04	Edgedrains and Drainable Base	12-5
12-4.05	Heavily Traveled Routes	12-5
12-5.0	INTERSECTIONS AND DRIVEWAY PAVEMENTS	12-5
12-6.0	BRIDGE END PAVEMENT	12-6
12-7.0	ESTIMATING QUANTITIES	12-6

Chapter 12

PAVEMENT DESIGN

This chapter addresses criteria and procedures for designing a pavement structure and for estimating quantities of contract pay items. Pavement design policies specifically applicable to State Aid projects are set forth in Chapter 13.

12-1.0 GENERAL RESPONSIBILITIES AND PROCEDURES

Responsibilities for analyses of materials and design of pavement structures are shared jointly by the District and Roadway Design Division and Materials Division when needed.

The normal procedure for the determination of the pavement structure is:

1. The District conducts an initial soils investigation and prepares an original soil profile which identifies the characteristics of the roadbed material. Based on this data, the District will submit an earthwork plan which identifies areas of unsuitable material (i.e., high-volume change soil, muck, etc.) and the recommended treatment for these areas. The District will also recommend a preliminary pavement structure design so that contract plans may be developed and the project graded to the estimated structure thickness.

   The District Materials Engineer may submit soil samples to the Materials Division for analyses and recommendations of any special design soil treatments.

2. Upon completion of the grading project, the District conducts an additional soils investigation and prepares a final design soil profile. A final pavement structure design recommendation is prepared by the District and is submitted to the Roadway Design Division for review and evaluation.

   The Roadway Design Division will prepare an economic analysis, when necessary, and finalize the design to be shown on the contract plans.

   On projects which include grading and paving, the District will recommend an earthwork plan and final pavement structure based on the original soil profile. After the contract is let and grading is complete, a final soil profile is prepared. If it is discovered that changes in the pavement design are necessary, they are made at this time.

3. The designer is advised of the pavement structure details and is responsible for the development of the roadway typical sections and elevations to accommodate the pavement structure. The designer is also responsible for estimating the pay item quantities.

4. Any rehabilitation of existing pavement must be justified according to the MDOT Pavement Rehabilitation Policy. This Policy takes into account non-destructive test results, pavement condition ratings, core analysis and other engineering factors. This recommendation is prepared by the District and is submitted to the Roadway Design Division for review and evaluation.

12-2.0 PAVEMENT STRUCTURE COMPONENTS

The Department recognizes four basic components of a pavement structure:

1. pavement,
2. base course,
3. subbase course, and
4. design soil.

The various alternatives for each of the basic components are described below.

12-2.01 Pavement

The pavement is the top component of a pavement structure on which vehicles travel. There are three standard types of pavement:

1. bituminous surface treatment,
2. hot mix asphalt, and
3. portland cement concrete pavement.

The use of bituminous surface treatment is limited to the lower traffic volume roads with a projected ADT of 1000 or less. Hot mix asphalt or portland cement concrete pavement may be specified for major highways. The choice usually is based on economics, soil conditions and other governing factors.

12-2.02 Base Course

The component immediately below the pavement is the base course. The four most common and frequently used bases are:

1. granular base course,
2. cement treated granular base course,
3. hot mix asphalt, and
4. lime fly-ash treated granular base course.

The granular base course ordinarily is limited to the lower traffic volume roads with bituminous surface treatment. A hot mix asphalt base usually is used under hot mix asphalt pavement and portland cement concrete pavement. Cement treated granular base courses are used primarily under concrete pavement or bituminous surface treatment. The lime fly-ash treated granular base course may be specified under all three of the pavement types.

12-2.03 Subbase Course

The subbase course is beneath the base. It is usually constructed of a more economical material of lesser quality than the base course. The typical application is a minimum 6-in. granular subbase course.

12-2.04 Design Soil

"Design soil" is defined as that portion of the roadbed consisting of the 3 ft of soil immediately below the subgrade. It supports the subbase course. This may be undisturbed natural earth strata in areas of excavation or embankment constructed with unclassified excavation or borrow excavation.

Design soil treatment is considered in the design of pavement structures. Although the treatment is not considered a part of the total pavement structure, its allowable credit may eliminate or reduce the thickness of the pavement structure. Control of the design soil must be considered on all projects which involve grading.

It is advantageous for this course to be fully controlled. To accomplish this, the District Materials Engineer shall recommend the class borrow to be used for the construction of the design soil prism in both cut and fill sections. This method will require removal of the top 3 ft below subgrade if material in the cut sections does not

meet the desired specifications. An acceptable reason for not controlling the design soil is anticipated excessive costs.

If high-volume change soil (i.e., VC > 60%) exists in the design soil prism, one of three methods in SOP #TMD-20-14-00-000M must be chosen to treat the material. All three methods require cut slopes to be 6:1 and fill slopes to be 5:1. The three methods are as follows:

1. replacement method (control of design soil),
2. moisture barrier method, and
3. lime treatment method (not applicable in Yazoo clay formations).

See SOP #TMD-20-14-00-000M for details.

12-3.0 PAVEMENT STRUCTURE DESIGN

The total pavement structure is comprised of various thicknesses of the structure components. The principal factors that influence the thickness of the pavement structure are traffic volumes, proportionate number of trucks, California Bearing Ratio (i.e., CBR) and soil characteristics of the roadbed.

The method of determining the thickness of each component is described in "Pavement Design Systems for Mississippi Highways," Final Report, State Study No. 66, September 1983; and AASHTO *Guide for Design of Pavement Structures.*

12-4.0 TYPICAL PAVEMENT STRUCTURE DETAILS

The design details of the pavement structure courses must be clearly identified in the contract plans. Typical sections should be prepared showing the thickness, geometric shapes and dimensions to be constructed. A detail of each type of pavement is illustrated in Figure 12-4A. Example typical sections are provided in the model contract plans in Chapter 15.

12-4.01 Layers of Hot Mix Asphalt Pavement

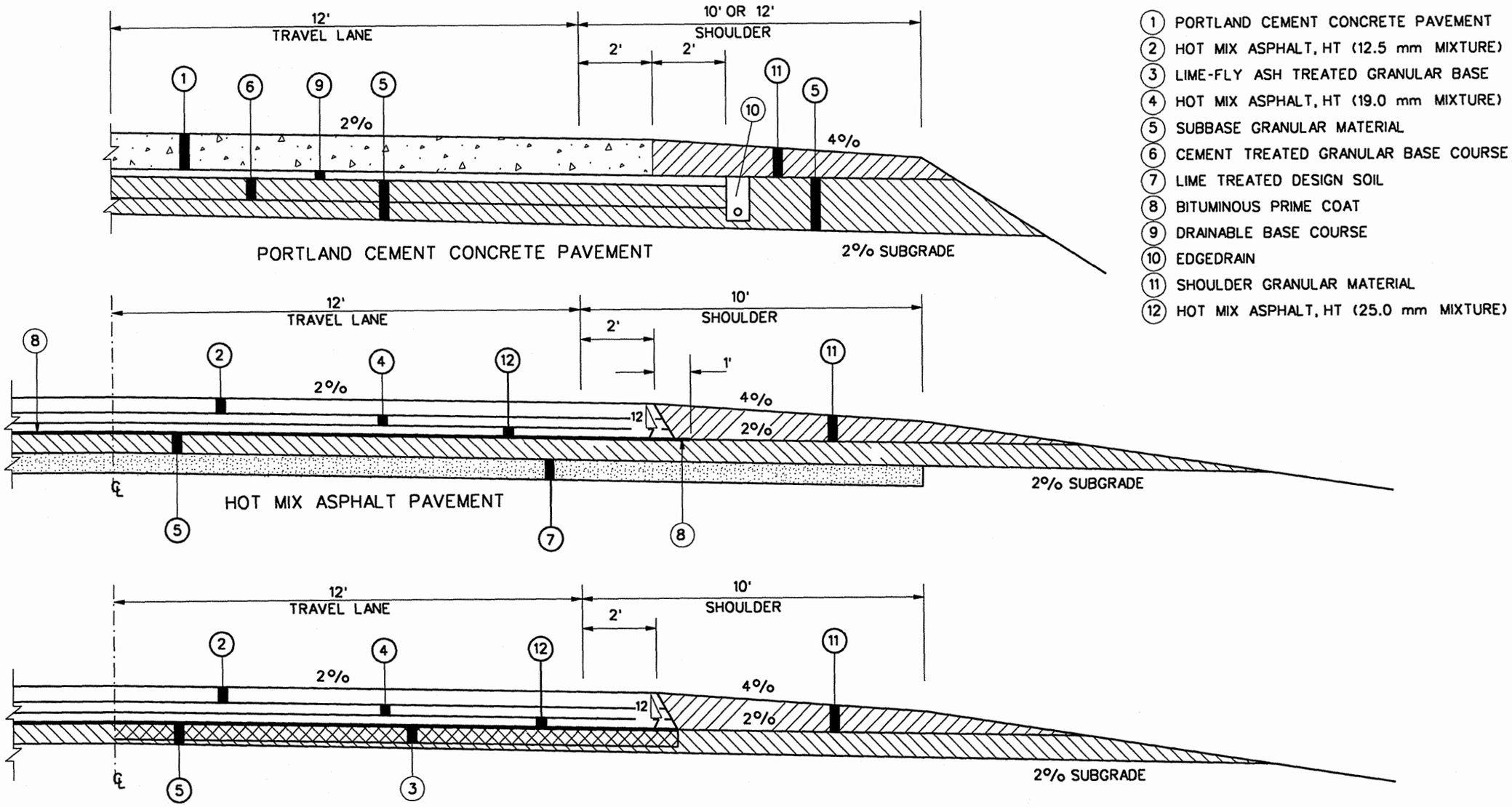
Hot mix asphalt is normally designed and constructed using multiple lifts of asphalt. Each lift is defined by one of the four maximum nominal size aggregates (9.5 mm, 12.5 mm, 19.0 mm or 25.0 mm) and one of the four mix designs (ST, MT, HT or Polymer Modified HT). Each layer is identified separately on the typical sections because each layer is measured and paid for separately.

Generally only one mix design will be used in each pavement design. Exceptions will occur in mix designs to be used on heavily traveled truck routes. These pavement designs will include both HT and polymer modified HT mix designs, as mentioned in Section 12-4.05.

Acceptable lift thicknesses are shown in Table 12-4A.

Table 12-4A
ACCEPTABLE LIFT THICKNESSES

Maximum Nominal Aggregate Size (mm)	Minimum Lift Thickness (in)	Maximum Lift Thickness (in)
9.5	1	1½
12.5	1½	2
19.0	2¼	3
25.0	3	4



TYPICAL PAVEMENT DETAILS
Figure 12-4A

12-4.02 Stage Construction

Under certain conditions, some form of stage construction may be specified for paving a project. These may include:

1. a grading and drainage project (i.e., with all paving planned as a future project); or

2. a paving project which includes the construction of a pavement structure (i.e., subbase, base and pavement) with grading completed on a previous project.

In these cases, the typical section should clearly illustrate details of the work to be included in the current contract. Any future pavement should be shown with a dotted line to identify the thickness of the ultimate pavement structure. Detailed descriptions of the future course need not be provided.

12-4.03 Special Designs

In addition to providing typical sections of the mainline, separate typical sections may be needed for paving of supplemental roadways such as:

1. interchange ramps,
2. frontage roads,
3. intersecting roads,
4. driveways,
5. detours,
6. crossovers,
7. bridge end pavement, and
8. extra width at guardrail.

Because soil conditions may vary, the details of the pavement structure may not be consistent throughout the project length. Thicknesses may vary and special treatment of unstable material may be specified for some sections. Each different situation must be represented by a typical section which clearly indicates the design details and applicable limits by stationing.

12-4.04 Edgedrains and Drainable Base (Concrete Pavement)

All concrete pavement sections must include a drainable base layer and edgedrains similar to the typical section shown in Figure 12-4A. The purpose of this system is to eliminate moisture within the pavement structure.

All new asphalt pavements on four-lane routes will also include a drainage base layer and edge drains.

12-4.05 Heavily Traveled Routes

Heavily traveled truck routes which will be constructed with asphalt pavement will require "Polymer Modified High Type Asphalt" (i.e., special rut resistant mix design). These routes are designated on a map titled "Routes Designated for High Type Asphaltic Mixtures." This map is published by the Planning Division and is based on the number of 80-kN ESAL's in the 10-year design life.

12-5.0 INTERSECTIONS AND DRIVEWAY PAVEMENTS

The paving of intersections must be identified on the plans and quantities included in the estimate. Details of the pavement structure should be illustrated on typical sections and identified with locations as needed. The following general criteria are applicable:

1. Public Road Intersections. At intersections with paved public roads, a new pavement may extend through the limits of reconstruction. Unpaved public roads will normally be paved within the intersection limits.

2. Driveways. A driveway is an access constructed within the right-of-way connecting the public roadway with adjacent property.

Existing paved driveways are reconstructed and paved as required to provide a smooth connection. Driveways not currently paved should be paved to the curb returns or to the highway shoulder line in open shoulder sections. In either case, a layer of granular material and/or aggregate will be provided on the remainder of the driveway.

12-6.0 BRIDGE END PAVEMENT

Because of potential differential settlement between a bridge deck and adjacent roadway pavement, a specially designed reinforced concrete slab shall be provided to assure a continuing smooth transition. The bridge end pavement will be the same width as the paving bracket on the bridge. Other design details are included in the *Roadway Design Standard Drawings*. Figure 12-6A illustrates the bridge end pavement design.

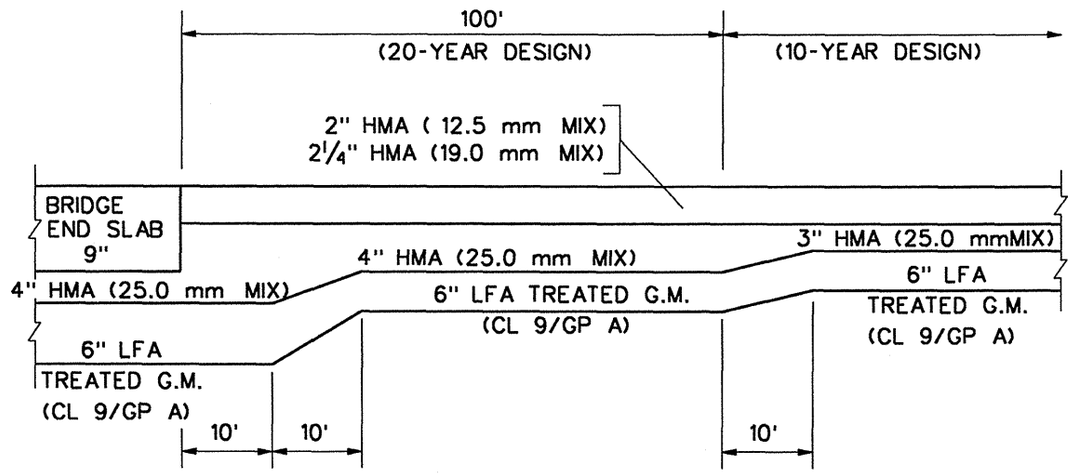
12-7.0 ESTIMATING QUANTITIES

The designer should compute estimated quantities of required materials in terms established for contract pay items. Computation worksheets should be documented carefully and placed in the project file. Estimated quantities, including pay item numbers, are recorded in the Summary of Quantities in the contract plans.

Generally, quantity estimates should be made in accordance with the following criteria:

1. Concrete Pavement. Concrete pavement is measured in square yards of surface area. Estimates should be made separately for each type of pavement (e.g., reinforced cement concrete, plain cement concrete, continuously reinforced concrete) and each thickness.

   Costs for transverse contraction joints and the sawing and sealing of joints are normally absorbed in the cost of concrete pavement. Expansion joints (i.e., with or without dowels) are paid for under the appropriate pay item in feet.



TYPICAL BRIDGE END PAVEMENT DESIGN

Figure 12-6A

2. Hot Mix Asphalt Pavement. Hot mix asphalt pavement is estimated in tons of material. A unit weight of 110 lbs/yd^2/in. of pavement thickness should be used for estimating purposes.

   Separate estimates are made for each of the identified layers of hot mix asphalt pavement.

3. Bituminous Surface Treatment. A bituminous surface treatment consists of asphaltic cement and one or more layers of aggregate cover material. A single bituminous surface treatment (SBST) consists of an application of asphaltic cement and layer of aggregate cover material (i.e., coarse or seal). A double bituminous surface treatment (DBST) consists of an application of asphaltic cement and a layer of coarse aggregate cover material, and then a second application of asphaltic cement and a layer of seal aggregate cover material.

   Coarse aggregate cover material may be slag, stone or gravel (i.e., size nos. 5, 6 or 56). Seal aggregate cover material may be slag, stone or crushed gravel (i.e., size nos. 7, 8 or 89) or expanded clay (i.e., size no. 8).

   The type of aggregate cover material and gradation size are specified as a portion of the pay item nomenclature. Normally, coarse aggregate cover material will be specified as slag, stone or gravel (i.e., size nos. 5, 6 or 56) and seal aggregate cover material will be specified as slag, stone or crushed gravel (i.e., size no. 7, 8 or 89).

   For estimating purposes, the rates of application are:

Coarse Aggregate:	0.52 ft^3/yd^2
Seal Aggregate:	0.28 ft^3/yd^2
Asphaltic Cement (SBST):	0.28 gal/yd^2
Asphaltic Cement (DBST):	0.68 gal/yd^2

4. Prime Coat. A bituminous prime coat will not be shown on the typical sections if the granular material is chemically treated (i.e., lime fly-ash or cement). A bituminous prime coat should be specified for treatment of the compacted granular base course which will precede construction of a hot mix asphalt base, hot mix asphalt pavement or bituminous surface treatment. The prime coat is expressed in gallons at a rate of application of 0.35 gal/yd^2. Note that the prime coat is applied to a width 2 ft greater than the pavement width (i.e., 1 ft on each side). See Figure 12-4A for an illustration.

5. Cement Treated Subbase or Base Course. Three contract pay items should be estimated for cement treated courses:

   a. cubic yards (yd^3) of granular material,

   b. hundred-weight (CWT) of portland cement, and

   c. square yards (yd^2) of soil-cement-water mixing (i.e., for each depth course).

   The rate of application for the cement will vary depending upon the soils analysis for the individual project. The designer will be advised of the appropriate rate of cement application in terms of dry-weight granular material.

   Soil-cement-water mixing is measured by the square yard (yd^2) , complete-in-place for each required course. Therefore, only one mixing operation is estimated regardless of the number of applications of material and the number of mixing operations.

   The CWT of portland cement is estimated by the following formula:

$$PC = (L)(W)(D)(1/27)(R)(0.01)(CR)(20) = 0.0074\ (L)(W)(D)(R)(CR) \quad \text{(Equation 12-7.1)}$$

$$WM = (L)(W)(1/9) \quad \text{(Equation 12-7.2)}$$

Where:

PC	=	Portland cement in CWT
WM	=	Soil cement - water mixing in square yards (yd^2)
L	=	Length in feet (ft)
W	=	Width of treatment in feet (ft)
D	=	Depth of treatment in feet (ft)
R	=	Percent of cement application (for 3%, R=3)
CR	=	Conversion rate of granular material in tons per cubic feet (ton/ft^3)
20	=	Factor to convert tons to CWT
1/27	=	Factor to convert cubic feet (ft^3) to cubic yards (yd^3)

7. Granular Subbase or Base Course. Granular material for subbase or base is measured and estimated by the cubic yard (yd^3) or ton. Each different class or group designation should be identified and estimated separately.

When measured by the ton (t), the mass shall be determined by dry-mass at 100% proctor density with an estimated unit dry-mass for each class as follows:

Class	lbs/ft^3	$tons/yd^3$
1 and 2	131	1.7685
3 and 4	129	1.7415
5 and 6	127	1.7145
Crushed Stone	125	1.6875
Soil	100	1.3500
Other Classes	114	1.5392

Note that the following shrinkage factors are used for granular material:

Pay Item Description	Shrinkage Factor
LVM (yd^3)	50% (multiply by 1.5)
AEA (yd^3)	0% (no shrinkage)
Ton (ton)	20% (multiply by 1.2)

8. Lime Treated Design Soil Course. Two contract pay items should be estimated for lime treated design soil:

a. tons of hydrated lime, and

b. square yards (yd^2) of soil-lime-water mixing (i.e., for each depth and class).

The rate of application for the lime will vary depending on the soils analysis for individual projects or even sections of highway within a project. The designer will be advised of station limits and width for treatment and the appropriate class and rate of lime application in terms of percent of dry-weight.

Soil-lime-water mixing is measured by the square yard (yd^2), complete-in-place, for each required course. Therefore, only one mixing operation is estimated regardless of the number of applications of materials and the number of mixing operations.

The estimated tons of hydrated lime are:

$$HL = (0.01)(L)(W)(D)(R)(100)(1/2000) = 0.0005\,(L)(W)(D)(R) \quad \text{(Equation 12-7.3)}$$

$$WM = (L)(W)(1/9) \quad \text{(Equation 12-7.4)}$$

Where:

HL	=	Hydrated lime in tons
WM	=	Soil lime water mixing in square yards (yd^2)
L	=	Length in feet (ft)
W	=	Width of treatment in feet (ft)
D	=	Depth in feet (ft)
R	=	Percent (%)
0.01	=	Factor to convert to decimal (%)
100	=	Estimated weight of soil in $pounds/ft^3$
1/2000	=	Factor to convert pounds to tons

9. Lime-Fly Ash Treated Subbase or Design Soil Course. Four contract pay items should be estimated for lime-fly ash treated subbase or design soil:

a. tons of hydrated lime (HL);

b. tons of fly ash (FA);

c. square yards (yd^2) of processing lime and fly-ash treated course (i.e., for each depth and class); and

d. cubic yards (yd^3) of granular material.

The rate of application for the lime and fly ash will vary depending on the soils analysis for individual projects or even sections of highway within a project. The designer will be advised of station limits for treatment and the appropriate class and rate of lime application in terms of percentage of dry-weight granular material and the appropriate rate of fly ash application in terms of percentage of dry-weight granular material.

Processing lime and fly-ash treated course is measured by the square yard (yd^2), complete-in-place, for each required course. Therefore, only one mixing operation is estimated regardless of the number of applications of material and the number of mixing operations.

The estimated tons of hydrated lime and fly-ash and square feet (ft^2) of soil lime fly-ash water mixing is as follows:

$$HL = 0.01\ (L)(W)(D)(CR)(LR)(1/27) \quad \text{(Equation 12-7.5)}$$
$$FA = 0.01\ (L)(W)(D)(CR)(FR)(1/27) \quad \text{(Equation 12-7.6)}$$
$$LFAM = (L)(W)(1/9) \quad \text{(Equation 12-7.7)}$$

Where:

HL	=	Hydrated lime in tons
FA	=	Fly-ash in tons
LFAM	=	Processing lime and fly-ash treated course in square yards (yd^2)
L	=	Length in feet (ft)
W	=	Width in feet (ft) (to include any extra width)
D	=	Depth in feet (ft)
CR	=	Conversion rate of granular material in tons per cubic yard ($tons/yd^3$)
LR	=	Lime rate (%)
FR	=	Fly-ash rate (%)
1/27	=	Factor to convert cubic feet (ft^3) to cubic yards (yd^3)
1/9	=	Factor to convert square feet (ft^2) to square yards (yd^2)

Chapter 13

Geometric Design of State-aid Projects

MISSISSIPPI DESIGN MANUAL

Chapter 13

GEOMETRIC DESIGN OF STATE-AID PROJECTS

Section		Page
13-1.0	GENERAL	13-1
13-1.01	Legislative Background	13-1
13-1.02	Responsibilities of Office of State-Aid Road Construction	13-1
13-1.03	County Responsibilities	13-2
13-1.04	Sources of Information	13-2
13-1.04.01	Design Guides	13-2
13-1.04.02	Standard Specifications and Drawings	13-2
13-1.04.03	Technical Assistance	13-3
13-2.0	BASIC DESIGN CONTROLS	13-3
13-2.01	Highway Systems	13-3
13-2.01.01	Functional Classification System	13-3
13-2.01.02	Federal-Aid System	13-3
13-2.02	Project Scope of Work	13-3
13-3.0	GEOMETRIC DESIGN CRITERIA (New Construction/ Reconstruction)	13-3
13-3.01	Design Traffic Volumes	13-3
13-3.02	Design Speed	13-4
13-3.03	Stopping and Passing Sight Distance	13-4
13-3.04	Horizontal Alignment	13-4
13-3.04.01	Curve Types	13-4
13-3.04.02	Traveled Way Widening	13-4
13-3.04.03	Superelevation Rate and Maximum Degree of Curvature	13-5
13-3.04.04	Superelevation Transition Length	13-5
13-3.04.05	Superelevation Axis of Rotation	13-5
13-3.05	Vertical Alignment	13-5
13-3.06	Cross Section Elements	13-5
13-3.06.01	Lane Widths	13-5
13-3.06.02	Shoulder Width and Type	13-6
13-3.06.03	Right-of-Way Width	13-6
13-3.06.04	Bridges	13-6
13-3.06.05	Roadside Clear Zone	13-6
13-3.06.06	Vertical Clearances	13-7
13-3.06.07	Surface and Shoulder Cross Slopes	13-7
13-3.06.08	Pavement Skid Resistance	13-7
13-3.06.09	Side Slopes	13-7
13-3.07	At-Grade Intersections	13-8
13-3.08	Traffic Control Devices	13-8
13-3.09	Erosion Control	13-8
13-3.10	Roadside Safety	13-8
13-3.11	Geometric Design Criteria	13-8
13-3.12	Exceptions to Geometric Design Criteria	13-9
13-4.0	GEOMETRIC DESIGN CRITERIA (3R Projects)	13-17
13-4.01	3R Project Evaluation	13-17
13-4.02	Design Controls	13-17
13-4.03	Cross Section Elements	13-17
13-4.03.01	General	13-17
13-4.03.02	Bridges	13-18
13-4.04	Horizontal Alignment	13-19
13-4.05	Vertical Alignment	13-20
13-4.06	At-Grade Intersections	13-20
13-4.07	Roadside Safety	13-20
13-4.08	3R Table of Geometric Design Criteria	13-20
13-4.09	Exceptions to Geometric Design Criteria	13-20
13-5.0	PAVEMENT DESIGN	13-23
13-5.01	General Criteria	13-23
13-5.02	Design Requirements	13-23
13-5.03	Design Procedure	13-24
13-5.03.01	Traffic Counts	13-24
13-5.03.02	Analysis of Traffic	13-24
13-5.03.03	Analysis of Subgrade Material	13-24
13-5.03.04	Design of Pavement Structures	13-24

Chapter 13

GEOMETRIC DESIGN OF STATE-AID PROJECTS

13-1.0 GENERAL

This Chapter will provide the geometric design criteria for projects administered by the Office of State-Aid Road Construction. In many cases, the design criteria and procedures for projects on State highways are equally applicable to the design of State-Aid projects. Where appropriate, the material presented in other chapters will be referenced.

13-1.01 Legislative Background

The Mississippi Legislature established the State-Aid Division within the Mississippi Highway Department in 1949 to administer the State-Aid road program. The legislation established a State-Aid system consisting of rural Federal-aid secondary routes (based on the old Federal-aid system) and off-system rural routes which are not administered by the State Highway Department.

The roads selected were those main collector and distributor routes serving local trade areas and connecting with the State highway network. The selection criteria placed a special emphasis on those roads essential to the conservation and development of natural resources and for economic and social value. The criteria were also intended to encourage desirable land use. The intent is that the program serve the agricultural, business, educational and industrial interests of the counties.

The State-Aid funds are eligible for constructing, improving, widening, straightening, surfacing or reconstructing roads on the State-Aid system. The funds are not eligible for routine maintenance activities.

In 1992 the Mississippi State Highway Department became the Mississippi Department Of Transportation (MDOT), and the State-Aid Division became the Office Of State-Aid Road Construction.

The 1994 Regular Session of the Legislature passed House Bill 1302 establishing a Local System Bridge Replacement and Rehabilitation Program (LSBP). The State-Aid Engineer administers the Program, which assists counties and municipalities in the replacement and rehabilitation of certain bridges located on local road systems in the counties and within incorporated municipalities within the counties. All subsequent and applicable references to County Engineer shall be construed to also mean LSBP Engineer.

Section 65-9 of Chapter 9 of the Mississippi Code provides additional information on the legislative authorization of the State-Aid program.

13-1.02 Responsibilities of Office Of State-Aid Road Construction

The Office of State-Aid Road Construction is directed by the State-Aid Engineer. It is the responsibility of the State-Aid Engineer to:

1. prepare and adopt uniform design criteria and specifications for construction and maintenance of State-Aid projects;

2. approve revisions to the State-Aid system;

3. approve all construction programs;

4. approve plans and specifications for construction projects;

5. authorize advertisement for bids;

6. concur in the awards of contracts;

7. approve and disburse all payments made from the project fund;

8. approve final acceptance of completed projects; and

9. make annual and other periodic inspections of completed projects to ensure that they are properly maintained by the respective County Board of Supervisors.

The Office sets design criteria and approves programs, contracts and expenditures by the counties on projects that involve State revenues. In addition, the Office is responsible for distributing Federal-aid funds eligible for projects on county roads and for monitoring the design of those projects. These activities are coordinated with the FHWA through the Mississippi Department of Transportation.

13-1.03 County Responsibilities

Eighty-two Mississippi counties are responsible for roads on the State-Aid system. Each County Board of Supervisors is required to appoint a County Engineer to act on behalf of the Board to administer the engineering functions at the county level, subject to the rules and regulations promulgated by the State-Aid Engineer. It is the responsibility of the Board of Supervisors to:

1. designate the county roads to be included in the State-Aid system;
2. prepare and adopt annual construction programs;
3. acquire rights-of-way for State-Aid projects;
4. advertise for bids;
5. award contracts;
6. ensure adequate maintenance of completed projects; and
7. provide for adjustment of utilities as may be required.

The County Engineers are responsible for making surveys, preparing plans, specifications and contract proposal documents, attending lettings, supervising and inspecting construction, and submitting Progress and Final Estimates. Counties may obtain the services of consulting engineers for any of these responsibilities but only as County Engineers and not Consulting Engineers.

13-1.04 Sources of Information

13-1.04.01 Design Guides

Design criteria for State-Aid projects adhere closely to the policies established by AASHTO. The AASHTO publications *A Policy on Geometric Design of Highways and Streets* and the *Roadside Design Guide* provide the basis for geometric design and roadside safety criteria. These are supplemented with policies and criteria established by the Office of State-Aid Road Construction. The design of traffic control devices will be according to the current edition of the *Manual on Uniform Traffic Control Devices.*

13-1.04.02 Standard Specifications and Drawings

The Office of State-Aid Road Construction has published a book of *Standard Specifications for State-Aid Road and Bridge Construction.* This publication and appropriate supplemental specifications and special provisions are furnished to County Engineers.

Standard drawings have been developed by the Office which address specific details of bridges, guardrail, superelevation and traffic control devices (e.g., striping, signing). These drawings are available to the County Engineers and shall be incorporated in the contract plans when applicable.

13-1.04.03 Technical Assistance

The County Engineers may obtain technical guidance and assistance from the Office of State-Aid Road Construction and from the various Divisions of the Mississippi Department of Transportation as needed in specialized cases. The Office will review all designs and plans submitted by the counties for adequacy and compliance with the applicable design criteria.

13-2.0 BASIC DESIGN CONTROLS

13-2.01 Highway Systems

13-2.01.01 Functional Classification System

Geometric design criteria for State-Aid projects are based on the functional classification system. Section 2-1.01 describes the concept and presents brief descriptions of each functional classification. The majority of roads on the State-Aid system are rural collectors, although some of the roads are classified as rural local. These designations lead directly to the tables of geometric design criteria for State-Aid projects in Section 13-3.0.

13-2.01.02 Federal-Aid System

The Department, together with the county governments and FHWA, have designated routes within the State that are eligible for Federal-aid highway funds. These routes are all major rural collectors. Approximately two-thirds are administrated by the Office of State-Aid Road Construction and one-third by the Mississippi Department of Transportation. In addition, the State-Aid Road System also includes those off-system routes which are not administered by the Mississippi Department of Transportation. The Federal-aid system is discussed in Section 2-1.02.

13-2.02 Project Scope of Work

The Office of State-Aid's District Engineer and the County Engineer will determine the scope of work for a proposed State-Aid project. The State-Aid Engineer will approve or disapprove the recommendation. The definitions presented in Section 2-5.0 also apply to State-Aid projects.

13-3.0 GEOMETRIC DESIGN CRITERIA (New Construction/Reconstruction)

Guidelines presented in the following sections, combined with referenced material from other chapters, should be used in the design of State-Aid roads. These criteria apply to both collector and local roads unless otherwise noted.

13-3.01 Design Traffic Volumes

For new construction and reconstruction, State-Aid projects should be designed to accommodate traffic volumes that will occur within the projected life of the facility. However, if the DHV is less than 100, the current ADT may be used for design. The Department's Planning Division maintains current records of traffic data and develops projections of expected future traffic data. These include the design hourly volume (DHV) and average annual daily traffic (AADT).

State-Aid projects are usually on roads with low traffic volumes and, therefore, do not typically require a highway capacity analysis. Where necessary, the designer should use the procedures in the *Highway Capacity Manual* to ensure that the road and its intersections will operate at the design level of service (usually LOS "B") for the DHV. The designer will most often use Chapter 8 "Two-Lane Highways" and Chapter 10 "Unsignalized Intersections" from the *Highway Capacity Manual.*

13-3.02 Design Speed

The design speed for State-Aid projects should be selected based on the topography and the functional classification. In general, low design speeds are applicable to highways with winding alignment in rolling terrain or where environmental conditions dictate. High design speeds are applicable to highways in level terrain and where environmental conditions are favorable. In addition, on all projects the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility.

Design speed is discussed in detail in Section 2-2.01. Design speeds for State-Aid projects and various terrain conditions are provided in Tables 13-3E, F and G.

13-3.03 Stopping and Passing Sight Distance

Table 2-7A presents stopping sight distance values and passing sight distance values for various design speeds. Stopping sight distance is measured from the driver's eye, which is 3.5 ft above the pavement, to a 6-in. height of object. Passing sight distance is measured from a 3.5-ft height of eye to a 4.25-ft height of object. Sight distance is discussed in Section 2-6.02.

13-3.04 Horizontal Alignment

The criteria and procedures presented in Chapter 3 "Horizontal Alignment" will apply in general to State-Aid projects. However, the following sections offer additional guidance which applies specifically to State-Aid projects.

13-3.04.01 Curve Types

Deflection changes in alignment on State-Aid projects will almost always be accomplished by using a simple curve. Spiral or compound curves are rarely used. Simple curves are discussed in Section 3-3.01.

13-3.04.02 Traveled Way Widening

Traveled way widening will be provided on State-Aid projects according to Table 13-3A.

Table 13-3A

DESIGN VALUES FOR TRAVELED WAY WIDENING ON STATE-AID PROJECTS (Feet)

Degree of Curvature	20-ft Traveled Way	22-ft Traveled Way	24-ft Traveled Way
$D < 4°$	0	0	0
$4° \leq D < 8°$	2 ft	1 ft	0
$D \geq 8°$	3 ft	2 ft	0

Note: Extra width to be added on the inside of curve. The specified extra width should be added at a uniform rate throughout superelevation runoff. Centerline stripe should equally divide widened traveled way.

13-3.04.03 Superelevation Rate and Maximum Degree of Curvature

The Department has adopted e_{max} = 10% for the design of State-Aid projects. Table 13-3B provides the maximum degree of curvature for e = 10.0%. Where practical, the designer should provide flatter horizontal curves. The minimum length of a horizontal curve for new construction shall be 200 ft plus the superelevation runoff length.

Table 13-3B
MAXIMUM DEGREE OF CURVATURE

Design Speed (mph)	Maximum Degree of Curvature (ft)
30	24°45′
40	13°15′
50	8°15′
55	6°30′
60	5°15′

Note: e_{max} = 10%

13-3.04.04 Superelevation Transition Length

At a minimum, the total superelevation transition length shall be accomplished according to State-Aid Standard SA-SE-1.

13-3.04.05 Superelevation Axis of Rotation

The superelevation axis of rotation on State-Aid projects should normally follow the centerline of the roadway. The superelevation axis of rotation is illustrated on State-Aid Standard SA-SE-1.

13-3.05 Vertical Alignment

The criteria and procedures presented in Chapter 4 "Vertical Alignment" also apply to State-Aid projects. The following presents a few observations on the application to State-Aid projects:

1. Maximum Grades. Tables 13-3E, F and G present the maximum grades for State-Aid projects.

2. Minimum Grades. Section 4-3.02 states that the desirable minimum longitudinal gradient on uncurbed pavements should be 0.4 percent. On State-Aid projects the designer should make every reasonable effort to meet this minimum gradient.

3. Minimum Length of Vertical Curve. On State-Aid projects, this should be 200 ft unless the curve is located at a stop condition, at a temporary grade tie or where other special conditions exist. In no case will the length be less than 100 ft.

13-3.06 Cross Section Elements

Figure 13-3A illustrates a typical highway cross section on a State-Aid project. The design values for the cross section elements are provided in Tables 13-3E, F and G. Section 2-6.03 presents a discussion on each cross section element, and these apply in general to State-Aid projects. The following additional guidance is provided specifically for State-Aid projects.

13-3.06.01 Lane Widths

Surfaced lane widths vary according to functional classification and traffic volumes. These are presented in Tables 13-3E, F and G.

13-3.06.02 Shoulder Width and Type

Shoulders should be well graded; surfaced shoulders are rarely provided on State-Aid projects. The widths in Tables 13-3E, F and G refer to the usable shoulder width.

13-3.06.03 Right-of-Way Width

According to Mississippi State law, the right-of-way for State-Aid projects must be provided by the County Board of Supervisors at no cost to the project fund. The right-of-way width should accommodate the construction, adequate drainage and proper maintenance of the highway facility. Desirably, this will be without easements. The minimum right-of-way width should not be less than the required cross section elements for design, including the side slopes and/or roadside clear zones. In unusual circumstances, where back slopes extend beyond the normal right-of-way width, temporary construction easements may be obtained.

13-3.06.04 Bridges

Tables 13-3E and 13-3F provide the criteria for the minimum width of new and reconstructed bridges on all State-Aid projects, except LSBP projects. The structural design on these bridges will be according to the latest edition of the AASHTO publication *Standard Specifications for Highway Bridges*. The minimum design loading structural capacity for new and reconstructed bridges will be HS-20. The minimum design loading structural capacity for existing bridges to remain in place (including reconstructed bridges) will be HS-15.

For a reconstruction project, there may be one or several existing bridges within the project limits. Tables 13-3E and 13-3F also provide the roadway width and structural loading capacity to evaluate the adequacy of the existing bridge(s). If the bridge meets the design loading structural capacity, it is unlikely to be cost effective to improve the geometrics of the bridge. However, the geometric deficiencies may be severe, and/or there may be an adverse accident experience at the bridge. Therefore, the designer should consider widening the bridge to meet new construction/reconstruction criteria as part of the reconstruction project.

If the existing bridge width is less than the width in Tables 13-3E and 13-3F and if it will remain in place, it will be necessary to obtain a design exception. In all cases, the signing and pavement markings for the bridge must meet the criteria in the MUTCD. In addition, all existing bridge rails and the approach transitions must be evaluated to determine if any improvements are necessary. The designer should consult the State-Aid's Bridge Division for assistance.

An existing bridge on a State-Aid road may be reconstructed as a spot improvement project. The criteria in Section 11-4.0 will apply to the geometric design of the bridge.

The bridge width criteria for LSBP projects is provided in Table 13-3G.

13-3.06.05 Roadside Clear Zone

Adequate clearance between the edge of traveled way and roadside obstructions is an important safety factor. Table 13-3C presents the minimum required clear zone for State-Aid projects. Where practical, more clearance should be available. The roadside clear zone should be free of all unyielding objects such as trees, non-breakaway sign supports or poles, and any other fixed object that may severely damage an out-of-control vehicle. Utilities may be placed near the outer limits of the right-of-way according to S.O.P. No. SA II-2-8. Clear zones are discussed in detail in Section 9-2.0.

Table 13-3C
RECOMMENDED CLEAR ZONES

Design Speed	Clear Zone
$V \leq 45$ mph	10 ft
$V > 45$ mph	See Section 9-2.0

13-3.06.06 Vertical Clearances

The minimum vertical clearance at underpasses for new and reconstructed bridges on State-Aid projects will be 14 ft over the entire roadway, with an additional 6 in. allowed for future resurfacing; i.e., the total vertical clearance is 14 ft-6in. Where existing bridges will remain in place, the minimum vertical clearance is 14 ft. These clearances apply to the rural collector or local road passing underneath. For structures over railroads, the minimum vertical clearance between the top of rail and bottom of the highway structure is 23 ft.

13-3.06.07 Surface and Shoulder Cross Slopes

The pavement surface should be sloped sufficiently to ensure proper drainage and not affect vehicular operation. Recommended pavement cross slopes for State-Aid projects are presented in Table 13-3D. Cross slopes for the double bituminous surface treatment (DBST), which is considered an intermediate type pavement, should be 2%. All shoulder cross slopes should be 4.0%.

13-3.06.08 Pavement Skid Resistance

All hot plant mixed bituminous pavements for surface courses on Federally funded projects will be required to generate a friction value of at least 0.35 to secure Federal participation in the project. This requirement is only for those roads which have a posted speed limit of 45 mph or greater.

13-3.06.09 Side Slopes

The maximum rate of side slope depends on the stability of the soil as determined by investigations presented in the soil profiles. Slopes should be as flat as practical to help provide adequate vehicular recovery areas. Side slopes on State-Aid projects should be no steeper than 3:1. Slopes steeper than 3:1 with fill heights greater than 10 ft may require guardrail. Tables 13-3E, F and G provide the side slope criteria for State-Aid projects.

Rounding of the cross section should be provided at ditch bottoms and tops of backslopes. All side slopes should be vegetated. Chapter 8 discusses erosion control and ditch treatments.

Table 13-3D

RECOMMENDED PAVEMENT CROSS SLOPES
(for State-Aid Projects)

Surface Type	Range of Cross Slope (percent)
High	2%
Intermediate	2% to 3%
Low	2% to 4%

Note: High-type pavements are those that retain smooth riding qualities and good nonskid properties in all weather under heavy traffic volumes and loadings with little maintenance.

Intermediate type pavements are those designed to retain smooth riding qualities and good nonskid properties in all weather, but under lighter loads and lesser traffic volumes.

Low-type surfaces are those with surface-treated earth surfaces and those with loose surfaces such as shell and gravel.

13-3.07 At-Grade Intersections

Chapter 6 presents the Department's criteria for the design of at-grade intersections. These apply in general to State-Aid projects. However, the designer should consider the following when using Chapter 6:

1. General Design Controls. Section 6-1.0 applies to State-Aid projects. However, considering the lower traffic volumes on these projects, it may not be cost effective to meet some of the design criteria in Section 6-1.0 (e.g., a 1% gradient on the storage platform).

2. Turning Radii Design. The criteria for rural intersections in Section 6-2.0 applies directly to State-Aid projects.

3. Turning Roadways. These will seldom be warranted on State-Aid projects. Where used, the designer should apply the criteria in Section 6-7.0.

4. Auxiliary Lanes. These will seldom be warranted on State-Aid projects. Where used, the designer should apply the criteria in Section 6-4.0.

5. Intersection Sight Distance. The criteria for intersection sight distance in Section 6-6.0 applies directly to State-Aid projects.

6. Channelization. This is seldom warranted for State-Aid projects. Where used, the designer should apply the criteria in Section 6-7.0.

7. Driveway Design. The criteria in Section 6-9.0 applies directly to State-Aid projects.

13-3.08 Traffic Control Devices

Signs and pavement markings and, where appropriate, traffic signal controls are essential elements for the highway. Traffic control devices for State-Aid projects shall comply with the current edition of the *Manual on Uniform Traffic Control Devices.*

13-3.09 Erosion Control

All slopes and drainage areas should be designed with proper regard for the desired natural ground cover and growth regeneration on areas opened during construction. Seeding and mulching of slopes and the sodding of swales and other erodible areas should be a part of the design.

Specific construction procedures are specified under applicable sections of the *Standard Specifications.*

13-3.10 Roadside Safety

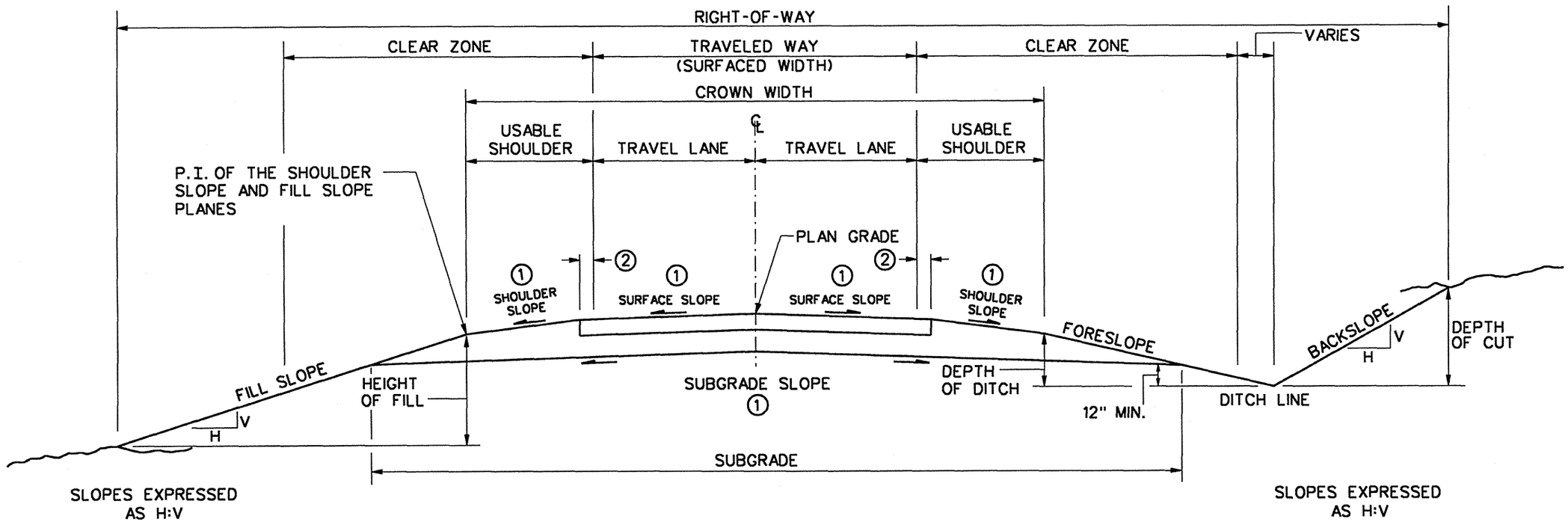
Chapter 9 presents the Department's roadside safety criteria. These criteria apply, as practical, to the roadside safety design of new construction/reconstruction of State-Aid projects.

13-3.11 Geometric Design Criteria

This Section presents design criteria for State-Aid projects which are new construction or reconstruction. Figure 13-3A illustrates the typical cross section elements for State-Aid projects. State-Aid design criteria for cross section and alignment elements are presented in Table 13-3E for rural State-Aid and Federal-Aid collector roads, Table 13-3F for local rural roads and Table 13-3G for LSBP projects.

13-3.12 Exceptions to Geometric Design Criteria

Designers of State-Aid projects are responsible for meeting all design criteria in the *Mississippi Design Manual*. The designer should also make a reasonable effort to provide a design which exceeds the minimum values. The Mississippi Department of Transportation and FHWA have established the controlling criteria for all Federal-aid projects. These design elements and the procedures for exceptions are presented in Section 2-8.0. The formal design exception process will also apply to non-Federal-aid projects.



Notes: ① Typical cross slopes:
Pavement Surfaces:
High: 2%
Intermediate: 2%-3%
Low: 2%-4%
Double Bituminous Surface Treatment: 2%
Shoulders: 4%
Subgrade: 3%

② Extend pavement base course 6 in. beyond the edge of the surfaced traveled way.

TYPICAL SECTION NOMENCLATURE
(State-Aid Roads)
Figure 13-3A

Table 13-3E

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (State-Aid and Federal-Aid Routes) (New Construction/Reconstruction)

	DESIGN ELEMENT		Manual Section	Under 400 ADT	400 - 1500 ADT	1501-2000 ADT	Over 2000 ADT
Design Controls	✳Design Speed (1)	Level	13-3.02	40 mph	50 mph		60 mph
		Rolling		30 mph	40 mph		50 mph
	Control of Access		2-4.0	Control by Regulation (Type 3)			
	Level of Service		2-3.04	C			
Cross Section Elements	✳Lane Width (2)		13-3.06	10′	11′		12′
	✳Usable Shoulder Width		13-3.06	4′	5′	6′	8′
	✳Cross Slope	Travel Lane	13-3.06	See Table 13-3D			
		Shoulder		4%			
	Auxiliary Lanes	Lane Width	2-6.03	10′	11′		
		Usable Shoulder Width		4′	5′	6′	8′
	✳New and Reconstructed Bridges	Structural Capacity	13-3.06; 11-4.0	HS-20			
		Minimum Width (3)		26′	28′	30′	40′
	✳Existing Bridges to Remain in Place	Structural Capacity	13-3.06; 11-4.0	HS-15			
		Minimum Width (4)		22′	22′	24′	28′
	Minimum Right-of-Way Width		13-3.06	See Section 13-3.06			
	Roadside Clear Zone	Guardrail	13-3.06; 9-4.03; 9-2.0	Usable Shoulder Width			
		Obstruction		See Note (5)			
	Slope Schedule (6) — Cut	Foreslope (within clear zone)	13-3.06	V ≤ 55 mph: 3:1 V > 55 mph: 4:1			
		Depth of Ditch (7)		1′			
		Backslope (8)		3:1			
	Slope Schedule (6) — Fill			Desirable — 6:1 Maximum — 3:1			

	DESIGN SPEED		30 mph	35 mph	40 mph	45 mph	50 mph	55 mph	60 mph
Alignment Elements	✳Minimum Stopping Sight Distance	13-3.03	185′	225′	275′	325′	380′	435′	505′
	✳Maximum Degree of Curvature (e = 10%)	13-3.04	24°45′	17°45′	13°15′	10°30′	8°15′	6°30′	5°15′
	✳Superelevation Rate	13-3.04	See State-Aid Standard SA-SE-1						
	✳Horizontal Sight Distance	3-5.0	(9)						
	✳Maximum Grades — Level	4-3.01	7%	7%	7%	6.5%	6%	5.5%	5%
	✳Maximum Grades — Rolling		9%	8.5%	8%	7.5%	7%	6.5%	6%
	Minimum Grades	13-3.05	Desirable: 0.4% Minimum: 0.0%						
	✳Vertical Curve (K-values)	4-4.0	Crest: 26 Sag: 33	Crest: 39 Sag: 43	Crest: 57 Sag: 56	Crest: 80 Sag: 69	Crest: 109 Sag: 84	Crest: 143 Sag: 99	Crest: 192 Sag: 118
	✳Minimum Vertical Clearance (Collector Under) — New/Reconstructed Bridges (10)	13-3.06	14′-6″						
	✳Minimum Vertical Clearance (Collector Under) — Existing Bridges		14′						
	Vertical Clearance (Collector Over Railroad)	13-3.06	23′						

✳Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(State-Aid and Federal-Aid Routes)
(New Construction/Reconstruction)

Footnotes to Table 13-3E

1. Design Speed. A rural collector may pass through a relatively built-up area. In these sections, a design speed as low as the posted speed limit may be used.

2. Lane Width. 12-ft lane widths are desirable for industrial routes.

3. Bridge Width (New/Reconstructed Bridges). For bridges in excess of 100 ft in length, a minimum bridge width of the traveled way plus 3 ft on each side will be acceptable. Note that the bridge widths in the Table assume minimum lane and shoulder widths; widen the bridge accordingly for wider approach roadway. Where the approach roadway is surfaced, the surfaced width shall be carried across all structures. See Section 11-4.0 for bridge projects performed as a spot improvement. Approaching guardrail transitions to the bridge rail are required on new bridges.

4. Bridge Width (Existing Bridges to Remain in Place). Clear width between curbs or rails, whichever is less, should be at least the same as the approach traveled way width. The Table does not apply when bridge length is greater than 100 ft. These structures should be evaluated individually considering clear width, traffic volumes, remaining life of structure, design speed and any other relevant factors. In addition, the criteria in the Table for the width of existing bridges to remain in place will apply when work is only performed on the bridge substructure (e.g., reinforcement) but not the bridge deck. Approaching guardrail transitions should be evaluated and replaced if warranted.

5. Roadside Clear Zone. See Section 13-3.06 for clear zone distances. All values are measured from the edge of traveled way.

6. Slopes. If high-volume change soil is present, see Section 12-2.04.

7. Depth of Ditch. For DHV $\leq$ 200 and design speeds of 45 mph and below, the depth of ditch may be reduced to 2 ft-6in. Maintain a minimum 1-ft depth of ditch from the subgrade shoulder. Adjust the foreslope distance as required in cuts in superelevated sections.

8. Backslope. These may be modified when located in special geological formation and/or land uses as follows:

   a. in loess formation, 1:4 (H:V) (not vegetated);
   b. in rock formations, 1:4 (H:V) (not vegetated); and
   c. in delta-cultivated lands, 1:1 to 2:1 (not vegetated).

9. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

10. Minimum Vertical Clearances. Table value includes an additional 6-in. allowance for future resurfacing.

Table 13-3F

GEOMETRIC DESIGN CRITERIA FOR STATE-AID LOCAL RURAL ROADS (Non-Federal-Aid Routes) (New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Under 100 ADT	100-250 ADT	> 250 ADT
Design Controls	✻Design Speed (1)		Level	13-3.02	30 mph	40 mph	SEE TABLE 13-3E
			Rolling		30 mph	30 mph	
	Control of Access			2-4.0	Control by Regulation (Type 3)		
	Level of Service			2-3.04	C		
Cross Section Elements	✻Lane Width			13-3.06	10′		
	✻Usable Shoulder Width			13-3.06	3′	4′	
	✻Cross Slope		Travel Lane	13-3.06	See Table 13-3D		
			Shoulder		4%		
	Auxiliary Lanes		Lane Width	2-6.03	9′		
			Usable Shoulder Width		4′		
	✻New and Reconstructed Bridges		Structural Capacity	13-3.06; 11-4.0	HS-20		
			Minimum Width (2)		26′		
	✻Existing Bridges to Remain in Place		Structural Capacity	13-3.06; 11-4.0	HS-15		
			Minimum Width (3)		22′		
	Minimum Right-of-Way Width			13-3.06	See Section 13-3.06		
	Roadside Clear Zone		Guardrail	13-3.06; 9-4.03; 9-2.0	Usable Shoulder Width		
			Obstruction (4)		10′		
	Slope Schedule (5)	Cut	Foreslope (within clear zone)	13-3.06	3:1		
			Depth of Ditch (6)		3′		
			Backslope (7)		1:1 Maximum		
		Fill	0.0 - 4 ft Height		3:1		
			> 4 ft Height		2:1		

	DESIGN SPEED		Manual Section	30 mph	35 mph	40 mph	45 mph	50 mph
Alignment Elements	✻Minimum Stopping Sight Distance		13-3.03	185′	225′	275′	325′	380′
	✻Maximum Degree of Curvature (e = 10%)		13-3.04	24°45′	17°45′	13°15′	10°30′	8°15′
	✻Superelevation Rate		13-3.04	See State-Aid Standard SA-SE-1				
	✻Horizontal Sight Distance		3-5.0	(8)				
	✻Maximum Grades	Level	4-3.01	7%	7%	7%	6.5%	6%
		Rolling		10%	9.5%	9%	8.5%	8%
	Minimum Grades		13-3.05	Desirable: 0.4% Minimum: 0.0%				
	✻Vertical Curve (K-values)		4-4.0	Crest: 26 Sag: 33	Crest: 39 Sag: 43	Crest: 57 Sag: 56	Crest: 80 Sag: 69	Crest: 109 Sag: 84
	✻Minimum Vertical Clearance (Local Under)	New/Reconstructed Bridges (9)	13-3.06	14′-6″				
		Existing Bridges		14′				
	Vertical Clearance (Local Over Railroad)		13-3.06	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR STATE-AID LOCAL RURAL ROADS
(Non-Federal-Aid Routes)
(New Construction/Reconstruction)

Footnotes to Table 13-3F

1. Design Speed. A local rural road may pass through a relatively built-up area. In these sections, a design speed as low as the posted speed limit may be used.

2. Bridge Width (New/Reconstructed Bridges). For bridges in excess of 100 ft in length, a minimum bridge width of the traveled way plus 1ft on each side will be acceptable. Note that the bridge widths assume minimum lane and shoulder widths; widen the bridge accordingly for wider approach roadway. See Section 11-4.0 for bridge projects performed as a spot improvement. Approaching guardrail transitions to the bridge rails are required on new bridges.

3. Bridge Width (Existing Bridges to Remain in Place). Clear width between curbs or rails, whichever is less, should be at least the same as the approach traveled way width. Table does not apply when bridge length is greater than 100 ft. These structures should be evaluated individually considering clear width, traffic volumes, remaining life of structure, design speed and any other relevant factors. In addition, the criteria in the table for the width of existing bridges to remain in place will apply when work is only performed on the bridge substructure (e.g., reinforcement) but not the bridge deck. Approaching guardrail transitions should be considered in special situations.

4. Roadside Clear Zone. See Section 13-3.06 for clear zone distances. All values are measured from the edge of traveled way.

5. Slopes. If high-volume change soil is present, see Section 12-2.04.

6. Depth of Ditch. Maintain a minimum 1-ft depth of ditch from the subgrade shoulder. Adjust the foreslope distance as required in cuts in superelevated sections.

7. Backslope. These may be modified when located in special geological formation and/or land uses as follows:

   a. in loess formation, 1:4 (H:V) (not vegetated);
   b. in rock formations, 1:4 (H:V) (not vegetated); and
   c. in delta-cultivated lands, 1:1 to 2:1 (not vegetated).

8. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

9. Minimum Vertical Clearances. Table value includes an additional 6-in. allowance for future resurfacing.

Table 13-3G
GEOMETRIC DESIGN CRITERIA FOR RURAL LSBP PROJECTS
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	Under 50 ADT	50 - 250 ADT	251 - 400 ADT	401 - 1500 ADT	1501 - 2000 ADT	Over 2000 ADT
Design Controls	✻Design Speed (1)		Level	13-3.02	30 mph	35 mph	40 mph	50 mph	60 mph	60 mph
			Rolling		30 mph	30 mph	30 mph	40 mph	50 mph	50 mph
	Control of Access			2-4.0	Control by Regulation (Type 3)					
	Level of Service			2-3.04	C					
Cross Section Elements	✻Lane Width			13-3.06	10′			11′	12′	
	✻Usable Shoulder Width			13-3.06	3′	4′ (2)	4′	6′	8′	
	✻Cross Slope		Travel Lane	13-3.06	See Table 13-3D					
			Shoulder		4%					
	Auxiliary Lanes		Lane Width	2-6.03	10′			11′		
			Usable Shoulder Width		3′	4′ (2)	4′	6′	8′	
	✻Bridges		Structural Capacity	13-3.06; 11-4.0	HS-20					
			Minimum Width/New and Reconstructed(3)		24′	26′	26′	28′	32′ (4)	40′ (4)
			Minimum Width/Rehabilitated (5)		22′	22′	22′	22′	24′	28′
	Minimum Right-of-Way Width			13-3.06	60′		See Section 13-3.06			
	Roadside Clear Zone		Guardrail	13-3.06; 9-4.03; 9-2.0	Usable Shoulder Width					
			Obstruction		See Note (6)					
	Slope Schedule	Cut	Foreslope (within clear zone)	13-3.06	2:1 Max.	3:1 Max (7)	3:1 Max.			
			Depth of Ditch (8)		2′	2′-6″	3′			
			Backslope (9)		3:1 to 1¼:1		3:1			
		Fill			2:1 Max.	0.0 - 4′ Ht — 3:1 > 4′ Ht — 2:1	3:1 Max.			
Alignment Elements	DESIGN SPEED				30 mph	35 mph	40 mph	45 mph	50 mph	55 mph	65 mph
	✻Minimum Stopping Sight Distance			13-3.03	185′	225′	275′	325′	380′	435′	505′
	✻Maximum Degree of Curvature (e = 10%)			13-3.04	24°45′	17°45′	13°15′	10°30′	8°15′	6°30′	5°15′
	✻Superelevation Rate			13-3.04	See State-Aid Standard SA-SE-1						
	✻Horizontal Sight Distance			3-5.0	(10)						
	✻Maximum Grades		Level	4-3.01	7% (11)	7% (11)	7%	7%	6%	6%	5%
			Rolling		9% (11)	8.5% (11)	8%	8%	7%	7%	6%
	Minimum Grades			13-3.05	Desirable: 0.4% Minimum: 0.0%						
	✻Vertical Curve (K-values)			4-4.0	Crest: 26 Sag: 33	Crest: 39 Sag: 43	Crest: 57 Sag: 56	Crest: 80 Sag: 69	Crest: 109 Sag: 84	Crest: 143 Sag: 99	Crest: 192 Sag: 118
	✻Minimum Vertical Clearance (Road Under)		New/Reconstructed Bridges (12)	13-3.06	14′-6″						
			Existing Bridges		14′						
	Vertical Clearance (Road Over Railroad)			13-3.06	23′						

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR RURAL LSBP PROJECTS (New Construction/Reconstruction)

Footnotes to Table 13-3G

1. Design Speed. Minimum values for design speed are presented.

2. Shoulder Width. Use 3 ft minimum for ADT under 100.

3. Bridge Width (New and Reconstructed Bridges). Note that the bridge widths assume minimum lane and shoulder widths; widen the bridge accordingly for a wider approach roadway. See Section 11-4.0 for bridges performed as a spot improvement.

4. Bridge Width (New and Reconstructed Bridges (ADT > 1500)). For bridges in excess of 100 ft in length, a minimum bridge width of the traveled way plus 3 ft on each side will be acceptable.

5. Bridge Width (Rehabilitated Bridges). In no case shall the minimum clear width be less than the approach traveled way width. See Section 11-4.0 for bridges performed as a spot improvement.

6. Roadside Clear Zone. See Section 13-3.06 for clear zone distances. All values are measured from the edge of traveled way.

7. Foreslope. Use 2:1 maximum for ADT under 100.

8. Depth of Ditch. For DHV ≤ 200 and design speeds of 40 mph and below, the depth of ditch may be reduced to 2′-6″. Maintain a minimum 1 ft depth of ditch from the subgrade shoulder. Adjust the foreslope distance as required in cuts and superelevated sections.

9. Backslope. These may be modified when located in special geological formation and/or land uses as follows:

   a. in loess formation, 1:4 (H:V) (not vegetated);
   b. in rock formations, 1:4 (H:V) (not vegetated); and
   c. in delta-cultivated lands, 1:1 to 2:1 (not vegetated).

10. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

11. Maximum Grades. For design speeds of 40 mph or less and ADT under 100, the maximum approach grade to a bridge without vertical curves shall be 3%; the maximum length of approach for bridges shall be 300 ft; the minimum length of approach for bridges shall be 100 ft; and the roadway width shall be the same width from the bridge end to a point a minimum of 21 ft from the bridge end.

12. Minimum Vertical Clearances. Table value includes an additional 6-in. allowance for future resurfacing.

13-4.0 GEOMETRIC DESIGN CRITERIA (3R Projects)

The criteria in this Section applies to the design of 3R projects on State-Aid roads which only applies to rural major collector facilities (State-Aid and Federal-Aid). The format is based on Section 11-2.0 "3R Projects (Non-Freeways)." The designer should refer to that Section for more discussion on the various design elements. In addition, much of the design criteria presented in Section 13-3.0 "Geometric Design Criteria (New Construction/Reconstruction)" and in other chapters will apply to 3R projects on State-Aid projects. Unless stated otherwise, the criteria for the new construction/reconstruction of State-Aid roads will apply to 3R projects.

13-4.01 3R Project Evaluation

Section 11-2.01 presents several factors which should be evaluated in the design of a 3R project (e.g., accident records). These apply to the evaluation of 3R projects on State-Aid roads.

13-4.02 Design Controls

The following will apply:

1. Design Speed. Table 13-4A presents the criteria for design speed. In addition, the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility.

2. Design Year Volume. Desirably, all elements of the facility will meet the level-of-service (LOS) criteria for an ADT or DHV determined for 10 years beyond the expected construction completion date. At a minimum, the highway facility should accommodate the LOS based on current traffic volumes as determined at the construction completion date.

3. Traffic Data. The necessary level of improvement will be determined by an analysis of the traffic data. In addition to the mainline ADT and DHV, this will include:

   a. percent of trucks and buses,
   b. high-accident locations,
   c. turning movements and traffic volumes at intersections, and
   d. any known near or future traffic impact.

13-4.03 Cross Section Elements

13-4.03.01 General

Table 13-4A presents the Department's criteria for cross section elements on 3R projects for State-Aid roads. The cross section widths of the existing road should be evaluated against the criteria in the 3R table. The Department's policy on the application of the cross section widths in the tables is as follows:

1. Existing Width Does Not Meet The 3R Width. The designer should consider widening the element. If the decision is made to widen the cross section element, the designer should provide a width which at least meets the 3R criteria. This will be sufficient for the majority of 3R projects. However, if practical, it may be appropriate to widen the element to meet new construction/reconstruction criteria.

2. Existing Width Does Meet The 3R Width. In most cases, it will not be cost effective to widen the cross section element. However, if practical, it may be appropriate to widen the element to meet new construction/reconstruction criteria.

Section 13-3.06 provides an elaboration on the cross section elements on State-Aid projects (e.g., surface and shoulder cross slopes) for new construction/reconstruction. Much of this discussion also applies to 3R projects. In addition, the cross section criteria in the following sections will apply.

13-4.03.02 Bridges

Scope of Work

For bridges within the limits of a 3R project, the bridge may be replaced, reconstructed, rehabilitated or remain in place. The existing structural capacity, geometrics, accident history, costs and historical significance of the bridge will determine the appropriate action. However, if an existing bridge is structurally sound and if it meets the Department's design loading structural capacity, it is unlikely to be cost effective to improve the geometrics of the bridge. However, the geometric deficiencies may be severe, and/or there may be an adverse accident experience at the bridge.

Bridge Width

In summary, the following will apply to widths of bridges within the limits of a 3R project:

1. Bridge Replacement. For this scope of work, the full approach roadway width according to new construction criteria (based on a 20-year traffic projection) will be carried across the bridge. See Section 13-3.0. However, bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to economic, traffic and safety studies.

2. Bridge Reconstruction/Bridge Deck Rehabilitation. For these scopes of work, the full approach 3R roadway width will be carried across the bridge. See Table 13-4A. Traffic volume projections will be based on the same design year as for the overall 3R project. However, bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to economic, traffic and safety studies.

3. Existing Bridges to Remain in Place. The existing width of bridges proposed to remain in place will be evaluated against the criteria in Table 13-4A. If the existing width does not meet these criteria, the designer must either widen the bridge as part of the 3R project or seek a design exception. However, bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to economic, traffic and safety studies.

   On low-volume roads, it may be appropriate to leave in place a bridge with a clear width of 18 ft or less. In this case, the approaches should be properly transitioned, signed and/or signalized to operate as a one-way bridge.

   All bridges which are narrower than the approach roadway width (including shoulders) should be evaluated for special narrow bridge treatments. At a minimum, the signing and pavement markings must meet the criteria of the MUTCD. In addition, NCHRP 203 *Safety at Narrow Bridge Sites* provides criteria specifically for narrow bridges (e.g., special pavement markings). The designer should evaluate the value of these additional treatments at the bridge site.

Bridge Rails/Transitions

For new, reconstructed or rehabilitated bridges, the existing bridge rails and approaching guardrail-to-bridge-rail transitions must be upgraded to meet current Department criteria. For existing bridges to remain in place within the limits of a 3R project, the Bridge Division will evaluate the adequacy of the existing bridge rail to determine if it should be upgraded. The roadway designer will evaluate the adequacy of the existing approaching bridge rail transition for needed upgrading. See Chapter 9 for more information on guardrail-to-bridge-rail transitions.

If an existing rail which will remain is structurally inadequate and does not meet the AASHTO loading and performance requirements (e.g., timber railing or the metal flex rail on a precast bridge), then a design exception will be requested by letter from FHWA on Federal-aid projects. Reasons for an exception request might include excessive cost to upgrade the rail, no accident history, or the possibility of increasing the number of posts to strengthen the rail.

All existing guardrail transitions approaching a bridge rail should be evaluated for practical improvements. The following specific criteria will apply:

1. Precast bridges with concrete post, metal rail systems and concrete rail and curb bridges of any length and width will be blocked out to the gutter line across the entire structure and connected to the approach guardrail.

2. Bridges that have existing approach guardrail that does not meet the current guardrail design shall have the old guardrail removed and new guardrail installed according to currently approved criteria.

13-4.04 Horizontal Alignment

The horizontal alignment criteria in Section 13-3.04 will apply to 3R projects on State-Aid roads, except as noted in the following paragraphs.

Figure 11-2A will be used to determine the design speed of an existing horizontal curve. This may require field measurements to attain the as-built information. The design speed of the existing curve should be compared to the design speed of the 3R project as determined from Table 13-4A.

Figure 11-2A can only be used to decide if corrective action should be considered. Once the decision has been made to improve the curve, the designer should use the criteria in Section 13-3.04 to determine the proper combination of degree of curvature and superelevation to meet the 3R design speed. This may be achieved by either flattening the curve or by increasing the superelevation rate on the curve.

Figure 11-2A can also be used to determine the appropriate advisory speed plate for signing curves which are below the 3R design speed but that will remain in place.

If the existing curve satisfies the 3R design speed, the designer will not normally need to check other details of the horizontal curve (e.g., superelevation runoff length, distribution of superelevation between tangent and curve).

13-4.05 Vertical Alignment

Section 11-2.05 presents the Department's vertical alignment criteria for 3R projects. This includes revised criteria for grades, crest vertical curves and sag vertical curves. These criteria apply to 3R projects on State-Aid roads, and they have been incorporated into Table 13-4A.

13-4.06 At-Grade Intersections

The design criteria in Chapter 6 will apply to 3R projects on State-Aid roads, except as noted in the following paragraph on intersection sight distance.

Section 6-6.0 presents the intersection sight distance (ISD) criteria for new construction/reconstruction projects. These also apply to ISD on 3R projects with the following revisions:

1. Location of Eye. At stop-controlled intersections, the location of the eye is 15 ft from the edge of traveled way on new construction/reconstruction projects. For 3R projects on State-Aid roads ($V \leq 45$ mph), the location of the eye may be assumed to be 10 ft from the edge of traveled way.

2. Sight Flares. It is often not practical to meet the Department's sight flare criteria on 3R projects.

13-4.07 Roadside Safety

The discussion and criteria in Section 11-2.08 on roadside safety applies to 3R projects on State-Aid roads, except as noted in the following for roadside clear zones:

1. Design Speed Greater Than 45 mph. The basic clear zone will be 14 ft from the edge of the traveled way. Side drains, utility poles, trees and other obstacles should be removed or relocated, if practical. If a side drain is relocated, it will desirably be placed at a distance equal to the clear zone for

new construction/reconstruction (see Section 13-3.06.05). The side slope on an intersecting drive or public road should be flattened, if practical. In addition, any headwalls or parapets that are within 4 ft of the usable shoulder should be relocated.

2. Design Speed 45 mph or Less (ADT > 750). The basic clear zone will be 10 ft from the edge of the traveled way. The discussion in #1 on side drains and intersecting slopes will also apply. In addition, any headwalls and parapets that are within 2 ft of the usable shoulder should be relocated. If the headwall or parapet is not relocated but is within 6 ft of the edge of the traveled way, an object marker should be installed according to the MUTCD.

3. Design Speed 45 mph or Less (ADT < 750). The basic clear zone will be 6 ft. The discussion in #1 on side drains and intersecting slopes will also apply. In addition, any headwalls and parapets that are within 1 ft of the usable shoulder should be relocated. If the headwall or parapet is not relocated but is within 6 ft of the edge of the traveled way, an object marker should be installed according to the MUTCD.

13-4.08 3R Table of Geometric Design Criteria

Table 13-4A presents the Department's 3R criteria for the geometric design of State-Aid projects (rural collectors).

13-4.09 Exceptions to Geometric Design Criteria

The discussion in Section 2-8.0 on design exceptions applies equally to the geometric design of 3R projects on State-Aid roads. The only difference, obviously, is that the designer will be evaluating the proposed design against the criteria presented in Section 13-4.0.

Table 13-4A
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (State-Aid and Federal-Aid Routes) (3R Projects)

	DESIGN ELEMENT		Design Year ADT (1) / T	Manual Section	0 - 750		751 - 1500		1501 - 2000		2001 - 4000		> 4000	
					T ≥ 10%	T < 10%	T ≥ 10%	T < 10%	T ≥ 10%	T < 10%	T ≥ 10%	T < 10%	T ≥ 10%	T < 10%
Design Controls	⁎Design Speed (2)		Level	13-4.02	40 mph		50 mph							
			Rolling		30 mph		40 mph							
	Control of Access			2-4.0	Control by Regulation (Type 3)									
	Level of Service			2-3.04	Desirable: C Minimum: D									
Cross Section Elements	⁎Lane Width (3)			13-4.03	10′		11′		V ≤ 45: 11′ V ≥ 50: 12′	11′	12′			
	⁎Usable Shoulder Width			13-4.03	2′		V ≤ 45: 2′ V ≥ 50:3′		V ≤ 45: 2′ V ≥ 50: 3′		6′	5′	6′	5′
	⁎Cross Slope		Travel Lane	13-3.06	See Table 13-3D									
			Shoulder		4%									
	Auxiliary Lanes		Lane Width	13-4.03	10′		11′							
			Usable Shoulder Width		4′									
	⁎Reconstructed/Rehabilitated Bridges (4)		Structural Capacity	13-4.03	HS-20									
			Minimum Width (5)		22′		24′		24′		28′			
	⁎Existing Bridges to Remain in Place		Structural Capacity	13-4.03	HS-15									
			Minimum Width (5)		Approach Traveled Way Width		Approach Traveled Way Width + 2′ OR 22′, whichever is less		Approach Traveled Way Width + 2′ OR 24′, whichever is less		Approach Traveled Way Width Plus 4′		Approach Traveled Way Width + 4′ OR 28′, whichever is less	
	Minimum Right-of-Way Width			13-3.06	(6)									
	Roadside Clear Zone		Guardrail	11-2.08; 13-4.07	Usable Shoulder Width									
			Obstruction		See Note (7)									
	Slope Schedule	Cut	Foreslope (within clear zone)	13-3.06	3:1									
			Depth of Ditch		2′									
			Backslope		Ditch Line < 10′ from ETL: 2:1 Ditch Line ≥ 10′ from ETL: 1:1									
		Fill	Height ≤ 10′		2:1									
			Height > 10′		3:1									
Alignment Elements	DESIGN SPEED				30 mph		35 mph		40 mph		45 mph		50 mph	
	⁎Minimum Stopping Sight Distance			13-3.03	185′		225′		275′		325′		380′	
	⁎Maximum Degree of Curvature			13-4.04	See Section 13-4.04									
	⁎Superelevation Rate			13-4.04	See Section 13-4.04									
	⁎Horizontal Sight Distance			3-5.0	(8)									
	⁎Maximum Grades		Level	11-2.05	9%		9%		9%		8.5%		8%	
			Rolling		11%		10.5%		10%		9.5%		9%	
	Minimum Grades			13-3.05	Desirable: 0.4% Minimum: 0.0%									
	⁎Vertical Curve (K-values)			11-2.05	See Section 11-2.05									
	⁎Minimum Vertical Clearance (Collector Under)		New/Reconstructed Bridges (9)	13-3.06	14′-6″									
			Existing Bridges		14′									
	Vertical Clearance (Collector Over Railroad)			13-3.06	23′									

⁎Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (State-Aid and Federal-Aid Routes) (3R Projects)

Footnotes to Table 13-4A

1. Design Year ADT. Desirably, the design year ADT will be 10 years beyond the construction completion date. At a minimum, it will be the current traffic volumes as of the construction completion date.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Widths. 12-ft lane widths are desirable for industrial routes.

4. Bridge Width (New Bridges). For new bridges within the limits of a 3R project, the minimum width will be the full approach roadway width according to the Department's criteria for new construction. However, bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

5. Bridge Width (Reconstructed Bridges/Rehabilitated Bridges/Existing Bridges to Remain in Place). Bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

   For reconstructed and rehabilitated bridges within the limits of a 3R project, a 20-ft width is acceptable on non-Federal-aid projects where the ADT $\leq$ 250 and the approaching travel lanes are 10 ft wide each.

   For existing bridges to remain in place, a 20-ft width is acceptable on non-Federal-aid projects for all traffic volumes where the approaching travel lanes are 10 ft wide each.

6. Right-of-Way Width. The acquisition of significant amounts of right-of-way is typically outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. However, the designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

7. Roadside Clear Zone. See Section 13-4.07 for clear zones. All values are measured from the edge of traveled way.

8. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

9. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

13-5.0 PAVEMENT DESIGN

This section establishes the methods, procedures and design values used for the base and pavement structure design on State-Aid projects.

13-5.01 General Criteria

Base and pavement structure design of State-Aid projects should adhere to several general criteria:

1. There is available computer software for calculating structural numbers for the N-year design life and the 20-year design life for comparison. The program is easy to use and allows for inputting varying data to achieve a proper pavement structure design. The printed data from the program is transferred onto the plans for a record of parameters used in the design. The software can be obtained from the Office of State Aid Road Construction.

2. To ensure that low ADT's and high CBR's do not reduce the base thickness below a reasonable depth, structural numbers shall not be less than 1.26.

3. The minimum thickness of all subbase layer courses will be 3 in., except under full depth asphalt construction, which will require a minimum thickness of 4-in. granular subbase.

4. Embankment should be constructed and compacted according to governing State-Aid specification requirements.

5. The ground water table should be at least 3 ft below the riding surface of the roadway.

6. Subbase and base courses in the pavement structure should be placed, manipulated and compacted according to governing State-Aid specification requirements.

13-5.02 Design Requirements

1. Pavement structures for State-Aid projects shall be designed for a maximum gross load of 84,000 lbs.

2. The designer should use the coefficients of relative strength for each layer thickness listed in Table 13-5D.

3. The minimum thicknesses for all base course layers are presented in Table 13-5A.

Table 13-5A
MINIMUM BASE COURSE THICKNESSES

Type of Base	Minimum Thickness
Granular Material, Class 1 - Class 4	6″
Cement Stabilized Granular Material	6″
Fly-ash Stabilized Granular Material	6″
Plant-Mixed Bituminous Base	4″
Asphalt Stabilized Base	6″

4. The minimum thickness of all subbase layer courses will be 3 in., except under full-depth asphalt construction, which will require a minimum thickness of 4-in. granular subbase.

5. On projects where a traffic analysis indicates a significant imbalance between auto and truck traffic, the designer should furnish all pertinent information (proposed industrial complex, recreational site and/or any other features that could influence the design traffic data) to the State-Aid District Engineer, who will request the design traffic data from the Department's Planning Division.

13-5.03 Design Procedure

13-5.03.01 Traffic Counts

Traffic counts should be obtained for each State-Aid project by using the latest edition of the Mississippi County Traffic Maps or, when required, the State-Aid District Engineer will request the Department's Planning Division to furnish the design traffic data.

13-5.03.02 Analysis of Traffic

The designer may assume the following information when using traffic counts from County Traffic Maps:

1. 10% trucks in the total ADT,
2. 675 trucks per 1000 for the design of flexible pavements, and
3. one-half of all trucks in each travel lane.

Traffic counts obtained from the Department's Planning Division should be furnished with all the required design information.

13-5.03.03 Analysis of Subgrade Material

A complete analysis of the subgrade material and underlying design soil should be conducted as follows:

1. Take a soil profile by boring and sampling the full depth of design soil (usually 3 ft) at intervals of between 200 ft and 500 ft.

2. Determine the AASHTO Soil Classification. Combination soils should be considered as the worst condition.

3. Determine the Unified Soil Classification.

4. Determine the Atterberg Limits and Volume Change.

5. Determine the Group Index.

6. Determine the Gradation.

7. Determine the Standard Density for A-3 Soils.

8. Estimate the CBR using Tables 13-5B or 13-5C for the appropriate soil classification.

A complete soils profile received from the preparing laboratory should contain all the information required in the above list.

13-5.03.04 Design of Pavement Structures

Using the traffic counts obtained from the County Traffic Maps or furnished by the Department's Planning Division and using the CBR determined by information from the soils profile, the designer should determine the pavement structure as follows:

1. Using the computer software that is available from the Office of State-Aid Road Construction, calculate the required Structural Number (SN) for the N-year design life and the 20-year design life:

   a. Based on the DHV, the following applies:

      - Five-year design life for DHV $\leq$ 200.
      - Eight-year design life for DHV 201-400.
      - Ten-year design life for DHV over 400.
      - Ten-year design life (minimum) for all curb and gutter construction.

b. To ensure that low ADT's and high CBR's do not reduce the base thickness below a reasonable depth, the required SN for the N-year design shall not be less than 1.26.

c. The value for PT shall be equal to 2.5.

2. After determining the required SN, the pavement structure may now be developed using the appropriate layer coefficients for the various pavement components. These are provided in Table 13-5D.

3. Thin hot plant mix surfaces, 2 in. or less on new construction, will be placed on a single surface treatment. This should generally apply to new untreated granular bases where hot mix is used as a riding surface for high traffic volumes or unusual truck traffic. This requirement may be waived on short sections such as bridge approaches or short sections over box culvert projects. Hot plant mix surface courses of less than 2 in. shall not be used as part of the base design thickness and granular bases.

Table 13-5B
METHOD OF ESTIMATING CBR
(A-4, A-6 and A-7 soils)

AASHTO Soil Class	Unified Soil Class(es)	Liquid Limit	Plasticity Index	Volume Change	Group	Estimated CBR
A-7	CH	41 Min.	11 Min.	125	-	1
	CH, MH			100-125	-	2
	CH MH CL			75-99	-	3
				63-74	-	4
				62	-	5
A-6	CL or ML-CL	40 Max.	11 Min.	35-55	-	5
				26-34	-	6
				17-25	-	7
				-	10	8
				-	9	9
	SC, SMu, or SM-SC	40 Max.	11 Min.	-	8	10
				-	7	11
				-	6	12
				-	5	14
				-	5	15
A-4	CL	40 Max.	10 Max.	17	-	7
				17	-	8
	ML or ML-CL	40 Max.	10 max.	16	-	8
				9-16	-	9
				0-8	-	10
	ML	25	NP	-	-	12
		25		-	-	13
	SC, SM or SM-SC	40 Max.	7-10	-	-	15
			4-6	-	-	16
			3	-	-	17
			2	-	-	18
			1	-	-	19
	SMd	40 Max.	NP	-	-	20

Controlling Data:

A-7 Soils: Volume Change
A-6 Soils: Volume Change & Group Index
A-4 Soils: Liquid Limit, Plasticity Index and Volume Change

Table 13-5C
METHOD OF ESTIMATING CBR
(A-2 and A-3 soils)

AASHTO Soil Class	Unified Soil Class(es)	Plasticity Index	Percent Passing No. 40 Sieve	Standard Density (lbs/ft³)	Estimated CBR
A-3	"S" Soils and "G" Soils	NP	-	116	11-15
			-	117 - 119	16-20
			-	119	20+
A-2	"S" Soils	7-12	-	-	24
		1-6	-	-	26
		NP	95-100	-	26
		NP	85-94	-	28
		NP	85	-	30
		15	-	-	45
		14	-	-	46
		13	-	-	47
		12	-	-	48
		11	-	-	49
		10	-	-	50
		9	-	-	51
		8	-	-	52
		7	-	-	53
		6	-	-	54
		5	-	-	55
		4	-	-	56
		3	-	-	57
		2	-	-	58
		1	-	-	59
		NP	-	-	60

Controlling Data:
A-3 Soils: Standard Density
A-2 Soils: Plasticity Index and/or Percent Passing No. 40 Sieve

Table 13-5D
PAVEMENT STRUCTURE LAYER COEFFICIENTS

Pavement Component	Coefficients (per inch)
Surface Course	
Hot Bituminous Pavement	0.44
Double Bituminous Surface Treatment	0.00
Base Course	
Plant Mix Bituminous Base Course	0.34
Cement Treated Granular Material (Class 1 or 2, Group D)	0.23
Cement Treated Granular Material (Class 3-6, Group D)	0.20
Cement Treated Granular Material (Class 9, Group D)	0.17
Granular Base Course (Class 1-4, Group A or B)	0.10
Crushed Stone	0.14
Subbase Course	
Granular Subbase Course (Class 1 or 2, Group C)	0.12
Granular Subbase Course (Class 3 or 4, Group C)	0.11
Granular Subbase Course (Class 5 or 6, Group C)	0.10
Granular Subbase Course (Class 9, Group C)	0.09
Cement Treated Granular Material (Class 9, Group D)	0.20
Design Soil Treatment	
Lime Treated Course	0.06
Lime-Cement Treated Course	0.15
Cement or Flyash Treated Course	0.15 - 0.20

Note: See Table 13-5A for minimum thickness requirements.

Chapter 14

Geometric Design of Urban Streets

MISSISSIPPI DESIGN MANUAL

Chapter 14
GEOMETRIC DESIGN OF URBAN STREETS

Section		Page
14-1.0	BASIC DESIGN CONTROLS	14-1
14-1.01	Highway Systems	14-1
14-1.01.01	Functional Classification System	14-1
14-1.01.02	Federal-Aid System	14-1
14-1.01.03	Jurisdictional Systems	14-1
14-1.02	Project Scope of Work	14-1
14-2.0	URBAN STREETS (New Construction/Reconstruction)	14-2
14-2.01	Design Traffic Volumes	14-2
14-2.01.01	Design Hourly Volume	14-2
14-2.01.02	Level of Service	14-2
14-2.02	Design Speed	14-2
14-2.03	Stopping Sight Distance	14-2
14-2.04	Horizontal Alignment	14-2
14-2.04.01	Curve Types	14-2
14-2.04.02	Superelevation Rate and Maximum Degree of Curvature	14-3
14-2.04.03	Superelevation Transition Length	14-6
14-2.04.04	Superelevation Axis of Rotation	14-6
14-2.04.05	Horizontal Sight Distance	14-7
14-2.05	Vertical Alignment	14-7
14-2.05.01	Grades	14-7
14-2.05.02	Vertical Curves	14-8
14-2.06	Cross Section Elements	14-8
14-2.06.01	Lane Widths	14-9
14-2.06.02	Shoulder Width and Type	14-9
14-2.06.03	On-Street Parking	14-9
14-2.06.04	Medians	14-11
14-2.06.05	Curb and Gutter Usage	14-11
14-2.06.06	Sidewalks	14-12
14-2.06.07	Bridges	14-13
14-2.06.08	Roadside Clear Zone	14-13
14-2.06.09	Right-of-Way Width	14-14
14-2.07	At-Grade Intersections	14-14
14-2.07.01	Application of Chapter 6	14-14
14-2.07.02	Urban Driveways	14-15
14-2.08	Geometric Design Criteria	14-19
14-2.09	Roadside Safety	14-19
14-2.10	Exceptions to Geometric Design Criteria	14-19
14-3.0	URBAN STREETS (3R Projects)	14-29
14-3.01	3R Project Evaluation	14-29
14-3.02	Design Controls	14-29
14-3.03	Cross Section Elements	14-29
14-3.04	Horizontal Alignment	14-29
14-3.05	Vertical Alignment	14-30
14-3.06	At-Grade Intersections	14-30
14-3.06.01	Turning Radii Design	14-30
14-3.06.02	Auxiliary Lanes	14-30
14-3.06.03	Intersection Sight Distance	14-31
14-3.07	Roadside Safety	14-31
14-3.08	3R Tables of Geometric Design Criteria	14-31
14-3.09	Exceptions to Geometric Design Criteria	14-31

Chapter 14

GEOMETRIC DESIGN OF URBAN STREETS

Rural and urban areas vary in the nature and intensity of development and in their relative traffic volumes. This warrants revised geometric design criteria for urban streets relative to the criteria for rural highways presented in Chapters 2 and 13. The differences include the criteria for superelevation rate, superelevation development, the length of vertical curve and the roadside clear zone. In addition, special cross section elements should be addressed in urban areas. These include parking lanes, sidewalks and the use of curbed sections.

This Chapter provides the geometric design criteria applicable to most urban streets. In many cases, the criteria and procedures presented in other chapters are equally applicable to urban streets. Where practical, this Chapter references the material presented elsewhere.

14-1.0 BASIC DESIGN CONTROLS

14-1.01 Highway Systems

14-1.01.01 Functional Classification System

The Department's design criteria for urban streets is based on the functional classification system. The classifications for urbanized areas include arterial streets, collector streets and local streets. These designations lead directly to the tables of geometric design criteria in Sections 14-2.0 and 14-3.0. Section 2-1.01 discusses functional classification further.

14-1.01.02 Federal-Aid System

The Department, together with the local municipalities and FHWA, has designated those routes within the urban areas which are eligible for Federal highway funds.

14-1.01.03 Jurisdictional Systems

The municipality is responsible for all urban streets which are not designated as State routes. This includes the design, construction and maintenance of these streets.

14-1.02 Project Scope of Work

The project scope of work may be new construction, reconstruction, 3R or spot improvement. The definitions presented in Section 2-5.0 also apply to projects on urban streets.

14-2.0 URBAN STREETS (New Construction/Reconstruction)

The criteria in this Section apply to all functional classes unless otherwise noted.

14-2.01 Design Traffic Volumes

14-2.01.01 Design Hourly Volume

For new construction and reconstruction, urban streets should be designed to accommodate traffic volumes that may occur within the projected life of the facility (usually 20 years from completion of construction). The Planning Division maintains current records of expected future traffic data, including the design hourly volume (DHV) and average daily traffic (ADT).

14-2.01.02 Level of Service

The highway mainline and at-grade intersections should be designed to accommodate the DHV at the selected level of service. The highway designer is responsible for the capacity analysis and should be familiar with the detailed methodologies presented in the *Highway Capacity Manual.* Section 2-3.0 provides highway capacity definitions. Level of service criteria for urban streets are provided in the tables in Section 14-2.08.

14-2.02 Design Speed

The design speed for urban streets should be selected based on the adjacent land use and the functional classification. The geometric design elements should be consistent with the selected design speed. Design speed is discussed in detail in Section 2-2.0. Design speeds for urban streets are provided in the tables in Section 14-2.08. Table 14-2F presents values for geometric design elements based on design speed.

14-2.03 Stopping Sight Distance

Stopping sight distance values are provided in Table 14-2F. Stopping sight distance is measured from the driver eye 3.5 ft above the pavement to a 6-in. height of object. Sight distance is discussed in more detail in Section 2-6.0.

14-2.04 Horizontal Alignment

The Department has adopted three sets of criteria for horizontal alignment based on design speed. These are briefly summarized:

1. V > 50 mph. The criteria and procedures presented in Chapter 3 "Horizontal Alignment" will apply to these urban roads and streets. This includes e_{max} = 10% and a theoretical distribution of "e" and "f" which AASHTO has designated as Method 5. See *A Policy on Geometric Design of Highways and Streets* for more information on the distribution methods.

2. V = 50 mph. For these urban roads and streets, e_{max} = 6%. The Method 5 distribution between "e" and "f" is also used. The following discussion in Section 14-2.04 presents the details of horizontal alignment for these facilities.

3. V ≤ 45 mph. For low-speed urban streets, e_{max} = 6%. The distribution of "e" and "f" is based on the AASHTO Method 2. See *A Policy on Geometric Design of Highways and Streets* for more information on this distribution method. The following discussion in Section 14-2.04 presents the details of horizontal alignment for low-speed urban streets.

14-2.04.01 Curve Types

Deflectional changes in alignment on urban streets (V ≤ 50 mph) are almost always accomplished by using a simple curve. Where some physical control prevents the

use of a simple curve, a spiral or compound curve may be used. Section 3-3.0 discusses the details of each curve type.

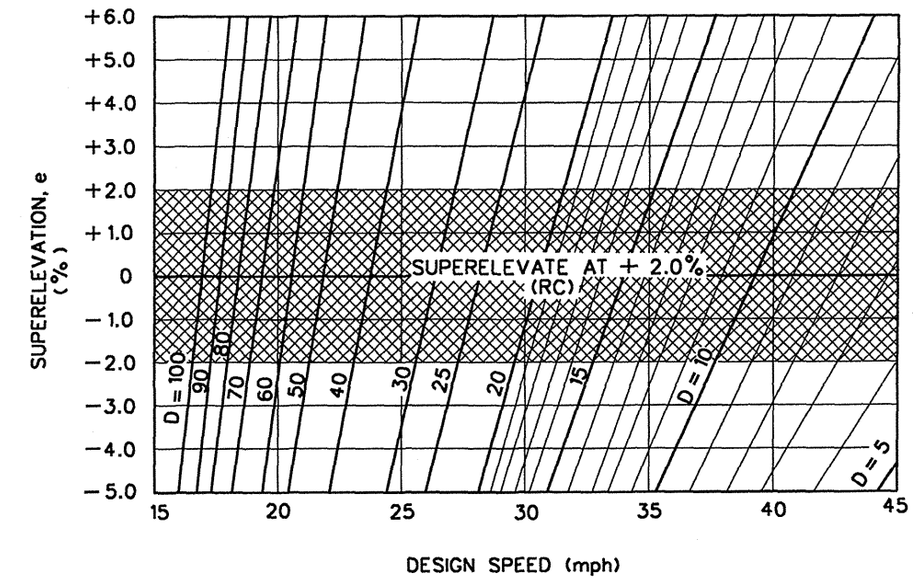
Very small deflection angles may not require a horizontal curve; i.e., the roadway may be designed with an angular break. As a general guide, the designer should evaluate the need for a curve when the deflection angle is less than 1 degree. The evaluation will be based on roadside development, aesthetics, construction costs and the visibility of the kink if left in.

14-2.04.02 Superelevation Rate and Maximum Degree of Curvature

The Department has adopted e_{max} = 6% for urban streets with V ≤ 50 mph. Table 14-2F provides the maximum degree of curvature for various design speeds. The designer should provide flatter curves where practical, especially where the maximum degree of curvature will not provide the necessary middle ordinate for SSD around horizontal curves (see Section 3-5.0). The following will apply:

1. V ≤ 45 mph. Figure 14-2A provides the criteria for superelevation rates for urban streets based on design speed and degree of curvature. When the Figure yields a superelevation rate greater than the negative cross slope, the curve should be superelevated at the normal cross slope across the entire pavement. Table 14-2A provides the corresponding degree of curvature range for a normal crown slope and a reverse crown slope at various design speeds assuming a cross slope of 2%. Examples 1, 2 and 3 illustrate the use of Figure 14-2A and Table 14-2A to determine the proper superelevation rate.

2. V = 50 mph. Table 14-2B provides the criteria for these urban roads and streets based on design speed and degree of curvature. Table 14-2C provides the degree of curvature range for a normal crown slope and a reverse crown slope. This is based on the design speed (50 mph) and typical cross slope (2%).



The basic equation for the Figure is: $D = \frac{85{,}660\ (e + f)}{V^2}$

where:
- *D* = *degree of curvature*
- *V* = *design speed, mph*
- *e* = *superelevation rate, decimal*
- *f* = *side-friction factor*

The values in the Figure are the calculated (unrounded) values. For design purposes, the rounded maximum degrees of curvature are used as follows:

	Design Speed, mph					
	20	25	30	35	40	45
D_{max} (e = 6.0%)	77°00′	42°45′	26°45′	18°00′	12°45′	9°30′

RATE OF SUPERELEVATION
(Low-Speed Urban Streets, V ≤ 45 mph)

Figure 14-2A

Table 14-2A

DEGREE OF CURVATURE RANGE FOR NORMAL CROWN SLOPE AND REVERSE CROWN SLOPE ($V \leq 45$ mph)

Cross Slope = 2%

Design Speed (mph)	Degree of Curvature			
	Normal Crown	Reverse Crown	See Figure 14-2A	D_{max}
20	D < 59°58′	59°58′ ≤ D ≤ 68°32′	D > 68°32′	77°00′
25	D < 31°48′	31°48′ ≤ D ≤ 37°17′	D > 37°17′	42°45′
30	D < 19°08′	19°08′ ≤ D ≤ 22°56′	D > 22°56′	26°45′
35	D < 12°23′	12°23′ ≤ D ≤ 15°10′	D > 15°10′	18°00′
40	D < 8°28′	8°28′ ≤ D ≤ 10°36′	D > 10°36′	12°45′
45	D < 6°03′	6°03′ ≤ D ≤ 7°44′	D > 7°44′	9°30′

Example 1:

Given: Design speed = 30 mph
Degree of curvature = 10°
Cross slope = 2%

Problem: Determine the superelevation rate.

Solution: From Table 14-2A, a normal crown should be maintained throughout the curve.

Example 2:

Given: Design speed = 40 mph
Degree of curvature = 10°
Cross slope = 2%

Problem: Determine the superelevation rate.

Solution: From Table 14-2A, a reverse crown section applies. The pavement should be superelevated at a rate of +2.0% across the entire pavement.

Example 3:

Given: Design speed = 45 mph
Degree of curvature = 8°30′
Cross slope = 2%

Problem: Determine the superelevation rate.

Solution: Table 14-2A references Figure 14-2A for the superelevation rate. Figure 14-2A yields a required superelevation rate of approximately 3.8%. Therefore, the entire pavement should be transitioned to this rate.

Table 14-2B

SUPERELEVATION RATE AND LENGTH OF RUNOFF (Urban Roads/Streets, V = 50 mph)

D*	V = 50 mph		
		L (ft)	
	e	A	B
0°30′	NC	0	0
0°45′	RC	150	150
1°00′	RC	150	150
1°30′	.028	150	150
2°00′	.035	150	150
2°30′	.040	150	150
3°00′	.045	150	160
3°30′	.048	150	170
4°00′	.052	150	180
5°00′	.056	150	200
6°00′	.059	150	210
	D_{max} = 6°45′		

Key:

D = Degree of curvature
V = Design speed, mph
e = Superelevation rate, %
L = Minimum length of superelevation runoff (from adverse crown removed to full superelevation), ft
NC = Normal crown (2% cross slope)
RC = Reverse crown; superelevate at normal crown slope (2%)

* *For degree of curvature intermediate between table values, use a straight-line interpolation to determine the superelevation rate.*

Table 14-2C

DEGREE OF CURVATURE RANGE FOR NORMAL CROWN AND REVERSE CROWN SLOPE (V = 50 mph)

Cross Slope = 2%

Design Speed (mph)	Degree of Curvature		
	Normal Crown	Reverse Crown	See Table 14-2B
50	D < 0°44′	0°44′ ≤ D ≤ 1°01′	D > 1°01′

Note: e_{max} = 6%.

Notes:

1. e_{max} = 6%

2. Superelevation runoff length (L) is: $L = W \times \frac{e}{S}$, subject to minimum design values,

   where: W = width of pavement rotation (12-ft lanes assumed)
   e = superelevation rate, decimal
   S = maximum longitudinal slope compared to normal highway profile

3. Maximum longitudinal slope:

Design Speed	50 mph
S	0.0050

4. The equation for superelevation runoff length is used directly for rotation of a 1-lane width (i.e., two 12-ft lanes rotated about the centerline), subject to minimum design values. These lengths are listed in Column A. Column B lists the lengths for rotation of a 2-lane width (i.e., a 2-lane pavement rotated about the edge of traveled way or a 4-lane undivided pavement rotated about the centerline). The Column B lengths are computed by:

$$L = \frac{1.5 \times W \times e}{S}$$

For greater widths of rotation:

L(CTWLTL) = 1.2 x L (Column B) *Note: Assuming that the axis of rotation is about centerline of 5-lane section.*
L (3 lanes) = 1.33 x L (Column B)
L (4 lanes) = 1.67 x L (Column B)

5. Total superelevation transition (from "NC" to "e") = superelevation runoff length (L) + tangent runout length. Tangent runouts will be 50 ft for a 1-lane width of rotation and 75 ft for a 2-lane (or more) width of rotation.

6. See Table 14-2C for a more precise range for using "NC" and "RC" to superelevate the roadway.

14-2.04.03 Superelevation Transition Length

The total superelevation transition length consists of two segments — the length of tangent runout and the length of superelevation runoff. Figure 14-2B illustrates the typical application to urban roads and streets (V ≤ 50 mph). The application of the total superelevation transition length should be approximately 50 percent of the runoff length within the circular curve. The tangent runout length is applied at the outer limits of the superelevation runoff sections. Superelevation transition length is discussed in detail in Section 3-4.02.

The following will apply:

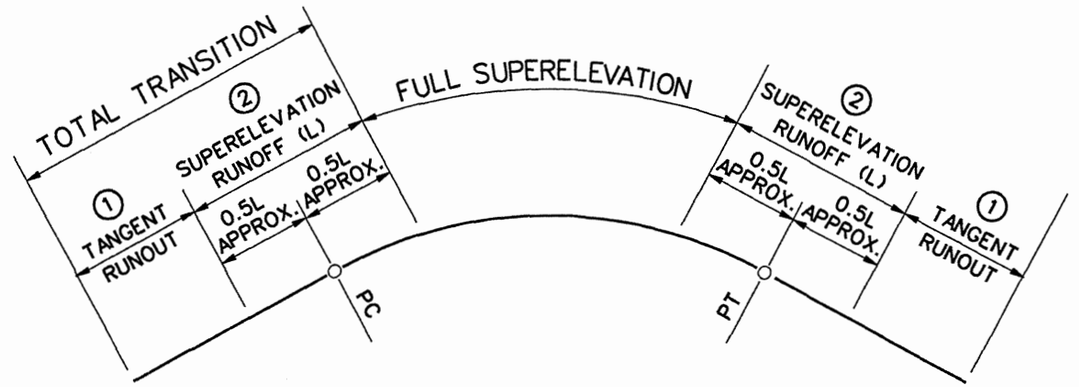
1. V ≤ 45 mph. Table 14-2D provides the minimum superelevation runoff lengths for combinations of design speed and superelevation rate. The tangent runout distance should be 25 ft.

2. V = 50 mph. Table 14-2B provides the minimum length of superelevation runoff for 50 mph, the degree of curvature and pavement rotation width. The tangent runout distance should be 50 ft for a 1-lane width of rotation and 75 ft for a 2-lane (or more) width of rotation.

3. V > 50 mph. See Chapter 3.

14-2.04.04 Superelevation Axis of Rotation

The superelevation axis of rotation on undivided urban streets most often follows the centerline of the roadway. This applies to 2-lane, 4-lane and 5-lane (CTWLTL) sections. Occasionally, it may be warranted to rotate the pavement about the outside edge of the traveled way on undivided facilities. This may be preferable where factors including roadside development, drainage requirements and utilities create the need to keep one edge at a fixed location. Superelevation axis of rotation is discussed in more detail in Section 3-4.0.



Notes:

① *Tangent runout lengths should be as follows:*

a. V ≤ 45 mph: 25 ft

b. V = 50 mph: 50 ft (1-lane width of rotation)
75 ft (2-lane or more width of rotation)

② *Superelevation runoff lengths should be as follows:*

a. V ≤ 45 mph: See Table 14-2D
b. V = 50 mph: See Table 14-2B

APPLICATION OF THE SUPERELEVATION TRANSITION LENGTH

Figure 14-2B

Table 14-2D

SUPERELEVATION RUNOFF LENGTHS
(Urban Streets, V ≤ 45 mph)

Design Speed (mph)	"e"						
	≤ 3.0%	3.5%	4.0%	4.5%	5.0%	5.5%	6.0%
20	40′	45′	50′	60′	65′	70′	75′
25	40′	50′	55′	60′	70′	75′	80′
30	45′	55′	60′	70′	75′	85′	90′
35	55′	60′	65′	75′	80′	90′	95′
40	60′	60′	70′	80′	85′	95′	105′
45	70′	70′	75′	85′	95′	100′	110′

Notes:

1. $e_{max} = 6\%$

2. *Superelevation runoff length (L) is:* $L = \frac{W \times e}{S}$ *, subject to minimum design values, where:*

   W = width of pavement rotation (12-ft lanes assumed)
   e = superelevation rate, decimal
   S = maximum relative longitudinal slope compared to normal highway profile

3. *Maximum relative longitudinal slope:*

Design Speed (mph)	20	25	30	35	40	45
S	.0096	.0090	.0082	.0076	.0071	.0066

4. *Superelevation runoff length (L) from table is used directly for rotation of 1-lane width (i.e., two 12-ft lanes rotated about the centerline). For greater widths of rotation:*

   L (2 lanes) = 1.5 x L (Table)
   L (3 lanes) = 2.0 x L (Table)

5. *Total superelevation transition (from "NC" to "e") = superelevation runoff length (L) + tangent runout length (25 ft).*

On divided urban streets with a raised median, the axes of rotation are typically about the two median edges.

See the *Roadway Design Standard Drawings* for additional details on superelevation development.

14-2.04.05 Horizontal Sight Distance

The criteria presented in Section 3-5.0 apply to horizontal sight distance on urban streets.

14-2.05 Vertical Alignment

The criteria and procedures presented in Chapter 4, "Vertical Alignment" will apply to urban streets with the following additional guidance.

14-2.05.01 Grades

Grades on urban projects are based on the functional classification and on controlling conditions including existing streets and adjacent property development. Maximum grades for urban streets are provided in Tables 14-2G through 14-2I. Where outside controlling factors are not severe, the normal practice of carrying the profile grade on the centerline or on the median edges is acceptable. Where the 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. Supplemental controls should be shown on the typical section and on the plan-profile sheets.

Special care should be taken to avoid flat spots where water may pond. Curb lines should have a gradient of at least 0.2% and preferably 0.4%. Curb elevations should be designed to permit drainage flow into the gutter and avoid ponding of water behind

the curb. On crest vertical curves, it is desirable to locate VPI's near the centerline of intersecting cross streets.

Spline curves can be helpful in laying grades in urban areas where it is necessary to meet numerous elevations in relatively short distances. Elevations along spline curves should be shown at 25-ft intervals.

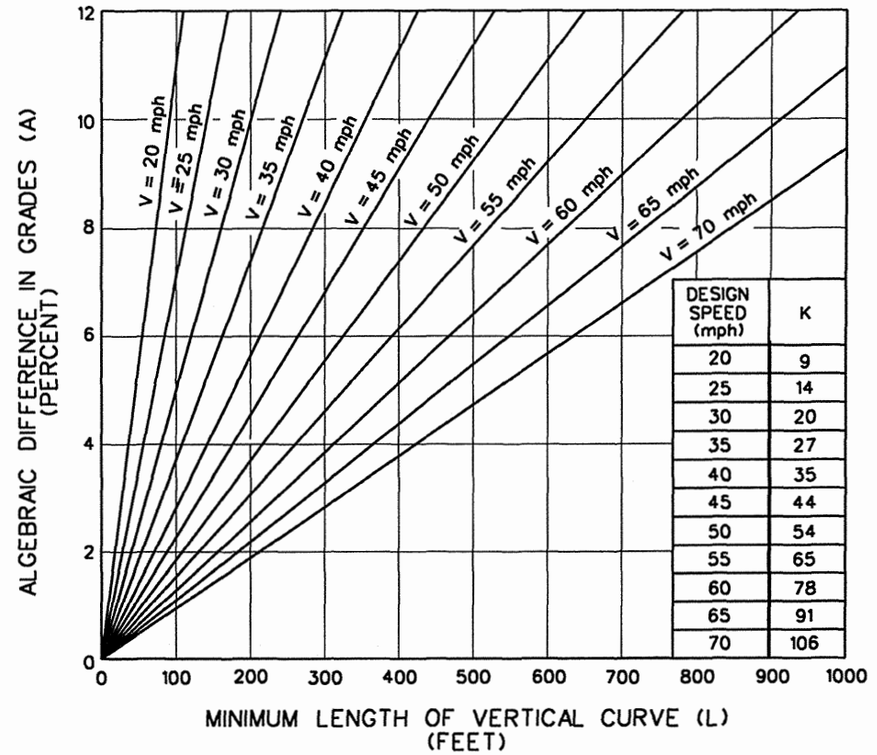
Where roadside development is extensive and the general elevation on one side is higher than on the other, an asymmetrical section may be required. The crown point may be offset from centerline, and the total drop from crown line to gutter line will be more than normal on one side and less than normal on the other. Asymmetrical features must be clearly defined in the typical sections and shown in the cross sections.

14-2.05.02 Vertical Curves

Vertical curves are not required where the algebraic difference in grades is 0.2% or less. Where vertical curves are required, they should be long enough to provide the required stopping sight distance. The criteria in Section 4-4.0 will apply to urban streets. In lighted areas on urban streets, the comfort criteria may be used for the design of sag vertical curves. Figure 14-2C provides the lengths of sags for various algebraic differences in grade and various design speeds. The minimum length of all vertical curves (crest and sag) for urban streets is 50 ft.

14-2.06 Cross Section Elements

Figures 14-2F and 14-2G illustrate typical highway cross sections on urban streets with or without curbs. The design criteria for the cross section elements are provided in Tables 14-2G through 14-2I. Section 2-6.0 presents a discussion on each cross section element, and these apply in part to the design of urban streets. The following additional guidance is provided specifically for urban street projects.



DESIGN SPEED (mph)	K
20	9
25	14
30	20
35	27
40	35
45	44
50	54
55	65
60	78
65	91
70	106

$$L = \frac{AV^2}{46.5}$$

$$L = KA$$

Where:
L = Length of vertical curve, ft
A = Algebraic difference between grades, %
K = Horizontal distance required to effect a 1% change in gradient, ft

Notes:
1. *L_{min} = 50 ft*
2. *This figure only applies in lighted areas on new construction and reconstruction. For unlighted areas, see Section 4-4.0.*

LENGTH OF SAG VERTICAL CURVES
(Comfort Criteria, Lighted Areas Only)

Figure 14-2C

14-2.06.01 Lane Widths

Surfaced lane widths vary according to functional classification and whether or not curbs are provided. These are presented in Tables 14-2G through 14-2I. Where a curb and gutter section is provided, the gutter pan shall not be considered part of the traveled way. However, for auxiliary lanes and where V ≤ 40 mph, the gutter pan may be considered part of the auxiliary lane width; where V ≥ 45 mph, this is not desirable. Also note that auxiliary lanes are not part of the traveled way.

14-2.06.02 Shoulder Width and Type

Shoulders are desirable on any urban street to provide space to maneuver and for emergency parking. However, space for shoulder width may be limited by the available right-of-way in built-up areas. The following should be used to determine the shoulder width on urban streets:

1. Where curbs are not provided, the shoulder width should equal the criteria provided in Tables 14-2G through 14-2I. The "Without Curb" column will apply.

2. Where curbs are provided, the shoulder width will be measured from the edge of the outside travel lane to the gutter line, including any gutter pan, at the base of the curb. This also applies where the gutter pan contrasts in type of material, color, texture and/or cross slope from the remaining pavement.

3. On curbed streets, the shoulder widths in Tables 14-2G through 14-2I should be provided. The "With Curb" column will apply. When selecting the shoulder width, the designer should evaluate the construction costs, right-of-way impacts, increased capacity, design speed, traffic volumes and the potential use of the wider shoulder as a continuous right-turn lane, particularly where there are frequent points of access to the street.

4. On curbed streets with a design speed greater than 40 mph, mountable curbs should be used.

5. Where on-street parking is provided, the parking lane widths in Tables 14-2G through 14-2I should be provided. However, if the parking lane may in the future be converted to a travel lane, the designer should consider increasing the width accordingly.

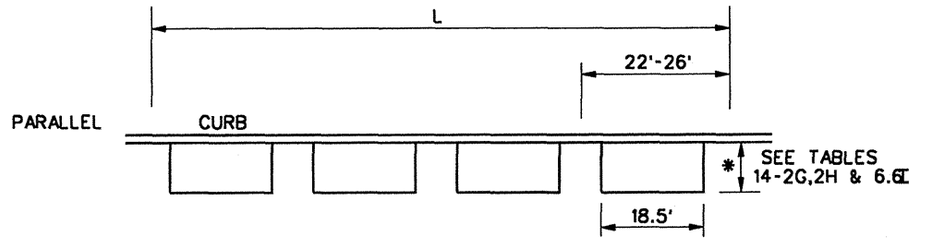
Where curbs are not provided, shoulders generally will not be surfaced. Usable shoulders should be graded and measured from the edge of the traveled way to the intersection of the shoulder slope and the foreslope (or safety slope) planes (see Figure 2-7A). Where curbs are provided, the shoulder width will be paved.

14-2.06.03 On-Street Parking

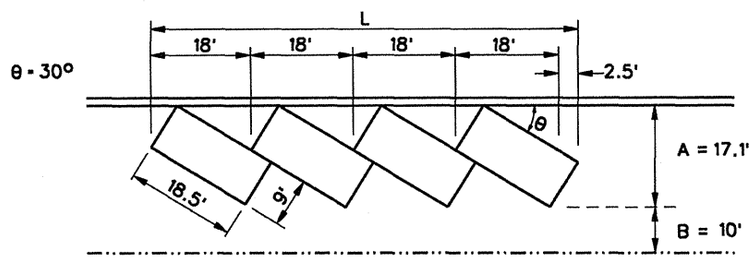
The designer should evaluate the following when providing on-street parking on an urban street:

1. Warrants. Adjacent land use may create the need to provide on-street parking along an urban street. This provides convenient access for motorists to businesses and residences. However, on-street parking reduces capacity, impedes traffic flow, may produce undesirable traffic operations and may increase the accident potential. In particular, highway safety may be significantly reduced where the design speed is above 45 mph.

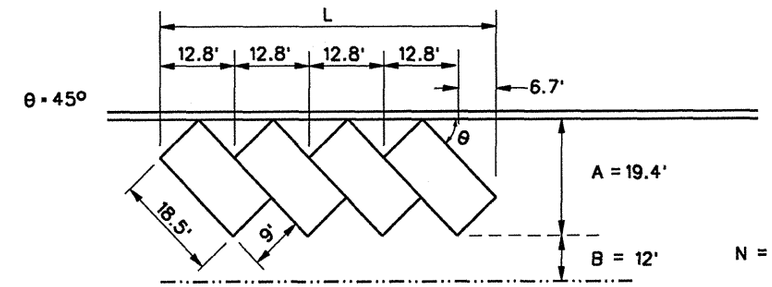
2. Configuration. The two basic types of on-street parking are parallel and angle parking. These are illustrated in Figure 14-2D. Parallel parking is the preferred arrangement where street space is limited and traffic capacity is a major factor. Angle parking provides more spaces per lineal foot than parallel parking, although a greater cross street width is necessary for its design. The total entrance and exit time for parallel parking exceeds that required for angle parking. Parallel parking also requires a vehicle to stop in the traveled way and await an opportunity to back into the parking space. However, the designer should also consider that angle parking requires the vehicle to back into the lane of travel when sight distance may be restricted by adjacent parked vehicles and when this maneuver may surprise an approaching motorist.



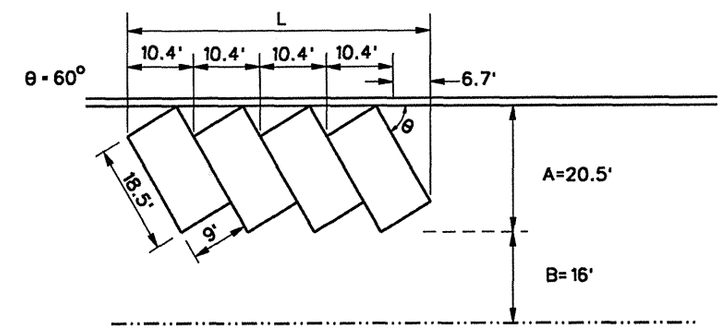
$$N = \frac{L}{22\ (\text{TO}\ 26)}$$



$$N = \frac{L - 2.5}{18.5}$$



$$N = \frac{L - 6.7}{12.8}$$



$$N = \frac{L - 6.7}{10.4}$$

Key: L = given curb length with parking spaces (ft)
N = number of parking spaces over distance L
A = required distance between face of curb and back of stall, assuming that bumper of parked car does not extend beyond curb face. In restricted locations, it can be assumed that the car will move forward until its tire contacts the curb. In these cases, the "A" distances in the figure may be reduced as follows:

Angle of Parking	Reduction in "A"
30°	1.2′
45°	1.8′
60°	2.2′

B = minimum clear distance needed for a parked vehicle to back out of stall while just clearing adjacent parked vehicles.

CURB PARKING CONFIGURATIONS
Figure 14-2D

When selecting the parking configuration, the designer should evaluate the operational consequences. In particular, the designer should consider the backing maneuver required by angle parking. As indicated in Figure 14-2D, the parked car will require a certain distance "B" to back out of its stall. Whether or not this is a safe maneuver will depend upon the number of lanes in each direction, lane widths, operating speeds, traffic volumes during peak hours, the parking demand and turnover rate of parked vehicles.

3. Stall Dimensions. Figure 14-2D provides the width and length criteria for parking stalls for various configurations. The Figure also indicates the number of stalls which can be provided for each type for a given curb length.

   The designer should also consider the effects of parking stalls on through traffic in adjacent travel lanes. Drivers will "shy" away from parked vehicles as they move along an urban street. For parallel parking, this characteristic is reflected in the parking lane widths provided in Tables 14-2G through 14-2I. For angle parking, the travel lane widths in Tables 14-2G through 14-2I should be increased by 2 ft to account for the "shy" distance.

   Note that the width of gutter pans, if provided, may be considered part of the parking lane width.

4. Location. Parking spaces should be located as follows:

   a. Parking should be prohibited within 20 ft to 30 ft of the beginning (PC) and end (PT) of the curb radius at intersections. This is especially important where the parking width is necessary to allow trucks to turn at intersections and not encroach into opposing lanes of traffic. See Section 6-2.0.

   b. Parking should be prohibited within 10 ft to 15 ft of the beginning of the curb radius at mid-block driveway entrances.

   c. Parking should be prohibited within 50 ft of the nearest rail of a railroad/ highway crossing.

   d. Parking should be prohibited from areas designated by local traffic and enforcement regulations (e.g., near fire hydrants, loading zones, bus stops).

14-2.06.04 Medians

A median may be considered on many multilane urban arterials. The principal functions of a median are to provide a separation from opposing traffic, to prevent undesirable turning movements, to provide pedestrian refuge and to provide width for future lanes. Medians on urban roads and streets may be raised, flush or depressed. The designer should refer to the AASHTO *A Policy on Geometric Design of Highways and Streets* for more information on medians.

14-2.06.05 Curb and Gutter Usage

Curbs are often provided on urban streets to control drainage, delineate the pavement edge, channelize vehicular movements, provide separation between vehicles and pedestrians, control access and roadside development, provide lateral support to the pavement and present an attractive appearance. The decision to use a curb should be based on an evaluation of these benefits combined with its installation costs, right-of-way impacts, highway capacity impacts, and any problems introduced by the surrounding terrain. In addition, curbs should not be used in front of guardrail or median barriers, if practical. See Chapter 9.

The two general classes of curbs are barrier curbs and mountable curbs. Each has distinct features and design details. Both curb types are typically designed with a gutter pan to form a combination curb and gutter section. Details of curbs are provided in the *Standard Roadway Design Drawings.* The following applies:

1. Barrier Curbs. Barrier curbs have a relatively high and steep face and are intended to discourage vehicles from leaving the roadway. They range from 6 in. to 8 in. in height and have a sloping face with a minimum 1 horizontal to 3 vertical rate. Type 1 curb in the *Standard Drawings* is considered a barrier curb. Type 1 curb is preferred where a sidewalk is used. Barrier curbs should not be used where subject to being impacted at speeds greater than 40 mph. Barrier curbs do not prevent a vehicle from leaving the roadway, and high-speed impacts may result in undesirable vehicular behavior.

2. Mountable Curbs. Mountable curbs are used where the design speed exceeds 40 mph. These locations may include pavement edges adjacent to a grass buffer strip or median, the nose of a gore area or the edges of channelized islands. Mountable curbs are usually 2 in. to 6 in. in height with rounded or plane sloping faces. The Types 2 and 3 curbs in the *Standard Drawings* are considered mountable. The lower face of the mountable curb may have a vertical section to allow for future resurfacing. This vertical portion should not exceed 2 in.

3. Curb Placement. The placement of curbing with respect to the traveled way is an important design consideration. Sections 14-2.06.01 and 14-2.06.02 discuss additional information regarding the presence of curb and gutter sections relative to travel lane and shoulder width. The designer should review these discussions when determining the placement of the curb and gutter. The beginning or nose of a curbed section along the roadway should be offset 2 ft to 4 ft or more from the traveled way and a gradual transition provided between the offset and normal curb alignment.

14-2.06.06 Sidewalks

Sidewalks are considered integral parts of the urban environment. Sidewalks should be strongly considered in the design of all projects on urban streets. The following applies:

1. Warrants (Roadway). Where sidewalks currently exist, they will be replaced if affected by the project. Where they do not currently exist, the need for sidewalks will be determined on a case-by-case basis. No numerical warrants are presented. In general, the designer should consider providing sidewalks along any street where pedestrians normally move or would be expected to move if they had a sidewalk available (i.e., a latent demand exists). The typical candidate is an urban or suburban street in residential or commercial areas with a design speed of 45 mph or less. In conducting the evaluation, the designer should consider the costs of a sidewalk (curbs, grading, right-of-way, drainage, utility locations, etc.) and any other mitigating factors (e.g., environmental impacts.)

   Streets leading to schools or libraries should have sidewalks on at least one side of the street, regardless of the pedestrian demand during the remainder of the day.

2. Warrants (Bridges). If a bridge is within the limits of a new construction or reconstruction project and if its bridge deck will be rehabilitated as part of the project, sidewalks will be provided on the bridge if provided on the approach roadway. Even if not provided on the approach roadway, sidewalks may still be warranted on the rehabilitated bridge deck if judged to be necessary to protect public safety. If the bridge deck will not be rehabilitated as part of the reconstruction project, it will rarely be warranted to perform this work solely to provide sidewalks on the bridge.

3. Sidewalk Width. The width of a sidewalk should be related to the street type and frequency of its use. The minimum width is 5 ft; 8 ft is desirable. In Central Business District (CBD) areas, sidewalks are often paved from the back of the curb to the front edge of the buildings.

   The designer should also consider the impacts of roadside appurtenances within the sidewalk. These will reduce the effective width because they interfere with pedestrian activity. Preferably, these appurtenances (utility poles, traffic signs, and fire hydrants) should be placed behind the sidewalk.

4. Accessibility Criteria. Several design elements for sidewalks may be required to meet the criteria of the *Americans with Disabilities Act*. These include curb-cut ramps, clear width area and longitudinal slopes. See Section 8-2.0 for the applicable accessibility criteria.

5. Buffer Area. If the available right-of-way is sufficient, buffer areas between the curb and sidewalk are desirable but are not required. The buffer area should be at least 3 ft wide to be effective. Wider buffer areas ease the problems with the vertical transitions of driveway entrances. Buffer areas have been used for the placement of roadside appurtenances (e.g., utility poles, traffic control devices, fire hydrants). However, this is undesirable because the closer proximity to the traveled way increases the likelihood of vehicle/fixed-object accidents, and their presence in buffer areas detracts from the appearance of the highway environment.

6. Driveways. The *Standard Roadway Design Drawings* provide the treatment of sidewalks at driveways and intersections. Where the walkway is located adjacent to the curb, special treatments may be necessary at driveway approaches. A 6-ft minimum transition should be used on either side of the driveway approach to avoid abrupt changes in grade for pedestrians. See Section 14-2.07.02 for more discussion on the design of driveway entrances.

7. Sidewalk Design on Bridges. Sidewalk widths on bridges should be the same as the width approaching the bridge. Buffer areas should not be maintained across the bridge. The sidewalk should be laterally transitioned back to the pavement edge and carried across the bridge. In some cases, a flush roadway shoulder may lead up to a bridge sidewalk. Where these installations exist, the ends of the sidewalk should be ramped from the shoulder at a rate of approximately 20:1.

14-2.06.07 Bridges

Tables 14-2G through 14-2I provide the criteria for the minimum width of new and reconstructed bridges on urban streets. The structural design of these bridges will be according to the latest edition of the AASHTO publication *Standard Specifications for Highway Bridges*. The minimum design loading structural capacity for new and reconstructed bridges will be HS-20.

For a reconstruction project, there may be one or several existing bridges within the project limits. Tables 14-2G through 14-2I also provide the roadway width and structural loading capacity to evaluate the adequacy of the existing bridge(s). If the bridge is structurally sound and if it meets the design loading structural capacity, it is unlikely to be cost effective to improve the geometrics of the bridge.

If the existing bridge width is less than the width in Tables 14-2G through 14-2I and if it will remain in place, it will be necessary to obtain a design exception on all Federal-aid projects. In these cases, the signing and pavement markings must meet the criteria in the MUTCD.

14-2.06.08 Roadside Clear Zone

Adequate horizontal clearance between the traveled way and roadside obstructions should be provided on urban streets. The roadside clear zone should be free of all unyielding objects including sign supports, light poles, utility poles, trees and any other fixed objects. However, limited right-of-way often restricts the available horizontal clearance on urban streets.

Where curbs are not provided, the criteria in Tables 14-2G through 14-2I will be used to determine the necessary clear zone, which is a reference to Section 9-2.0. These are based on design speed, traffic volumes and side slopes.

Where curbs are provided, the minimum required clearance for urban streets is 1.5 ft. This value is measured from the face of the curb to the edge of the obstruction, and it is measured from the face of curb whether or not an auxiliary lane or parking lane is present. However, the designer should provide the clear zone values for streets without curbs, if practical.

14-2.06.09 Right-of-Way Width

The minimum right-of-way width for urban streets is the sum of the various cross section elements, including lane widths, median width, shoulders and/or clear zones, side slopes, any public utilities and, where provided, frontage roads. Desirably, the minimum right-of-way width will be increased to allow for future expansion.

In built-up areas, it may be necessary to consider minimum values. The right-of-way widths in developed urban areas are governed primarily by economic considerations, physical obstructions or environmental constraints. The development may vary along a given facility and, therefore, the right-of-way criteria should not be a predetermined fixed value.

14-2.07 At-Grade Intersections

14-2.07.01 Application of Chapter 6

Chapter 6 "At-Grade Intersections" presents the Department's criteria for the design of intersections. These apply in general to the design of urban streets. However, the designer should consider the following when using Chapter 6:

1. General Design Controls. Section 6-1.0 applies directly to urban street projects. In addition, consider Section 6-1.03 which discusses profile criteria for intersecting roads. It will often be necessary to warp one or both of the pavement sections to provide a smooth riding surface through the intersection. This should be done within certain constraints. The minimum length of transition for the outside edge of pavement should be based on the maximum relative longitudinal slopes used for the superelevation of horizontal curves. For low-speed urban streets ($V \leq 45$ mph), the "S" values in Table 14-2D will apply. Where $V \geq 50$ mph, the "S" values in Table 3-4A will apply.

2. Turning Radii Design. The criteria for urban intersections in Section 6-2.0 applies directly to urban street projects.

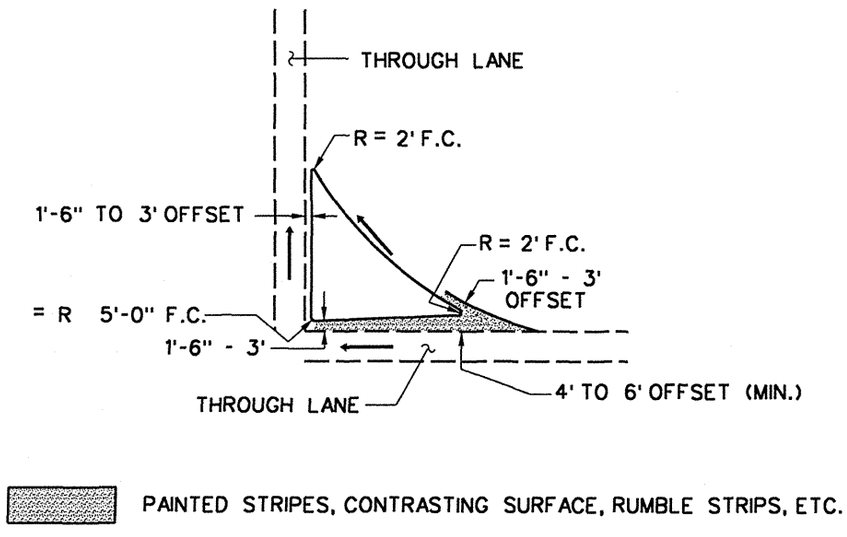
3. Turning Roadways. These will seldom be warranted on urban streets functionally classified as collector or local streets. On urban arterials, turning roadways may often be desirable at intersections as described in Section 6-7.03. Where used, their design should meet the criteria in that Section.

4. Auxiliary Lanes. The criteria in Section 6-4.0 applies directly to the design of urban streets. Because of the restricted conditions encountered in urban areas, it may frequently be necessary to only provide the minimum lengths of auxiliary lanes. These are presented in Section 6-4.03.

5. Continuous Two-Way Left Turn (CTWLT) Lanes. These are most often warranted on urban arterials with a high number of driveways per block. The criteria in Section 6-5.0 applies directly to the design of a CTWLT lane on an urban street.

6. Intersection Sight Distance. The criteria for intersection sight distance in Section 6-6.0 applies directly to projects on urban streets.

7. Channelization. This will seldom be warranted on collector or local streets but may be justified at intersections on urban arterials. Where used, the designer should apply the criteria in Section 6-7.0. Figure 14-2E presents the design of triangular islands on curbed streets without shoulders.



DESIGN OF TRIANGULAR ISLANDS
(Curbed Streets — No Shoulders)

Figure 14-2E

14-2.07.02 Urban Driveways

Urban driveways should be restricted to locations where vehicular movements into and out of them will minimize disruption to the flow of traffic on the major street. Driveway location and design are related to the functional classification of the street and the volume and type of traffic using the facility.

Table 14-2E and Figure 14-2F present design criteria for urban driveways. The *Standard Roadway Design Drawings* present other design details. In addition, several Department SOP's provide information related to driveways:

1. SOP No. MND-03-00-00-000 "Processing of Permit Applications."

2. SOP No. MND-03-03-01-000 "Driveway Connections."

3. SOP No. MND-03-04-01-000 "Construction and Maintenance of Driveway, County Road and Municipal Street Connections to State Highways."

4. SOP No. MND-04-03-01-000 "Driveway and Street Connections, Median Openings, Frontage Roads."

For convenience, the following reproduces selected information from SOP No. MND-04-03-01-000 which pertains to the design of urban driveways:

1. Islands Between Closely Spaced Driveways. Islands having a minimum length of 25 ft measured parallel to the highway will be built between closely spaced driveways to form definite entrances and exits as needed to channelize traffic and prevent paving of the entire frontage, thereby eliminating parking on the highway right-of-way directly in front of businesses and immediately adjacent to the roadway.

Table 14-2E

RECOMMENDED DESIGN CRITERIA FOR URBAN DRIVEWAYS

Driveway Design Element		Driveway Type		Functional Classification		
				Arterial	Collector	Local Road
Corner Spacing (C)		All		(1)	(1)	(1)
Driveway Spacing (D)		All		25′	25′	25′
Turning Radii (R)		Residential		3′ - 10′	3′ - 10′	3′ - 10′
		Commercial/Industrial		5′ - 30′	5′ - 30′	5′ - 30′
Width (W) (2)		Residential		16′	16′	16′
		Commercial/Industrial		30′ to 50′	30′ to 50′	30′ to 50′
Maximum Grades	G_1	All	Without Curbs	Cut Section: 2% - 4% (down); Fill Section: Shoulder Slope (4% Typical) (down)		
			With Curbs	10% or Flatter		
	G_2	All		15% (3)		
Change in Grade without Vertical Curve (ΔG)	At Driveway Entrance with Curbs	All		Maximum: 21%	Maximum: 21%	Maximum: 21%
	All Other Locations			Maximum: 12%	Maximum: 15%	Maximum: 15%
Driveway Side Slopes (without Curbs)		All		Maximum: 3:1	Maximum: 3:1	Maximum: 3:1
Sight Distance		All		(4)	(4)	(4)

Note: See Figure 14-2F for a definition of terms.

URBAN DRIVEWAYS

Footnotes to Table 14-2E

1. Corner Spacing (C). No part of a driveway entrance or exit is permitted within the turning radius of an intersection (see Section 6-2.0) or within the sight triangles (flares) of an intersection (see Section 6-6.0). If practical, greater distances for corner spacing should be provided.

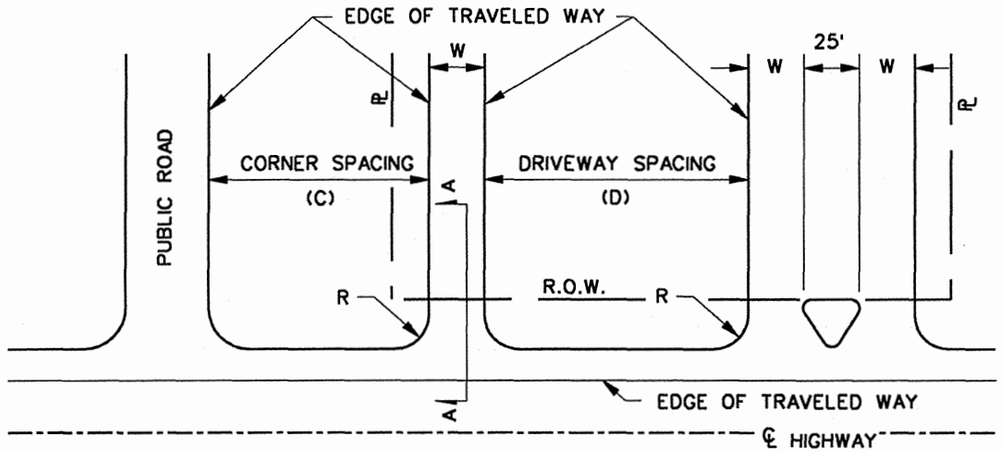
2. Widths. The following apply:

   a. These widths apply to the total, all-weather surface width available.

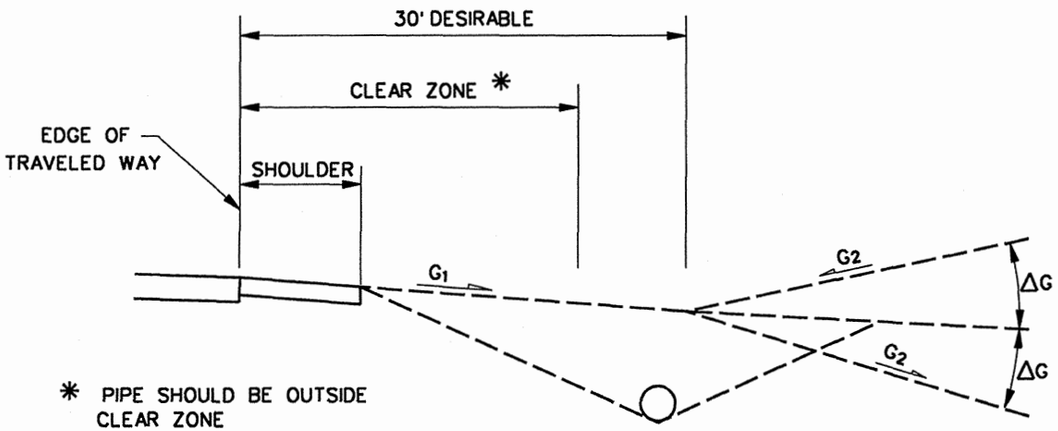
   b. These widths are sufficient for one-way operation. For two-way operation, the minimum width for multi-unit residential driveways should be 30 ft. For commercial/industrial driveways with two-way operation, widths should be provided based on the likelihood that vehicles will be entering and exiting from the driveway simultaneously.

3. Maximum Grades (G_2). In restricted locations, G_2 may be as steep as 20%.

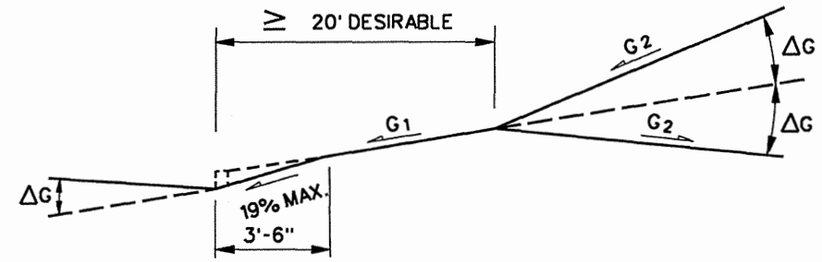
4. Sight Distance. Because of the typical low volumes at driveways, it is not warranted to explore extraordinary measures to improve sight distance at driveways. However, the designer should check for sight obstructions in the vicinity of the driveway entrance (e.g., large trees, hedgerows) which may cause a problem. To perform the check, it is reasonable to assume an eye location of approximately 10 ft from the edge of traveled way.



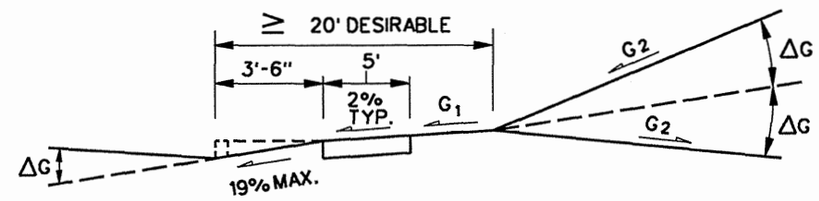
PLAN VIEW



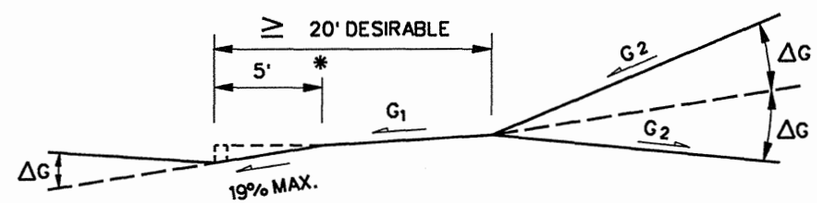
SECTION A-A
PROFILE WITH NO CURB



SECTION A-A
PROFILE WITH CURB AND NO SIDEWALK



SECTION A-A
PROFILE WITH CURB, SIDEWALK AND BUFFER STRIP



*FOR THIS DESIGN, IT WILL BE NECESSARY TO WARP THE SIDEWALK TO MEET THE SLOPE OF THE DRIVEWAY ENTRANCE.

SECTION A-A
PROFILE WITH CURB, SIDEWALK AND NO BUFFER STRIP

TYPICAL URBAN DRIVEWAYS

Figure 14-2F

2. Number of Entrances/Exits Based on Frontage. No more than two combined driveway entrances and exits shall be permitted for any parcel of property which has a frontage of 200 ft or less. Additional entrances or exits for parcels having a frontage in excess of 200 ft shall be permitted only after a showing of actual inconvenience and necessity. Where frontage is 50 ft or less, only one combined entrance and exit is permitted, the width of which shall not exceed 25 ft measured parallel to the centerline of the highway.

3. Location With Respect to Frontage. All driveways shall be so located within the applicant's frontage that the flared portion or return adjacent to the traveled way will not encroach on adjacent frontage. A minimum distance of 12.5 ft should be reserved between driveway and boundary or property line so as to preserve a 25 ft neutral area between driveways.

4. Vehicular Backing. Driveways at commercial establishments shall be built so that vehicles will not have to back into the roadway.

5. Entrances to Industrial/Commercial Buildings. Figure 14-2G illustrates a typical plan view for this treatment.

14-2.08 Geometric Design Criteria

This Section presents the Department's criteria for the design of urban streets on new and reconstruction projects. Geometric design criteria based on design speed are provided in Table 14-2F. Figures 14-2H and 14-2I illustrate the typical cross section elements for an urban street with or without curbs. Design criteria for cross section and alignment elements are presented in Table 14-2G (arterial streets), Table 14-2H (collector streets) and Table 14-2I (local streets).

14-2.09 Roadside Safety

Chapter 9 presents the Department's roadside safety criteria. These criteria apply, as practical, to new construction/reconstruction projects on urban streets.

14-2.10 Exceptions to Geometric Design Criteria

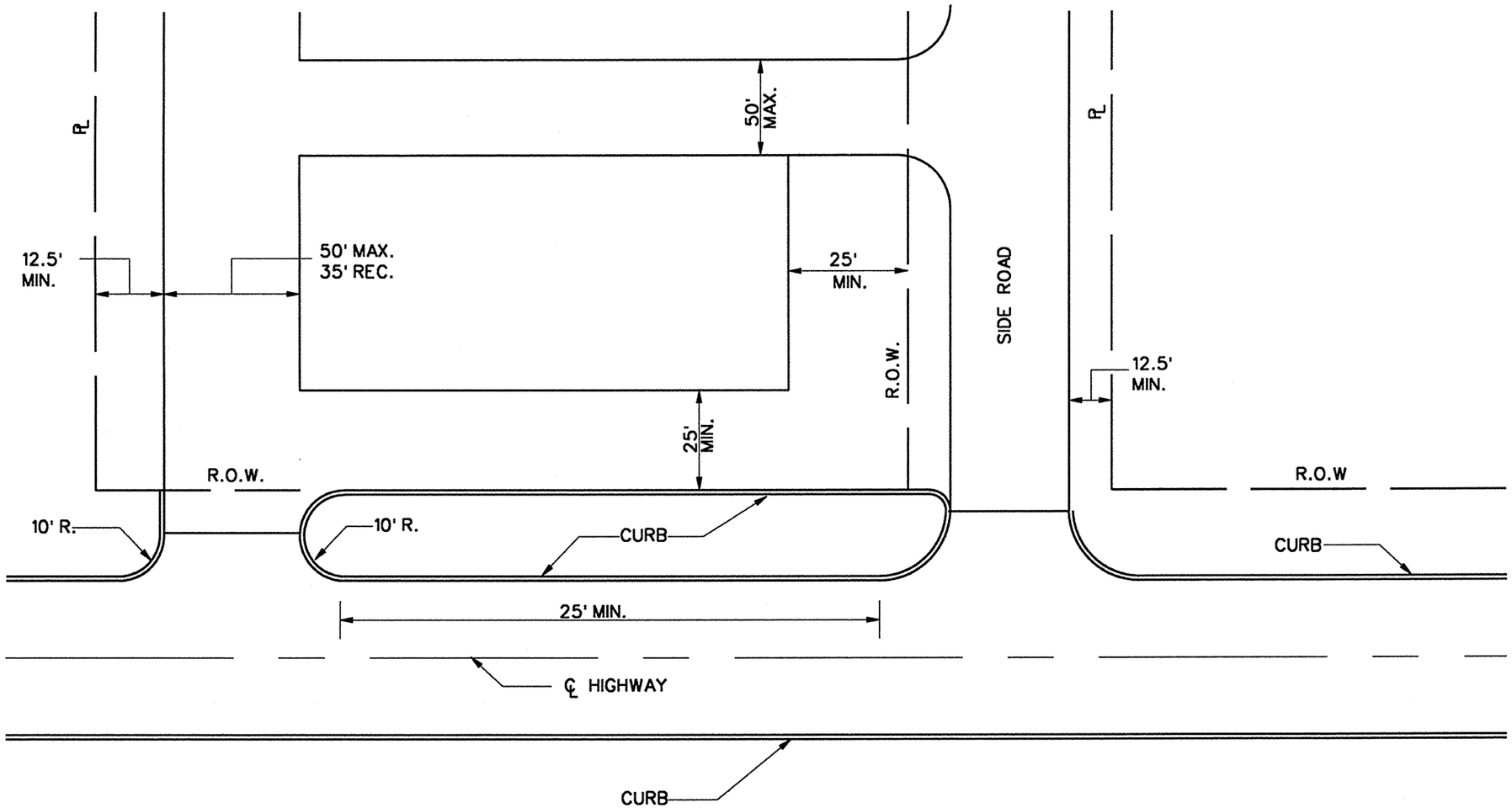
Designers of urban street projects are responsible for meeting or exceeding all design criteria in the *Manual*. In addition, the designer should make a reasonable effort to provide a design which equals or exceeds the minimum or maximum value. The Department has established the controlling criteria for all Federal-aid projects. These design elements and the procedures for exceptions are presented in Section 2-8.0 for new construction and reconstruction. This process also applies to all urban projects on State highways and to all Federal-aid projects. The formal design exception process will not apply to non-Federal-aid projects on municipal streets.

Table 14-2F

GEOMETRIC DESIGN CRITERIA BASED ON DESIGN SPEED (Urban Streets)

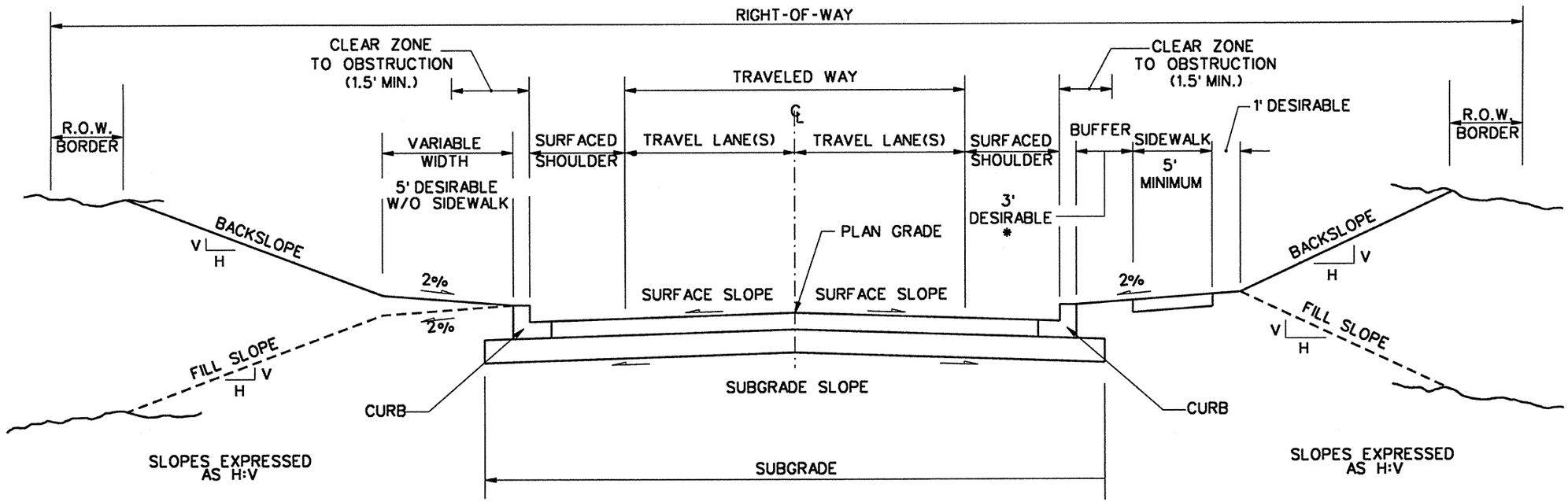
Design Element	Design Speed (mph)								
	20	25	30	35	40	45	50	55	60
Maximum Degree of Curvature	77°00′	42°45′	26°45′	18°00′	12°45′	9°30′	6°45′	6°30′	5°15′
Stopping Sight Distance (ft)	110′	150′	185′	225′	275′	325′	380′	435′	505′
	← e_{max} = 6% →						← e_{max} =10% →		

(1) *For design speeds of 45 mph and below, e_{max} = 6%. See Section 14-2.04 for a detailed discussion on horizontal curvature on low-speed urban streets. For design speed of 50 mph, e_{max} = 6%. See Table 14-2B. For design speeds greater than 50 mph, e_{max} = 10%. See Table 3-4A.*



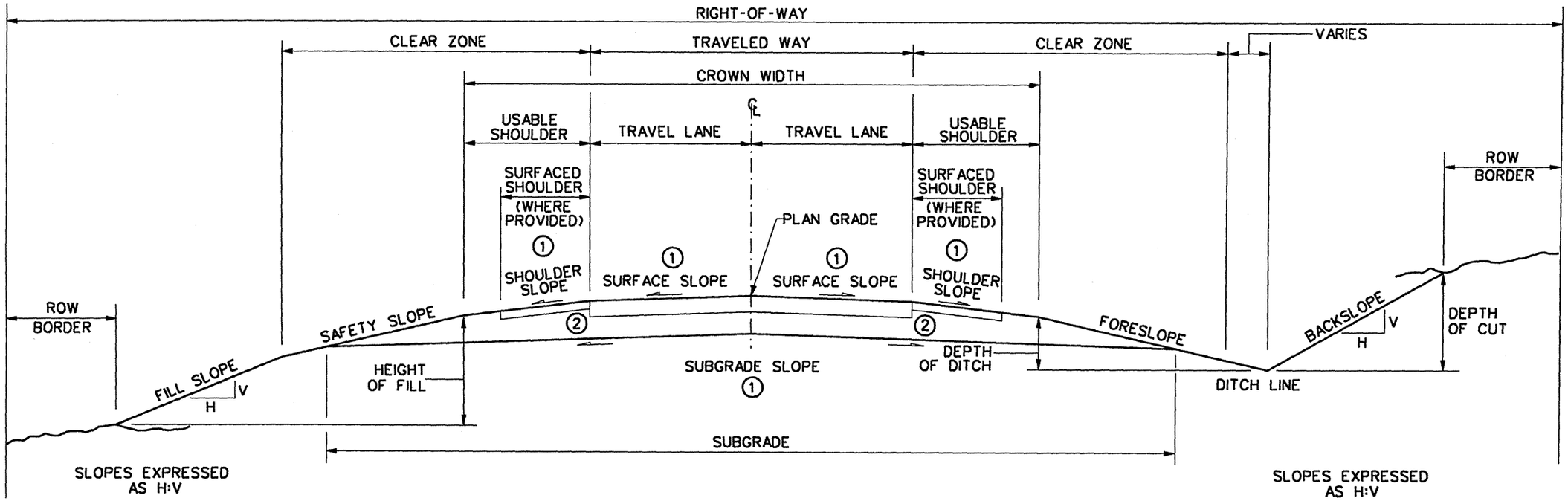
PLAN VIEW OF TYPICAL ENTRANCES
(Industrial/Commercial Buildings)

Figure 14-2G



*MUST PROVIDE ADEQUATE ROOM FOR CURB-CUT.

TYPICAL SECTION NOMENCLATURE
(Urban Streets With Curbs)
Figure 14-2H



Notes:

① *Typical cross slopes:*
Concrete and Bituminous = 2%
Shoulders = 4%
Subgrade = 2%-3%

② *For graded shoulders, extend pavement base course 1 ft beyond the edge of the surfaced traveled way.*

TYPICAL SECTION NOMENCLATURE
(Urban Streets Without Curbs)
Figure 14-2I

Table 14-2G
GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS
(New Construction/Reconstruction)

	DESIGN ELEMENT		Manual Section	2-Lane		Multilane	
				With Curb	Without Curb	With Curb	Without Curb
Design Controls	Design Forecast Year		14-2.01	20 Years		20 Years	
	✻Design Speed (1)		14-2.02	40 mph - 50 mph	45 mph - 55 mph	45 mph - 55 mph	45 mph - 60 mph
	Control of Access		2-4.0	Control by Regulation (Type 3)		Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)	
	Level of Service		14-2.01	Desirable: B Minimum: C		Desirable: B Minimum: C	
Cross Section Elements	✻Lane Width		14-2.06	12′	Desirable: 12′ Minimum: 11′ (2)	12′	
	Outside Shoulder Width	✻Usable	14-2.06	Desirable: 8′ Minimum: 6′	8′	Desirable: 10′ Minimum: 6′	10′
		Surfaced		Same as Usable	(3)	Same as Usable	8′
	Median Shoulder Width	✻Usable	14-2.06	N/A		Desirable: 4′ Minimum: 2′	8′
		Surfaced		N/A		Same as Usable	3′
	✻ Cross Slope	Travel Lane	14-2.06	2%		2%	
		Shoulder		Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%	Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%
	Auxiliary Lanes	Lane Width	14-2.06	Same as Travel Lane		Same as Travel Lane	
		Usable Shoulder Width		Desirable: 4′ Minimum: 2′	Same as Shoulder Mainline	Desirable: 4′ Minimum: 2′	Same as Shoulder Mainline
	CTWLT Lane Width		14-2.06	Desirable: 14′ Minimum: 12′		Desirable: 14′ Minimum: 12′	
	Parking Lane Width (4)		14-2.06	Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
	Sidewalk Width		14-2.06	5′ - 8′		5′ - 8′	
	Median Width		14-2.06	N/A		Depressed: 64′	
	✻New and Reconstructed Bridges	Structural Capacity	14-2.06; 11-4.0	HS-20		HS-20	
		Minimum Width (5)		Approach Roadway Width		Approach Roadway Width	
	✻Existing Bridges to Remain in Place	Structural Capacity	14-2.06; 11-4.0	HS-20		HS-20	
		Minimum Width (5)		Traveled Way + 2′ (each side)		Traveled Way + 2′ (each side)	
	Minimum Right-of-Way Width		14-2.06; 10-1.02	(6)		(6)	
	Roadside Clear Zone	Guardrail	9-4.03; 9-1.0	Usable Shoulder Width		Usable Shoulder Width	
		✻Obstruction (7a)		1.5′ (7b)	(7c)	1.5′ (7b)	(7c)
	Slope Schedule — Cut	Foreslope	14-2.06	+2%	4:1	+2%	4:1
		Depth of Ditch		N/A	1′	N/A	4′
		Backslope		Desirable: 3:1 Maximum: 2:1	3:1	Desirable: 3:1 Maximum: 2:1	3:1
	Slope Schedule — Fill	Height ≤ 10′		Desirable: 3:1 Maximum: 2:1	4:1	Desirable: 3:1 Maximum: 2:1	4:1
		Height > 10′			4:1 (within clear zone) 3:1 (outside clear zone)		4:1 (within clear zone) 3:1 (outside clear zone)

	DESIGN SPEED			40 mph	45 mph	50 mph	55 mph	60 mph
Alignment Elements	✻Minimum Stopping Sight Distance		14-2.03	275′	325′	380′	435′	505′
	✻Maximum Degree of Curvature		14-2.04	12°45′ (e = 6.0%)	9°30′ (e = 6.0%)	6°45′ (e = 6.0%)	6°30′ (e = 10.0%)	5°15′ (e = 10.0%)
	✻Superelevation Rate		14-2.04	See Figure 14-2A (e_{max} = 6%)		See Table 14-2B (e_{max} = 6%)	See Table 3-4A (e_{max} = 10%)	
	✻Horizontal Sight Distance		3-5.0	(8)				
	✻Maximum Grades	Level	14-2.05	7%	6.5%	6%	5.5%	5%
		Rolling		8%	7.5%	7%	6.5%	6%
	Minimum Grades		14-2.05	Desirable: 0.4% Minimum: 0.2% (With Curbs) 0.0% (Without Curbs)				
	✻Vertical Curve (K-values)		14-2.05	Crest: 57 Sag: 56	Crest: 80 Sag: 69	Crest: 109 Sag: 84	Crest: 143 Sag: 99	Crest: 192 Sag: 118
	✻Vertical Clearance (Arterial Under) (9)	New/Reconstructed Bridges	2-6.03	16.5′				
		Existing Bridges		14′-6″				
		Sign Truss		17.5′				
	Vertical Clearance (Arterial Over Railroad)		2-6.03	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS
(New Construction/Reconstruction)

Footnotes to Table 14-2G

1. Design Speed. A design speed of 30 mph may be used in restricted built-up areas.

2 Lane Width — Without Curb. On 2-lane facilities, use the minimum value of 11 ft only in areas where right-of-way, environmental or other constraints prohibit the use of 12 ft lanes.

3. Shoulder Surface Type (2-Lane Facilities). Generally, these shoulders will not be surfaced except as approved in special conditions.

4. Parking Lanes. These widths only apply to parallel parking. See Section 14-2.06 for the design of angle parking.

5. Bridge Width. Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Right-of-Way Width. The right-of-way limits will be determined by build-up, property values, etc.

7. Roadside Clear Zones. The following will apply:

   a. Controlling Design Criteria. The 1.5-ft horizontal clearance to obstructions on curbed facilities is a controlling design criteria. The clear zone criteria on uncurbed facilities is not a controlling design criteria.

   b With Curb. The 1.5 ft is measured from the face of the curb to the obstruction regardless of shoulder width.

   c. Without Curb. The recommended clear zones are based on the design speed, side slopes and traffic volumes. See Section 9-2.0. All values are measured from the edge of traveled way or auxiliary lane, if present.

8. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and degree of curvature. See Section 3-5.0 for the applicable criteria.

9. Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

Table 14-2H
GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR STREETS
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	2-Lane		Multilane	
					With Curb	Without Curb	With Curb	Without Curb
Design Controls	Design Forecast Year			14-2.01	20 Years		20 Years	
	✻Design Speed			14-2.02	30 mph - 50 mph	30 mph - 50 mph	30 mph - 50 mph	30 mph - 50 mph
	Control of Access			2-4.0	Control by Regulation (Type 3)		Control by Regulation (Type 3)	
	Level of Service			14-2.01	Desirable: B Minimum: C		Desirable: B Minimum: C	
Cross Section Elements	✻Lane Width			14-2.06	12′	Desirable: 12′ Minimum: 11′ (1)	12′	
	Outside Shoulder Width	✻Usable		14-2.06	Desirable: 6′ Minimum: 4′	6′	Desirable: 10′ Minimum: 2′	10′
		Surfaced			Same as Usable	(2)	Same as Usable	8′
	Median Shoulder Width	✻Usable		14-2.06	N/A		Desirable: 4′ Minimum: 2′	8′
		Surfaced			N/A		Same as Usable	3′
	Cross Slope	✻Travel Lane		14-2.06	2%		2%	
		Shoulder			Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%	Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%
	Auxiliary Lanes	Lane Width		14-2.06	Same as Travel Lane		Same as Travel Lane	
		Usable Shoulder Width			Desirable: 4′ Minimum: 2′	Same as Shoulder Mainline	Desirable: 4′ Minimum: 2′	Same as Shoulder Mainline
	CTWLT Lane Width			14-2.06	Desirable: 14′ Minimum: 12′		Desirable: 14′ Minimum: 12′	
	Parking Lane Width (3)			14-2.06	Desirable: 10′ Minimum: 8′		Desirable: 12′ Minimum: 10′	
	Sidewalk Width			14-2.06	5′ - 8′		5′ - 8′	
	Median Width			14-2.06	N/A		Depressed: 64′	
	✻New and Reconstructed Bridges	Structural Capacity		14-2.06; 11-4.0	HS-20		HS-20	
		Minimum Width (4)			Approach Roadway Width		Approach Roadway Width	
	✻Existing Bridges to Remain in Place	Structural Capacity		14-2.06; 11-4.0	HS-20		HS-20	
		Minimum Width			(5)		Traveled Way + 2′ (each side) (6)	
	Minimum Right-of-Way Width			14-2.06; 10-1.02	(7)		(7)	
	Roadside Clear Zone	Guardrail		9-4.03; 9-1.0	Usable Shoulder Width		Usable Shoulder Width	
		✻Obstruction (8a)			1.5′ (8b)	(8c)	1.5′ (8b)	(8c)
	Slope Schedule	Cut	Foreslope	14-2.06	+2%	3:1	+2%	4:1
			Depth of Ditch		N/A	3′	N/A	4′
			Backslope		Desirable: 3:1 Maximum: 2:1	3:1	Desirable: 3:1 Maximum: 2:1	3:1
		Fill	Height ≤ 5′		Desirable: 3:1 Maximum: 2:1	4:1	Desirable: 3:1 Maximum: 2:1	4:1
			Height > 5′			3:1		4:1 (within clear zone) 3:1 (outside clear zone)

	DESIGN SPEED			30 mph	35 mph	40 mph	45 mph	50 mph
Alignment Elements	✻Minimum Stopping Sight Distance		14-2.03	185′	225′	275′	325′	380′
	✻Maximum Degree of Curvature		14-2.04	26°45′ (e = 6.0%)	18°00′ (e = 6.0%)	12°45′ (e = 6.0%)	9°30′ (e = 6.0%)	6°45′ (e = 6.0%)
	✻Superelevation Rate		14-2.04	See Figure 14-2A (e_{max} = 6%)				See Table 14-2B (e_{max} = 6%)
	✻Horizontal Sight Distance		3-5.0	(9)				
	✻Maximum Grades	Level	14-2.05	9%	9%	9%	8%	7%
		Rolling		11%	10.5%	10%	9%	8%
	Minimum Grades		14-2.05	Desirable: 0.4% Minimum: 0.2% (With Curbs) 0.0% (Without Curbs)				
	✻Vertical Curve (K-values)		14-2.05	Crest: 26 Sag: 33	Crest: 39 Sag: 43	Crest: 57 Sag: 56	Crest: 80 Sag: 69	Crest: 109 Sag: 84
	✻Vertical Clearance (Collector Under)	New/Reconstructed Bridges (10)	2-6.03	16.5′				
		Existing Bridges		14′-6″				
	Vertical Clearance (Collector Over Railroad)		2-6.03	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR STREETS
(New Construction/Reconstruction)

Footnotes to Table 14-2H

2. Lane Width — Without Curb. On 2-lane facilities, use the minimum value (11 ft) only in areas where right-of-way, environmental or other constraints prohibit the use of 12 ft lanes.

3. Shoulder Surface Type (2-Lane Facilities). Generally, these shoulders will not be surfaced except as approved in special conditions.

3. Parking Lanes. These widths only apply to parallel parking. See Section 14-2.06 for the design of angle parking.

4. Bridge Width (New/Reconstructed Bridges). Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

5. Bridge Width (Existing Bridges to Remain in Place on 2-Lane Facilities). The following will apply to widths for bridges to remain in place within the limits of a reconstruction project:

ADT	Width
< 1500	22 ft
1500 - 2000	24 ft
> 2000	28 ft

Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Bridge Width (Existing Bridges to Remain in Place on Multilane Facilities). Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

7. Right-of-Way Width. The right-of-way limits will be determined by build-up, property values, etc.

8. Roadside Clear Zones. The following will apply:

   a. Controlling Design Criteria. The 1.5-ft horizontal clearance to obstructions on curbed facilities is a controlling design criteria. The clear zone criteria on uncurbed facilities is not a controlling design criteria.

   b. With Curb. The 1.5 ft is measured from the face of the curb to the obstruction regardless of shoulder width.

   c. Without Curb. The recommended clear zones are based on the design speed, side slopes and traffic volumes. See Section 9-2.0. All values are measured from the edge of traveled way or auxiliary lane, if present.

9. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

10. Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

Table 14-2I
GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREETS
(New Construction/Reconstruction)

	DESIGN ELEMENT			Manual Section	With Curb		Without Curb	
Design Controls	Design Forecast Year			14-2.01	20 Years			
	✻Design Speed			14-2.02	20 mph - 30 mph		20 mph - 30 mph	
	Control of Access			2-4.0	Control by Regulation (Type 3)			
	Level of Service			14-2.01	Desirable: B Minimum: D			
Cross Section Elements	✻Lane Width (1)			14-2.06	11′		Desirable: 11′ Minimum: 10′	
	Outside Shoulder Width	✻Usable		14-2.06	Desirable: 4′ Minimum: 2′		4′	
		Surfaced			Same as Usable		(2)	
	Cross Slope	✻Travel Lane		14-2.06	2%			
		Shoulder			Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%		4%	
	Auxiliary Lanes	Lane Width		14-2.06	Desirable: Same as Travel Lane Minimum: 10′			
		Usable Shoulder Width			Desirable: 4′ Minimum: 1′		2′	
	Parking Lane Width (3)			14-2.06	Desirable: 9′ Minimum: 7′			
	Sidewalk Width			14-2.06	5′ - 8′			
	✻New and Reconstructed Bridges	Structural Capacity		14-2.06; 11-4.0	HS-20			
		Minimum Width			Approach Roadway Width		(4)	
	✻Existing Bridges to Remain in Place	Structural Capacity		14-2.06; 11-4.0	HS-15			
		Minimum Width (5)			22′			
	Minimum Right-of-Way Width			14-2.06; 10-1.02	(6)			
	Roadside Clear Zone	Guardrail		9-4.03; 9-1.0	Usable Shoulder Width			
		✻Obstruction (7a)			1.5′ (7b)		(7c)	
	Slope Schedule	Cut	Foreslope	14-2.06	+2%		3:1	
			Depth of Ditch		N/A		3′	
			Backslope		Desirable: 3:1 Maximum: 2:1		Desirable: 3:1 Maximum: 2:1	
		Fill	Height ≤ 5′		Desirable: 3:1 Maximum: 2:1		Desirable: 4:1 Maximum: 3:1	
			Height > 5′				3:1	
Alignment Elements	DESIGN SPEED				20 mph	25 mph		30 mph
	✻Minimum Stopping Sight Distance			14-2.03	110′	150′		185′
	✻Maximum Degree of Curvature			14-2.04	77°00′ (e = 6.0%)	42°45′ (e = 6.0%)		26°45′ (e = 6.0%)
	✻Superelevation Rate			14-2.04	See Figure 14-2A (e_{max} = 6%)			
	✻Horizontal Sight Distance			3-5.0	(8)			
	✻Maximum Grades	Residential		14-2.05	Desirable: 10% Maximum: 15%			
		Commercial/Industrial			Desirable: 5% Maximum: 8%			
	Minimum Grades			14-2.05	Desirable: 0.4% Minimum: 0.2% (With Curbs) 0.0% (Without Curbs)			
	✻Vertical Curve (K-values)			14-2.05	Crest: 10 Sag: 16	Crest: 17 Sag: 25		Crest: 26 Sag: 33
	✻Vertical Clearance (Local Under) (9)	New/Reconstructed Bridges		2-6.03	14′-6″			
		Existing Bridges			14′			
	Vertical Clearance (Local Over Railroad)			2-6.03	23′			

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREETS
(New Construction/Reconstruction)

Footnotes to Table 14-2I

1. Lane Width. 12-ft lane widths are desirable for industrial routes.

2. Shoulder Surface Type. Generally, these shoulders will not be surfaced except as approved in special conditions.

3. Parking Lanes. These widths only apply to parallel parking. See Section 14-2.06 for the design of angle parking.

4. Bridge Width (New/Reconstructed Bridges, Facilities without Curbs). The following will apply to bridges within the limits of new/reconstruction projects:

ADT	Width
< 400	Traveled Way + 2 ft (each side)
400 - 2000	Traveled Way + 3 ft (each side)
> 2000	Approach Roadway Width

Bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

5. Bridge Width (Existing Bridges to Remain in Place). Bridges which are considered major structures (i.e., lengths greater than 100 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Right-of-Way Width. The right-of-way limits will be determined by build-up, property values, etc.

7. Roadside Clear Zones. The following will apply:

   a. Controlling Design Criteria. The 1.5-ft horizontal clearance to obstructions on curbed facilities is a controlling design criteria. The clear zone criteria on uncurbed facilities is not a controlling design criteria.

   b. With Curb. The 1.5 ft is measured from the face of the curb to the obstruction regardless of shoulder width.

   c. Without Curb. The recommended clear zones are based on the design speed, side slopes and traffic volumes. See Section 9-2.0. All values are measured from the edge of traveled way or auxiliary lane, if present.

8. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

9. Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

14-3.0 URBAN STREETS (3R Projects)

The criteria in this Section applies to 3R projects on urban streets. The format is based on Section 11-2.0, "3R Projects (Non-Freeways)." The designer should refer to that Section for more discussion on the various design elements. In addition, much of the design criteria presented in Section 14-2.0, "Urban Streets (New Construction/ Reconstruction)" and in other chapters of the *Design Manual* will apply to 3R projects on urban streets. Unless stated otherwise, the criteria for the new construction/ reconstruction of urban streets will apply to 3R projects.

14-3.01 3R Project Evaluation

Section 11-2.01 presents several factors which should be evaluated in the design of a 3R project (e.g., accident records). These apply to the evaluation of 3R projects on urban streets.

14-3.02 Design Controls

The following will apply:

1. Design Speed. Tables 14-3A and 14-3B present the criteria for design speed on 3R projects. In addition, the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility.

2. Design Volume. Desirably, all elements of the facility will meet the level-of-service (LOS) criteria for an ADT or DHV determined for 10 years beyond the expected construction completion date. At a minimum, the highway facility should accommodate the LOS based on current traffic volumes as determined at the construction completion date.

14-3.03 Cross Section Elements

Tables 14-3A and 14-3B present the Department's criteria for cross section elements on 3R projects for urban streets. The cross section widths of the existing street should be evaluated against the criteria in the 3R tables. The Department's policy on the application of the cross section widths in the tables is as follows:

1. Existing Width Does Not Meet the 3R Width. The designer should consider widening the element. If the decision is made to widen the lane or shoulder width, the designer should provide a width which at least meets the 3R criteria. This will be sufficient for the majority of 3R projects. However, if practical, it may be appropriate to widen the roadway width to meet new construction/ reconstruction criteria.

2. Existing Width Does Meet the 3R Width. In most cases, it will not be cost effective to widen the roadway. However, if practical, it may be appropriate to widen the width to meet new construction/reconstruction criteria.

Section 14-2.06 provides an elaboration on the cross section elements for urban streets for new construction/reconstruction projects (e.g., how to measure lane and shoulder width where a gutter pan is provided). This discussion also applies to 3R projects. In addition, Section 11-2.03 provides an elaboration on the application of several cross section elements to 3R projects (e.g., bridge width). This discussion also applies to urban streets.

14-3.04 Horizontal Alignment

Section 14-2.04 presents the Department's criteria for the horizontal alignment on urban streets on new construction/reconstruction projects. Section 11-2.04 presents the Department's horizontal alignment criteria for 3R (rural) projects. The following will apply to the horizontal alignment of urban streets on 3R projects:

1. V ≥ 50 mph. Figure 11-2A will be used to determine the design speed of an existing horizontal curve. This may require field measurements to attain the as-built information. The design speed of the existing curve should be compared to the design speed of the 3R project as determined from Tables 14-3A and 14-3B. Figure 11-2A can only be used to decide if corrective action should be considered. Once the decision has been made to improve the curve, the designer should use the criteria in Table 14-2B (V = 50 mph) or in Table 3-4A (V ≥ 55 mph) to determine the proper combination of degree of curvature and superelevation to meet the 3R design speed. This may be achieved by either flattening the curve or by increasing the superelevation rate on the curve. Figure 11-2A can also be used to determine the appropriate advisory speed plate for signing curves which are below the 3R design speed but will remain in place.

2. V ≤ 45 mph. Figure 14-2A will be used to determine the design speed of an existing horizontal curve on a low-speed urban street.

14-3.05 Vertical Alignment

Section 11-2.05 presents the Department's vertical alignment criteria for 3R projects. This includes revised criteria for grades, crest vertical curves and sag vertical curves. These criteria apply to 3R projects on urban streets, and they have been incorporated into Tables 14-3A and 14-3B.

14-3.06 At-Grade Intersections

The design criteria in Chapter 6 will apply to 3R projects on urban streets, except as noted in the following subsection(s).

14-3.06.01 Turning Radii Design

Section 6-2.0 presents criteria for the selection of a design vehicle, for tolerable encroachment, and for turning radii criteria at intersections. Where practical, these criteria should be met on 3R urban street projects. However, in urban areas space limitations and existing curb radii have a significant impact on selecting a practical design for right-turning vehicles. The designer should consider the following when determining the appropriate right-turn treatment for urban intersections on 3R projects:

1. Simple radii of 15 ft to 25 ft are adequate for passenger vehicles. These radii may be retained on 3R projects on existing urban streets at:

   a. intersections with minor roads where very few trucks will be turning;

   b. intersections where the encroachment of SU and semitrailer vehicles onto adjacent lanes is tolerable; and

   c. intersections where a parking lane is present, and parking is restricted a sufficient distance from the intersection, and the lane is used as a parking lane throughout the day.

2. Where practical, simple radii of 30 ft or simple radii with tapers (for an SU design vehicle) should be used at all major intersections and at all minor intersections with appreciable truck turning volumes.

3. At intersections where semitrailer combinations and buses turn frequently, a simple radius of 40 ft or more should be provided. Preferably, the designer will use a radius with taper offsets for the selected design vehicle.

14-3.06.02 Auxiliary Lanes

Section 6-4.0 presents warrants for right- and left-turn lanes, and it presents design criteria for the lengths of exclusive turn lanes. However, in urban areas these criteria

may be impractical on 3R projects because of restricted conditions. In these cases, the designer will provide the best design practical for the existing field conditions.

14-3.06.03 Intersection Sight Distance

Section 6-6.0 presents the intersection sight distance (ISD) criteria for new construction/ reconstruction projects. These also apply to ISD on 3R projects with the following revisions:

1. Location of Eye. At stop-controlled intersections, the location of the eye is 15 ft from the edge of traveled way on new construction/reconstruction projects. For 3R projects on low-speed urban streets ($V \leq 45$ mph), the location of the eye may be assumed to be 10 ft from the edge of traveled way.

2. Sight Flares. It is often not practical to meet the Department's sight flare criteria on 3R urban street projects.

14-3.07 Roadside Safety

The discussion and criteria in Section 11-2.08 on roadside safety applies to 3R projects on urban streets.

14-3.08 3R Tables of Geometric Design Criteria

Tables 14-3A and 14-3B present the Department's 3R criteria for the geometric design of urban streets (arterials and collectors).

14-3.09 Exceptions to Geometric Design Criteria

The discussion in Section 2-8.0 on exceptions applies equally to the geometric design of 3R projects on urban streets. The only difference, obviously, is that the designer will be evaluating the proposed design against the criteria presented in Section 14-3.0.

Table 14-3A
GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS
(3R Projects)

	DESIGN ELEMENT		Manual Section	2-Lane		Multilane (1)	
				With Curb	Without Curb	With Curb	Without Curb
Design Controls	Design Forecast Year		14-3.02	Desirable: 10 Years Minimum: Current		Desirable: 10 Years Minimum: Current	
	✻Design Speed (2)		14-3.02	40 mph - 50 mph	45 mph - 55 mph	40 mph - 55 mph	45 mph - 60 mph
	Control of Access		2-4.0	Control by Regulation (Type 3)		Desirable: Partial (Type 2A or 2B) Minimum: Control by Regulation (Type 3)	
	Level of Service		14-2.01	Desirable: B Minimum: D		Desirable: B Minimum: D	
Cross Section Elements	✻Lane Width		14-3.03	NHS: 12′ Non-NHS: Desirable: 12′; Minimum: 11′		NHS: 12′ Non-NHS: Desirable: 12′; Minimum: 11′	
	Outside Shoulder Width	✻Usable	14-3.03	Desirable: 8′ Minimum: 2′	4′	Desirable: 10′ Minimum: 1′	Desirable: 10′ Minimum: 4′
		Surfaced		Same as Usable	(3)	Same as Usable	Desirable: 8′ Minimum: 2′
	Median Shoulder Width	✻Usable	14-3.03	N/A		Desirable: 4′ Minimum: 1′	Desirable: 8′ Minimum: 4′
		Surfaced		N/A		Same as Usable	Desirable: 3′ Minimum: 1′
	✻ Cross Slope	Travel Lane	14-3.03	2%		2%	
		Shoulder		Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%	Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%
	Auxiliary Lanes	Lane Width	14-3.03	Desirable: Same as Travel Lane Minimum: 11′		Desirable: Same as Travel Lane Minimum: 11′	
		Usable Shoulder Width		Desirable: 4′ Minimum: 1′	2′	Desirable: 4′ Minimum: 1′	4′
	CTWLT Lane Width		14-3.03	Desirable: 14′ Minimum: 11′		Desirable: 14′ Minimum: 11′	
	Parking Lane Width (4)		14-3.03	Desirable: 12′ Minimum: 8′		Desirable: 12′ Minimum: 8′	
	Sidewalk Width		14-2.06	5′ - 8′		5′ - 8′	
	Median Width		14-2.06	N/A		Existing	
	✻Reconstructed/ Rehabilitated Bridges (6)	Structural Capacity	11-2.03	HS-20		HS-20	
		Minimum Width (5)		ADT < 4000: Traveled Way +4′ ADT > 4000: Traveled Way +6′		ADT < 4000: Traveled Way +4′ ADT > 4000: Traveled Way +6′	
	✻Existing Bridges to Remain in Place	Structural Capacity	11-2.03	HS-20		HS-20	
		Minimum Width (6)		ADT < 2000: Traveled Way +2′ ADT >2000: Traveled Way +4′		Traveled Way + 4′ (Each Roadway)	
	Minimum Right-of-Way Width		14-2.06; 10-1.02	(7)		(7)	
	Roadside Clear Zone	Guardrail	11-2.08	Usable Shoulder Width		Usable Shoulder Width	
		✻Obstruction (8a)		1.5′ (8b)	(8c)	1.5′ (8b)	(8c)
	Slope Schedule — Cut	Foreslope	14-2.06	+2%	4:1	+2%	4:1
		Depth of Ditch		N/A	3′	N/A	4′
		Backslope		Desirable: 3:1 Maximum: 2:1	3:1	Desirable: 3:1 Maximum: 2:1	3:1
	Slope Schedule — Fill	Height ≤ 5′		Desirable: 3:1 Maximum: 2:1	Desirable: 4:1 Maximum: 3:1	Desirable: 3:1 Maximum: 2:1	Desirable: 4:1 Maximum: 3:1
		Height > 5′			3:1		3:1

	DESIGN SPEED			40 mph	45 mph	50 mph	55 mph	60 mph
Alignment Elements	✻Minimum Stopping Sight Distance		14-2.03	275′	325′	380′	435′	505′
	✻Maximum Degree of Curvature		14-3.04	See Section 14-3.04				
	✻Superelevation Rate		14-3.04	See Section 14-3.04				
	✻Horizontal Sight Distance		3-5.0	(9)				
	✻Maximum Grades	Level	11-2.05	9%	8.5%	8%	7.5%	7%
		Rolling		10%	9.5%	9%	8.5%	8%
	Minimum Grades		14-2.05	Desirable: 0.4% Minimum: 0.2% (With Curbs) 0.0% (Without Curbs)				
	✻Vertical Curve (K-values)		11-2.05	See Section 14-3.05				
	✻Vertical Clearance (Arterial Under)	New/Reconstructed Bridges (10)	2-6.03	16.5′				
		Existing Bridges		14′				
		Sign Truss (10)		17.5′				
	Vertical Clearance (Arterial Over Railroad)		2-6.03	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIALS
(3R Projects)

Footnotes to Table 14-3B

1. Multilane Facilities. The criteria in the table for 3R multilane facilities will apply to:

   a. an existing multilane facility, or
   b. an existing 2-lane facility which will become a divided highway.

   In the case of "b," the new roadway will be designed according to the Department's new construction criteria. See Table 14-2G.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Shoulder Surface Type (2-Lane Facilities). Generally, shoulders will not be surfaced except as approved in special conditions.

4. Parking Lanes. These widths only apply to parallel parking. See Section 14-2.06 for the design of angle parking.

5. Bridge Width (New Bridges). For new bridges within the limits of a 3R project, the minimum width will be the full approach roadway width according to the Department's criteria for new construction. However, bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Bridge Width (Reconstructed Bridges/Rehabilitated Bridges/Existing Bridges to Remain in Place). Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

7 Right-of-Way Width. The acquisition of significant amounts of right-of-way is typically outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. However, the designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

8. Roadside Clear Zone. The following will apply:

   a. Controlling Design Criteria. The 1.5 ft horizontal clearance to obstructions on curbed facilities is a controlling design criteria. The clear zone criteria on uncurbed facilities is not a controlling design criteria.

   b. With Curb. The 1.5 ft is measured from the face of the curb to the obstruction regardless of shoulder width.

   c. Without Curb. The recommended clear zones are based on the design speed, side slopes and traffic volumes. See Section 11-2.08. All values are measured from the edge of traveled way or auxiliary lane, if present.

9. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

10. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

Table 14-3B
GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR STREETS
(3R Projects)

	DESIGN ELEMENT			Manual Section	2-Lane		Multilane (1)	
					With Curb	Without Curb	With Curb	Without Curb
Design Controls	Design Forecast Year			14-3.02	Desirable: 10 Years Minimum: Current		Desirable: 10 Years Minimum: Current	
	✻Design Speed (2)			14-3.02	30 mph - 50 mph	30 mph - 50 mph	30 mph - 50 mph	30 mph - 50 mph
	Control of Access			2-4.0	Control by Regulation (Type 3)		Control by Regulation (Type 3)	
	Level of Service			14-2.01	Desirable: B Minimum: D		Desirable: B Minimum: D	
Cross Section Elements	✻Lane Width			14-3.03	NHS: 12′ Non-NHS: Desirable: 12′; Minimum: 11′		NHS: 12′ Non-NHS: Desirable: 12′; Minimum: 11′	
	Outside Shoulder Width	✻Usable		14-3.03	Desirable: 8′ Minimum: 1′	2′	Desirable: 10′ Minimum: 1′	Desirable: 10′ Minimum: 4′
		Surfaced			Same as Usable	(3)	Same as Usable	Desirable: 8′ Minimum: 2′
	Median Shoulder Width	✻Usable		14-3.03	N/A		Desirable: 4′ Minimum: 1′	Desirable: 8′ Minimum: 4′
		Surfaced			N/A		Same as Usable	Desirable: 3′ Minimum: 1′
	✻ Cross Slope	Travel Lane		14-3.03	2%		2%	
		Shoulder			Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%	Shoulder Width ≥ 4′: 4% Shoulder Width < 4′: 2%	4%
	Auxiliary Lanes	Lane Width		14-3.03	Desirable: Same as Travel Lane Minimum: 10′		Desirable: Same as Travel Lane Minimum: 10′	
		Usable Shoulder Width			Desirable: 4′ Minimum: 1′	2′	Desirable: 4′ Minimum: 1′	2′
	CTWLT Lane Width			14-3.03	Desirable: 14′ Minimum: 11′		Desirable: 14′ Minimum: 11′	
	Parking Lane Width (4)			14-3.03	Desirable: 12′ Minimum: 7′		Desirable: 12′ Minimum: 7′	
	Sidewalk Width			14-2.06	5′ - 8′		5′ - 8′	
	Median Width			14-2.06	N/A		Existing	
	✻New/Reconstructed/ Rehabilitated Bridges	Structural Capacity		11-2.03	HS-20		HS-20	
		Minimum Width			(5)		(5)	
	✻Existing Bridges to Remain in Place	Structural Capacity		11-2.03	HS-20		HS-20	
		Minimum Width			(6)		(6)	
	Minimum Right-of-Way Width			14-2.06; 10-1.02	(7)		(7)	
	Roadside Clear Zone	Guardrail		11-2.08	Usable Shoulder Width		Usable Shoulder Width	
		✻Obstruction (8a)			1.5′ (8b)	(8c)	1.5′ (8b)	(8c)
	Slope Schedule	Cut	Foreslope	14-2.06	+2%	4:1	+2%	4:1
			Depth of Ditch		N/A	3′	N/A	4′
			Backslope		Desirable: 3:1 Maximum: 2:1	3:1	Desirable: 3:1 Maximum: 2:1	3:1
		Fill	Height ≤ 5′		Desirable: 3:1 Maximum: 2:1	Desirable: 4:1 Maximum: 3:1	Desirable: 3:1 Maximum: 2:1	Desirable: 4:1 Maximum: 3:1
			Height > 5′			3:1		3:1
Alignment Elements	DESIGN SPEED				30 mph	35 mph	40 mph	45 mph	50 mph
	✻Minimum Stopping Sight Distance			14-2.03	185′	225′	275′	325′	380′
	✻Maximum Degree of Curvature			14-3.04	See Section 14-3.04				
	✻Superelevation Rate			14-3.04	See Section 14-3.04				
	✻Horizontal Sight Distance			3-5.0	(9)				
	✻Maximum Grades	Level		11-2.05	11%	11%	11%	10%	9%
		Rolling			13%	12.5%	12%	11%	10%
	Minimum Grades			14-2.05	Desirable: 0.4% Minimum: 0.2% (With Curbs) 0.0% (Without Curbs)				
	✻Vertical Curve (K-values)			11-2.05	See Section 14-3.05				
	✻Vertical Clearance (Collector Under)	New/Reconstructed Bridges (10)		2-6.03	16.5′				
		Existing Bridges			14′				
	Vertical Clearance (Collector Over Railroad)			2-6.03	23′				

✻Controlling design criteria. See Section 2-8.0.

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR STREETS (3R Projects)

Footnotes to Table 14-3B

1. Multilane Facilities. The criteria in the table for 3R multilane facilities will apply to:

   a. an existing multilane facility, or
   b. an existing 2-lane facility which will become a divided highway.

   In the case of "b," the new roadway will be designed according to the Department's new construction criteria. See Table 14-2G.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Shoulder Surface Type (2-Lane Facilities). Generally, shoulders will not be surfaced except as approved in special conditions.

4. Parking Lanes. These widths only apply to parallel parking. See Section 14-2.06 for the design of angle parking.

5. Bridge Width (New Bridges). For new bridges within the limits of a 3R project where the approach roadway is curbed, the minimum width will be the full approach roadway width according to the Department's criteria for new construction. For new bridges within the limits of a 3R project where the approach roadway is uncurbed, the width will be as follows:

ADT < 400:	Traveled way + 2 ft (each side)
400 ≤ ADT ≤ 1500:	Traveled way + 3 ft (each side)
1500 < ADT ≤ 2000:	Traveled way + 4 ft (each side)
ADT > 2000:	Approach roadway width

   For reconstructed/rehabilitated bridges, the following will apply:

   a. With Curb. The width of reconstructed/rehabilitated bridges will be:

ADT ≤ 2000:	Traveled way + 1 ft (each side)
ADT > 2000:	Traveled way + 3 ft (each side)

   b. Without Curb. The width of reconstructed/rehabilitated bridges will be:

ADT < 750:	Traveled way width
750 ≤ ADT ≤ 2000:	Traveled way + 1 ft (each side)
ADT > 2000:	Traveled way + 3 ft (each side)

   Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

6. Bridge Width (Existing Bridges to Remain in Place). The minimum width is the traveled way + 2 ft (curbed approach roadway) and the traveled way width (uncurbed approach roadway). Bridges which are considered major structures (i.e., lengths greater than 200 ft) are subject to individual economic, traffic and safety studies. See Section 11-4.0 for bridge projects performed as a spot improvement.

7. Right-of-Way Width. The acquisition of significant amounts of right-of-way is typically outside the scope of a 3R project. Therefore, the existing right-of-way will typically be unchanged by the 3R project. However, the designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR STREETS
(3R Projects)
(Continued)

Footnotes to Table 14-3B

8. Roadside Clear Zone. The following will apply:

   a. Controlling Design Criteria. The 1.5 ft horizontal clearance to obstructions on curbed facilities is a controlling design criteria. The clear zone criteria on uncurbed facilities is not a controlling design criteria.

   b. With Curb. The 1.5 ft is measured from the face of the curb to the obstruction regardless of shoulder width.

   c. Without Curb. The recommended clear zones are based on the design speed, side slopes and traffic volumes. See Section 11-2.08. All values are measured from the edge of traveled way or auxiliary lane, if present.

9. Horizontal Sight Distance. The necessary middle ordinate will be based on the design speed and maximum degree of curvature. See Section 3-5.0 for the applicable criteria.

10. Minimum Vertical Clearances. Table values include an additional 6-in. allowance for future resurfacing.

CHAPTER 15

CONTRACT PLAN ASSEMBLY

MISSISSIPPI DESIGN MANUAL

Chapter 15

CONTRACT PLAN ASSEMBLY

Section		Page
15-1.0	GENERAL INFORMATION	15-1
15-1.01	Content and Sequence	15-1
15-1.02	Project Identification	15-1
15-1.03	Numbering System	15-3
15-1.03.01	Working Numbers	15-3
15-1.03.02	Sheet Numbers	15-3
15-1.04	Revisions	15-4
15-1.05	Standard Symbols	15-4
15-1.06	Model Sheets	15-4
15-2.0	TITLE SHEET	15-4
15-3.0	DETAILED INDEX AND GENERAL NOTES	15-6
15-4.0	TYPICAL SECTIONS	15-7
15-4.01	Grading Typical Section — Model Sheet No. 3	15-7
15-4.02	Paving Typical Sections — Model Sheets No. 4 and No. 5	15-7
15-5.0	QUANTITY SHEETS	15-7
15-5.01	Summary of Quantities — Model Sheet No. 6; and Summary of Quantities (Bridge) — Model Sheet No. 7	15-8
15-5.02	Estimated Quantities (Roadway) — Model Sheet No. 8; and Estimated Quantities (Bridge) — Model Sheet No. 9	15-8
15-6.0	PLAN-PROFILE SHEETS	15-8
15-7.0	OTHER PLAN SHEETS	15-9
15-7.01	Interchange Layout Sheets	15-9
15-7.02	Detail Sheets and Standard Drawings	15-9
15-7.03	Cross Section Sheets	15-10
15-8.0	FORM GRADES	15-10
15-8.01	Plan View — Model Sheet No. 19	15-10
15-8.02	Profile View	15-10

Chapter 15

CONTRACT PLAN ASSEMBLY

15-1.0 GENERAL INFORMATION

Previous chapters have provided the designer with uniform criteria and procedures for the geometric design of the highway facility. These designs should be incorporated into the plans so that they can be clearly understood by contractors, material suppliers and Department personnel assigned to supervise and inspect the construction of the projects.

To ensure a consistent interpretation of the contract plans, individual sheets should have standard format and content, and the sequence of assembling the various types of sheets should always be the same. The purpose of this chapter is to present guidelines for uniform preparation of contract plans.

The contract plans are one portion of the Plans, Specifications and Estimates (PS&E) Assembly. The PS&E Assembly also includes specifications, all special provisions and supplemental specifications not approved as standard, the cost estimates, all approved agreements with railroads, utilities and municipalities, the proposal assembly and the utility and right-of-way certificates. Figure 15-1A illustrates the PS&E Assembly requirements for State-Aid projects.

15-1.01 Content and Sequence

Most sets of contract plans will contain the following types of sheets assembled in the sequence below:

1. Title Sheet;
2. Detailed Index and General Notes;
3. Typical Sections;
4. Summary of Quantities (Roadway and Bridge);
5. Estimated Quantities (Roadway and Bridge);
6. Plan-Profile Sheets;
7. Special Design Sheets — Roadway (Detail Sheets and Standard Drawings);
8. Bridge Drawings — boxes, wingwalls, etc.;
9. Special Design Sheets — Bridges; and
10. Cross Section Sheets.

Several other types of drawings may be required for use by the project engineer including sheets of form grades, property maps and other details.

15-1.02 Project Identification

Each sheet in the contract plans should be clearly identified with the project number. On the title sheet, this number is provided in the center heading and in a preprinted block located in the upper right corner of the sheet and also at the bottom of the sheet. On all other sheets, the project number is recorded in the upper right corner in a preprinted or stamped block and in the lower right-hand corner. The designer should obtain the project number from the approved project program. Each element of the number has a special meaning as discussed in Standard Operating Procedure (SOP) No. ADM-02-08-02-000.

FEDERAL-AID PROJECTS

PLANS

3 Sets of half-scale plans with standards
2 Sets of half-scale plans without standards

DOCUMENTS

4 Sets of contract documents

The PS&E Assembly also includes the following:

1. Right-of-Way Certificate
2. Satisfactory agreement concerning all utilities involved within the limits of this project
3. Copy of the County Engineer's preliminary estimate

STATE-AID PROJECTS

PLANS

3 Sets of half-scale plans with standards
2 Sets of half-scale plans without standards

DOCUMENTS

4 Sets of contract documents

The PS&E Assembly also includes the following:

1. Right-of-Way Certificate
2. Satisfactory agreement concerning all utilities involved within the limits of this project
3. Copy of the County Engineer's preliminary estimate

PS&E ASSEMBLY REQUIREMENTS
OFFICE OF STATE AID

Figure 15-1A

15-1.03 Numbering System

15-1.03.01 Working Numbers

Working numbers are assigned to groups of particular sheets for identification during the design process as follows:

1. Title Sheet. The title sheet is always No. 1.

2. Typical Section Sheet. The first typical section sheet is always TS-1, and any additional sheets are numbered TS-2, TS-3, etc.

3. Plan-Profile Sheet. The first mainline plan-profile sheet is always No. 3, and subsequent mainline plan-profile sheets are numbered in sequence. Additional plan-profile or special design sheets which supplement a particular sheet should have the same number as the mainline sheet plus an alphabetical designation in sequence.

Working numbers are temporary and have no value after the plans are printed for PS&E Assembly, except that working numbers recorded on the title sheet along the mainline are retained for reference purposes.

15-1.03.02 Sheet Numbers

Sheet numbers are assigned when the plans have been completed and are ready for PS&E Assembly. Each sheet is given a number in the proper sequence. Blocks of numbers have been reserved for the various types of sheets as presented in Table 15-1A. The title sheet will always be sheet No. 1. This number should appear in the preprinted block located in the upper right corner of the sheet. On all other sheets, the numbers should be recorded in a block in the lower right corner.

Table 15-1A

SHEET NUMBERS FOR CONTRACT PLANS

Sheet Numbers	Type of Drawing
1-100	*Title Sheet through Plan-Profile Sheets and other Special Design Roadway Sheets.
101-365	Standard Roadway Design Drawings.
366-465	Standard Box Culvert Drawings.
466-765	Bridge Drawing Sheets.
766-900	Special Design Sheets for Bridges.
901 and above	Cross Section Sheets.

**Note:* *If this group exceeds 100, use 100.01, 100.02, etc., for additional sheet numbers.*

15-1.04 Revisions

It is important that design revisions be carefully recorded by the designer when the revision occurs after the final right-of-way and utility plans have been submitted or after the PS&E Assembly.

Any design changes which are made after the final right-of-way and utility plans have been submitted to the Right-of-Way Division and the Utility Section are recorded on the plan-profile sheet in the revision block located in the upper right corner of the sheet. The revision block should contain the date and the designer's initials. The revision number should be placed in a triangle ⚠ opposite the revision block and also at the point on the sheet where the revision was made. Revised sheets should be submitted immediately to the Right-of-Way Division and Utility Section. They must have up-to-date information as this may affect right-of-way negotiations and/or utility adjustments.

Before the plans are printed for the PS&E Assembly (half-scale), all revision dates and revision symbols should be erased, and the Detailed Index Sheet should be stamped with a revision block. Model Sheet No. 2 at the end of Chapter 15 illustrates the revision block in the lower right corner. The date the plans are printed should be recorded in the revision blocks, and any subsequent revisions should be recorded by date and sheet number in the revision block. The revision date should also be recorded in the revision block of each individual sheet which is revised. The revision number should be placed in a triangle ⚠ opposite the revision block and also at the point on the sheet where the revision was made.

15-1.05 Standard Symbols

Commonly used standard symbols and abbreviations are shown on the last two sheets of this chapter. The designer should adhere to the standard symbols in the preparation of all contract plans. Other standard symbols can be found in Roadway Design's CADD cell library.

15-1.06 Model Sheets

The designer may refer to the model sheets assembled at the end of the chapter. These sheets do not represent a single complete set of plans; rather, they are selected individual sheets to serve as examples for particular conditions. The types of sheets included in the contract plans are discussed in the following sections with special instructions and comments related to each sheet. The County Engineer designing State-Aid projects should refer to model plans furnished by the Office of State Aid for correct plan assembly.

15-2.0 TITLE SHEET

The title sheet is always sheet No. 1 and serves as the cover sheet for the contract plans. It provides a considerable amount of information and serves as a general index and reference sheet for the project. This sheet should be prepared on vellum or matte film, and all information should be recorded in ink. Model Sheet No. 1., located at the end of the chapter, illustrates a typical title sheet. The following information should be included on the title sheet for the contract plans:

1. Title Information. The project number, the route number, the county, and a general description of the project location should be provided in the center of the sheet immediately below the words "State Highway." Project numbers are obtained from the approved project program. Project numbers should be recorded in two forms on the title sheet. The first number is a combination of alphabetical designations and numerals to comply with FHWA regulations, and the second number consists of a group of numerals to comply with Department regulations. This system of numbering is discussed in SOP ADM-02-08-02-000.

2. General Location. The general location of the project in relation to the entire State should be identified with a small circle on the small scale State map provided in the upper right corner of the sheet.

3. Project Layout. A small scale layout of the entire project alignment is provided in the center of the sheet. The scale is variable but generally is either 1 in. = 1000 ft or 1 in. = 2000 ft, depending on the length of project. The minimum information required on the project layout includes north arrow, township, range, section lines and number, urban and municipal limits, populations, county roads, all highways, rivers, all streams requiring bridges, railroads, airfields, cities or towns, approximate distance from either end to the next city or town, the project alignment, the last station of each plan-profile sheet and sheet number, omitted sections, bridges and the beginning and ending stations. The project alignment should be the predominate line of the layout. The area covered by each mainline plan-profile sheet should be sketched over the alignment, and each sheet should be identified by the working number and last station. Stationing should be in multiples of 3000 ft whenever the plan-profile sheets are drawn to a scale of 1 in. = 100 ft.

4. General Index. The number of sheets for each type of drawing and the total number of sheets in the plans should be recorded in the general index block located in the upper left corner of the sheet.

5. Bridge Locations. All bridges and box bridges should be listed immediately to the left of the project layout. Each structure is identified by an alphabetical designation, beginning station, total length, and number and length of spans. Even though several bridge lengths may be provided, only those bridges with lengths along the centerline of the project are included in the length data.

6. Equations. All equations along the centerline of the project should be listed. The list should identify the length of each equation with the appropriate sign. An algebraic summation is then made to obtain a total length of all equations.

7. Exceptions. Exceptions are omitted sections within the project limits. Bridges in place (excluding box bridges) are presented on the layout as omitted sections and included in the length of exceptions.

8. Length Data. The project length data are listed in five increments on the title sheet including length of roadway, length of bridges, net length of project, length of exceptions and gross length of project. The length of roadway is the difference between the beginning and ending stations plus (or minus) the total length of the equations minus the length of bridges and exceptions. The length of bridges is the sum of those bridges along the centerline which will be constructed on the project. A box bridge is defined as a box culvert which has a clear distance between the inside faces of the end supports exceeding 20 ft measured along the centerline of the project. It is included in the length of bridges on grading projects only. The length data for a box bridge is measured from outside face to outside face of the end supports along the centerline of the project. The length of exceptions is the sum of all omitted sections within the project limits. The net length of project is the sum of the roadway length and bridge length, and the gross length of project is the sum of the net length and the exceptions.

   As a general rule, the length data on 4-lane facilities are computed along the centerline of the median when the median width is a constant distance, or along the centerline of the right lane when the median width is variable. The accuracy of all computations for project length are recorded to the nearest 0.01 ft.

9. Design Controls. Design control data which are obtained from the project design data sheet (RWD-600) should be recorded in the space provided.

10. Access Control. All controlled access facilities, except interstates, require a statement on the title sheet declaring the project to be a controlled access facility and stating the type of control. Access control definitions and requirements are discussed in Section 2-4.0 and in SOP ADM-23-01-00-000.

11. Environmental Permit Stamp. Indicates the types of environmental permits required for the project.

15-3.0 DETAILED INDEX AND GENERAL NOTES

The second sheet of the contract plans normally will be a combination sheet titled "Detailed Index and General Notes." The detailed index will contain a description of each sheet in the plans grouped under the same titles and sequence as in the general index. The index is located in the upper left corner of the title sheet. The general notes will be limited to information specifically applicable to a particular project. Information contained in the *Standard Specifications* or Special Provisions should not be provided as a general note. Model Sheet No. 2, located at the end of the chapter, illustrates a typical detailed index and general notes sheet.

On the detailed index sheet, the titles of each type of sheet should be underlined and the total number of sheets within the group recorded in parentheses immediately to the right. The working number and sheet number should be provided adjacent to the description of each sheet (see Section 15-1.03). The latest revision date for a standard drawing, bridge or roadway should be provided adjacent to the sheet number.

Typical section sheets should be arranged in sequence with the most important typical section first and the least important section last. Sections should be presented in the following sequence:

1. main facility,
2. interchange ramps and loops,
3. interchange local road,
4. other highways,
5. streets,
6. frontage roads,
7. county roads,
8. access roads, and
9. detour roads.

Quantity sheets should be arranged in the following sequence:

1. Summary of Quantities (roadway),
2. Summary of Quantities (bridges),
3. Estimated Quantities (roadway), and
4. Estimated Quantities (bridges).

Plan-profile sheets should be arranged in the sequence as depicted on Model Sheet No. 2. As indicated, the first sheet should be the first mainline sheet, and the working number will be 3. The limits of this sheet contain an interchange. Five supplemental sheets (3-A through 3-E) are required to provide all features of the interchange and should be placed immediately after the mainline sheet. Immediately following the supplemental sheets are the remaining mainline sheets assigned working numbers 4 and 5.

The general index on the title sheet has the heading "Special Design — Roadway Items." This is separated into two categories on the detailed index sheet. The first category includes detail sheets and the second category includes Standard Drawings. The detail sheets may pertain to several locations or to the entire project and are not arranged in any particular sequence. The Standard Drawings should be arranged in numerical order.

The bridge sheets follow the roadway design sheets. Bridge drawings (standards) should be arranged in numerical order on the detailed index sheet, and the special design sheets — bridges should be arranged in alphabetical and numerical sequence.

15-4.0 TYPICAL SECTIONS

Depending on the complexity of the project, one or more typical section sheets may be required in each set of contract plans. The mainline typical section should be displayed first, with other typical sections following in the sequence as discussed in Section 15-3.0. Each typical section should be clearly identified by name and station limits. Model sheets, located at the end of the chapter, provide three different types of typical sections. Model Sheet No. 3 illustrates a grading project, and Model Sheets No. 4 and No. 5 illustrate paving projects. These are discussed in more detail in the following sections.

15-4.01 Grading Typical Section — Model Sheet No. 3

Details of the pavement structure are not required on typical sections when pavement work is not included in the project. However, the total pavement structure thickness (above subgrade) and the proposed slope of the pavement and shoulders should be shown as a broken line. The scale of a typical section is seldom specified, although a scale of 1 in.= 5 ft usually is sufficient for grading projects, including 4-lane facilities up to 64-ft constant median width. Variable median widths and widths over 64-ft generally require a scale of 1 in.= 10 ft to depict the full facility and independent details for one or both lanes.

Criteria for slopes as related to the height of cuts and fills should be provided on the typical section sheets. This information can be obtained from the project design data sheet (RWD-600), which should conform to the side slope criteria presented in the Tables of Geometric Design Criteria in Chapters 2, 11, 13 and 14. The typical section should be drawn and labeled as ½ cut section and ½ fill section. Special details such as slope benching should be provided on the typical section sheets. Notes which apply specifically to the typical section should be listed below the drawing.

15-4.02 Paving Typical Section — Model Sheets No. 4 and No. 5

Large scale drawings should be provided to clearly depict the various courses in the pavement structure for paving projects or combination grading and paving projects. Generally, a scale of 1 in.= 2 ft is adequate to clearly present all details.

The various courses should be indexed by number and described on the right side of the cross section sheet. All terminology should be identical to the terminology in the *Standard Specifications.*

All courses must be accurately described as to width, thickness and type of material. When the pavement structure of a 4-lane facility is symmetrical, only one roadway (2-lanes) must be provided. Pavement structure recommendations are submitted by the districts and, when approved by the Chief Engineer, become a part of the project file. On Federal-Aid projects, the FHWA must also approve pavement structure recommendations.

15-5.0 QUANTITY SHEETS

Model sheets, located at the end of the chapter, provide four different types of quantity sheets as follows:

1. Summary of Quantities (Roadway) — Model Sheet No. 6
2. Summary of Quantities (Bridge) — Model Sheet No. 7
3. Estimated Quantities (Roadway) — Model Sheet No. 8
4. Estimated Quantities (Bridge) — Model Sheet No. 9

If the project involves only roadway work, the sheets for bridge quantities are not included.

15-5.01 Summary of Quantities — Model Sheet No. 6; and Summary of Quantities (Bridge) — Model Sheet No. 7

The purpose of these sheets is to summarize all estimated quantities by pay item. Pay items are ordinarily arranged in three groups:

1. Roadway Items. Roadway pay items include pavement structure, erosion control, traffic control and all other items required for the construction of the project except those items relating to box bridges and bridges.

2. Box Bridge Items. Box bridge items consist of all pay items necessary to construct box bridge(s).

3. Bridge Items. Bridge items consist of all items necessary for the construction of bridge structure(s), including the slope paving beneath and equal to the width of the structure plus 8 ft (4-ft width additional on each side of the bridge). The remainder of slope paving, if any, is a roadway item.

On the summary of quantities sheet, the items should be arranged in numerical order by pay item within each category. The pay item numbers, the correct terminology and the unit of measure are all contained in the *Standard Specifications* or Special Provisions. The designer should record the unit of measure and preliminary quantities on the sheet and allow a blank column for final quantities. Quantities should also be divided by county and by urban and rural areas. Any notes required to explain how quantities were obtained or to define rates of application not contained in the *Standard Specifications* should be provided as indicated on the model sheet.

Bridge quantities are summarized separately from other items to allow the Bridge and Roadway Design Divisions to complete their plan work independently. Generally, the same rules apply to both roadway and bridge summary sheets.

15-5.02 Estimated Quantities (Roadway) — Model Sheet No. 8; and Estimated Quantities (Bridge) — Model Sheet No. 9

In addition to the summary of quantities sheet, roadway plans usually contain a second sheet entitled, "Estimated Quantities (Roadway)." If the project includes bridges, a similar sheet, "Estimated Quantities (Bridge)," is included. The purpose of these sheets is to be separate quantities by location or structure, to consolidate information presented throughout the plans and to provide information not included elsewhere in the contract plans.

Depending on the items required, the roadway sheet(s) generally will contain columns for pipe culverts, box culverts, box bridges, ditch treatment, guardrail, curb and gutter, riprap or other items which appear at several locations. The bridge sheet consolidates all information by individual structure and by components for various pay items. The format of each sheet should remain standardized as much as possible. Model Sheets Nos. 8 and 9 are considered typical for most projects. Quantities should also be divided by county and by urban and rural areas.

15-6.0 PLAN-PROFILE SHEETS

The plan-profile sheets present the vertical and horizontal alignments, topography, right-of-way, construction items and other details necessary for the construction of the project. Model sheets, located at the end of the chapter, are provided for six different conditions as follows:

1.	Rural 2-Lane	— Model Sheet No. 10
2.	Rural 4-Lane	— Model Sheet No. 11
3.	Rural 2-Lane, Paving Only	— Model Sheet No. 12
4.	Rural 4-Lane, Paving Only	— Model Sheet No. 13
5.	Urban	— Model Sheet No. 14
6.	Interchange Ramp	— Model Sheet No. 15

The basic survey information, including the horizontal alignment, curve data, topography, bench marks, groundline profile and groundline elevations, should be recorded first on the plan-profile sheets. This information should be inked. All other design information may be recorded in pencil. The following information should also be included on the plan-profile sheet for the contract plans:

1. Scales. Scales normally will be 1 in.= 100 ft horizontally (3000 ft per sheet) and 1 in.= 10 ft vertically. Large scales should be used in more restricted urban areas and other situations where there is need to provide considerable detail. Model Sheet No. 14 uses a horizontal scale of 1 in. = 20 ft (500 ft per sheet) and a vertical scale of 1 in. = 5 ft.

2. Elevations. The two rows of figures along the bottom of the plan-profile sheet represent the profile elevations. The bottom row provides the original groundline elevations, and the upper row provides the plan grade elevations. For paving plans, only the plan grade elevations are presented and not the groundline elevation. The groundline profile is shown as a broken line.

3. Earthwork Distribution. Earthwork distribution diagrams should appear above the plan grade elevations and below the original ground profile. Earthwork distribution diagrams may be shown on separate detail sheets for clarity. Guidelines for earthwork distribution diagrams are discussed in Section 5-3.04.

4. Notes. Notes on the plan-profile sheets should be brief, clear and consistent. The format and location of the various types of notes should follow the examples presented in the model sheets.

15-7.0 OTHER PLAN SHEETS

15-7.01 Interchange Layout Sheets

A layout sheet is required for all interchanges. Model Sheet No. 16, located at the end of the chapter, illustrates a typical layout sheet. The purpose of a layout sheet is to provide a plan view with the following information:

1. horizontal alignment and curve data,
2. pavement width throughout the interchange,
3. right-of-way required,
4. access controls,
5. bridges, and
6. design year ADT and distribution.

Topography, drainage structures and other information are not provided on a layout sheet. The layout sheet should be inserted in the contract plans immediately after the mainline plan-profile sheet which includes the intersection station of the mainline and local road.

15-7.02 Detail Sheets and Standard Drawings

The purpose of a detail sheet is to present design details that cannot be described adequately on the plan-profile sheets. Model sheets, located at the end of the chapter, provide two different types of detail sheets. Model Sheet No. 17 illustrates pavement marking details, and Model Sheet No. 18 illustrates intersection details. Additional detail sheets may be required for median barriers, special guardrail installations, slope paving, lighting or signal installations. Vegetation schedules are included with detail sheets.

The Department's *Standard Drawings* which are applicable to the project should be identified in the detailed index and inserted in the plans immediately following the detail sheets.

15-7.03 Cross Section Sheets

Cross section sheets are the last group of sheets listed in the detailed index. The first sheet is always sheet No. 901. Model sheets, located at the end of the chapter, provide typical cross sections for two different conditions. Model Sheet No. 901 illustrates a two-lane highway, and Model Sheet No. 968 illustrates a multilane highway with a divided median.

The entire mainline cross section sheets should be sequentially numbered from beginning to end without interruption. Any supplemental cross sections (ramps, frontage roads, local roads, channel changes, etc.) should be grouped after the mainline cross sections, with each sheet clearly labeled in the lower right corner as to purpose and location. Cross sections are a part of the contract plans but are printed separately. They are furnished to the contractor and the project engineer. Detailed guidelines for preparing cross section sheets are discussed in Chapter 5.

15-8.0 FORM GRADES

The purpose of preparing form grades is to graphically determine the profiles of ramps at intersections with roadways. This is usually done by coordinating the plan with the profile. In most cases, the plan-profiles are prepared on separate drawings due to the large scale of each drawing. The scale normally used for the plan view is 1 in. = 30 ft. The scale normally used for the profile view is 1 in. = 20 ft horizontally and 1 in. = 2 ft.

The objective of the designer is to establish a smooth profile for both edges of the ramp pavement at 25-ft intervals and to provide means for graphically determining the grade at any location. Model sheets, located at the end of the chapter, illustrate typical form grades. Model Sheet No. 19 illustrates the plan view. The plan and profile views are discussed in the following sections.

15-8.01 Plan View — Model Sheet No. 19

The purpose of the plan view is to:

1. determine location of construction joints or crossover crown line;
2. determine pavement slopes and slope transitions;
3. accurately scale pavement width at any location;
4. check rate of cross slope and maximum algebraic difference; and
5. check for proper pavement drainage.

The plan view should clearly present:

1. elevations at 25-ft intervals along the control line of the ramp;

2. pavement slope line, perpendicular (radially) to the ramp control line;

3. pavement slope (percent), recorded above the pavement slope line (with an arrow, if there is doubt as to direction of slope); and

4. pavement width, recorded below the pavement slope line.

15-8.02 Profile View

The purpose of the profile view is to:

1. graphically define the profiles of the ramp edges and intersecting roadways to a scale large enough to read to the nearest 0.01 ft; and

2. establish the best grades possible, by coordination with the plan view.

The profile view should be prepared by:

1. plotting the control elevations from the plan view (from tie points to nose points);

2. fitting splined curves to calculated profiles;

3. smoothing out any abrupt changes with splined curves and adjusting elevations as needed between control points; and

4. clearly labeling each pavement edge or joint line.

FED. ROAD REG. NO.	STATE	PROJECT NO.	SHEET NO.
4	MISS.	NH-0011-03(062)	1

GENERAL INDEX

FOR DETAILED INDEX OF PLANS SEE SHEET NO. 2

DESCRIPTION	NUMBER OF SHEETS
TITLE SHEET	1
DETAILED INDEX	3
GENERAL NOTES	5
TYPICAL SECTIONS	11
QUANTITIES	5
PLAN AND PROFILES	5
SPECIAL DESIGN - ROADWAY ITEMS	47 △1
BRIDGE DRAWINGS	20
SPECIAL DESIGN - BRIDGES	
CROSS-SECTIONS	210
TOTAL SHEETS	302 △1

STATE OF MISSISSIPPI

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

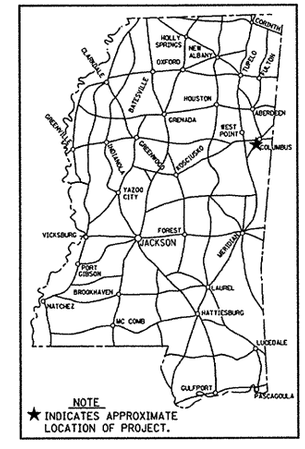
PLAN AND PROFILE OF PROPOSED STATE HIGHWAY FEDERAL AID PROJECT NO. NH-0011-03(062)

U.S. 45 FROM 4 MILES SOUTH OF U.S. 82 TO U.S 82
LOWNDES COUNTY

FMS CONST. 10041/301000

SCALES

PLAN		1 IN. = 100 FT.
PROFILE	HOR.	1 IN. = 100 FT.
	VERT.	1 IN. = 10 FT.
LAYOUT		1 IN. = 1500 FT.



NOTE
★ INDICATES APPROXIMATE LOCATION OF PROJECT.

BRIDGE STRUCTURES REQ'D.

(A) STA. 614 + 55.137 (LT. LANE) 3 @ 75 FT., 1 @ 105 FT.
LENGTH ALONG ℄ 330 FT.

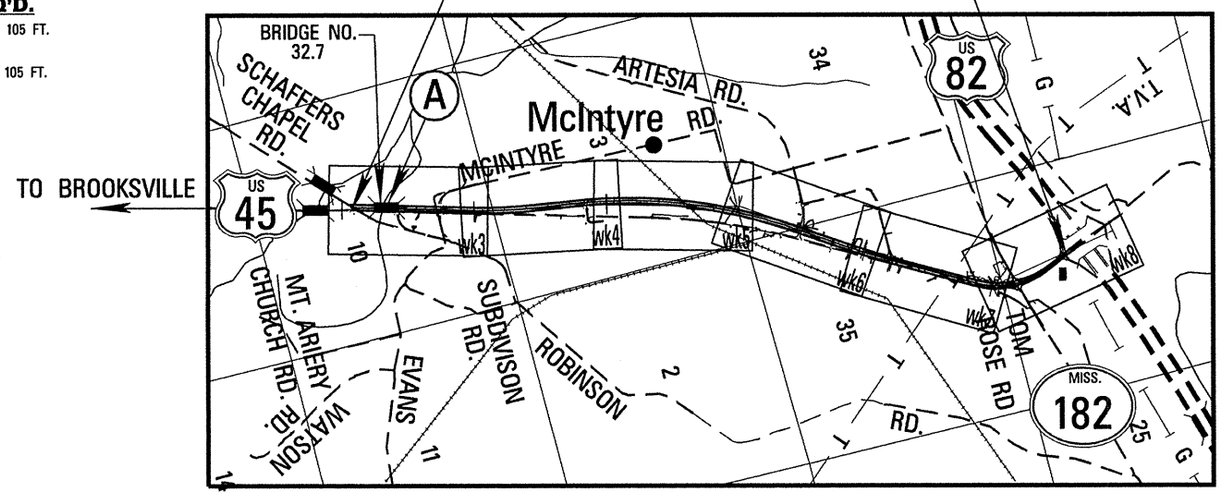
STA. 614 + 21.137 (RT. LANE) 3 @ 75 FT., 1 @ 105 FT.
LENGTH ALONG ℄ 330 FT.

BOX BRIDGE'S REQ'D.

NONE

B.O.P. 610 + 00

E.O.P. 778 + 00



DESIGN CONTROL

70 MPH = V (SPEED DESIGN)

ADT (2001) = 4750 : ADT (2021) = 8700

DHV = 780 : D = 55 % T= 10 %

ENVIRONMENTAL PERMITS

TYPE	REQUIRED YES	REQUIRED NO	BY
N.W. (WATERS OF U.S.)	X		CKP
N.W. (WETLANDS)		X	CKP
G.P.		X	CKP
404	X		CKP
STORMWATER	X		CKP

ACCESS CONTROL

NOTES:
1. Access to and exit from this highway will be permitted only through interchange or such other points as may be established by public authority and as shown on the plans.
2. This note applies the following station limits:
STA. 760 + 00 to STA. 771 + 00.
This project is declared by the Transportation Commission to be a Type 1 Controlled Access Facility, as defined in and subject to all restrictions shown by order of said Commission dated 6th. day of May, 1999 in minute book 4, page 183 and authorized under section 65-1-10(i) MCA (1972, as amended).

CONVENTIONAL SYMBOLS

COUNTY LINE
TOWN CORPORATION LINE
SECTION LINE
EXISTING ROAD OR TRAVELED WAY
PROPOSED ROAD OR TRAVELED WAY
RAILROAD
SURVEY LINE
BRIDGES

EQUATIONS

STA. 652 + 55.590 BK. = STA. 652 + 47.230 AH. = + 8.36
STA. 710 + 60.000 BK. = STA. 710 + 87.470 AH. = -27.47
-19.11

EXCEPTIONS

NONE

LENGTH DATA

LENGTH OF ROADWAY	16450.89	FT.	3.1157 MI.
LENGTH OF BRIDGES	330.00	FT.	0.0625 MI.
LENGTH OF PROJECT (NET)			3.1782 MI.
LENGTH OF EXCEPTIONS		FT.	MI.
LENGTH OF PROJECT (GROSS)	16780.89		3.1782 MI.

DATE	REVISED NO. OF SHEETS AND TOTALS REVISIONS	BY
5-5-99		RSW

APPROVED:
JAMES H. KOPF — CHIEF ENGINEER — DATE 2-1-01
HUGH LONG — EXECUTIVE DIRECTOR — DATE 2-1-01
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

APPROVED:
DIVISION ADMINISTRATOR — DATE
FEDERAL HIGHWAY ADMINISTRATION
DEPARTMENT OF TRANSPORTATION

TITLE
ROADWAY DESIGN DIVISION C.A.D.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	SDP-0000-00(0)

DESCRIPTION OF SHEET	REVISION DATE	WKG. NO.	SH. NO.
TITLE SHEET (1)			1
DETAILED INDEX & GENERAL NOTES (1)		DI-1	2
TYPICAL SECTION SHEETS (3)			
MAIN FACILITY - B.O.P. TO STA. 51+00 (CEDAR LAKE ROAD)		TS-1	3
MAIN FACILITY - STA. 51+00 (CEDAR LAKE ROAD) TO E.O.P.		TS-2	4
INTERCHANGE RAMPS		TS-3	5
QUANTITY SHEETS (4)			
SUMMARY OF QUANTITIES		SQ-1	6
SUMMARY OF QUANTITIES (BRIDGE ITEMS)		SQ-2	7
ESTIMATED QUANTITIES - DRAINAGE STRUCTURES, FENCING, GUARD RAIL, CURBING & PAVED DITCHES		EQ-1	8
ESTIMATED QUANTITIES - BRIDGE		EQ-2	9
PLAN & PROFILE SHEETS (8)			
MAIN FACILITY - B.O.P. TO STA. 60+00		3	10
INTERCHANGE LAYOUT - CEDAR LAKE ROAD INTERCHANGE		3A	11
S. E. RAMP		3B	12
S. W. RAMP		3C	13
N. E. RAMP		3D	14
N. W. RAMP		3E	15
MAIN FACILITY - STA. 60+00 TO STA. 90+00		4	16
MAIN FACILITY - STA. 90+00 TO E.O.P.		5	17
SPECIAL DESIGN SHEETS (10)			
TRAFFIC CONTROL PLAN - PHASE I TRAFFIC ON EXISTING LANE		TC-1	18
TRAFFIC CONTROL PLAN - PHASE II TRAFFIC ON PROPOSED NEW LANE		TC-2	19
TRAFFIC CONTROL PLAN - BARRICADE AND DRUM PLACEMENT PHASE I		TC-3	20
TRAFFIC CONTROL PLAN - BARRICADE AND DRUM PLACEMENT PHASE II		TC-4	21
INTERSECTION DETAIL - STA. 51+00		TC-5	22
TEMPORARY PAVEMENT MARKING DETAIL - STA. 51+00		TC-6	23
PAVEMENT MARKING DETAIL - STA. 51+00		TC-7	24
VEGETATION SCHEDULE		VS-1	25
VERTICAL BOX CULVERT BENDING DETAIL		BCB-1	26
EROSION CONTROL		EC-1	27
STANDARD DRAWINGS - ROADWAY SHEETS (9)			
BRIDGE END PAVEMENT		BE-1	107
PAVEMENT MARKING DETAILS FOR 2 & 4-LANE DIVIDED ROADWAYS	(12-01-99)	PM-1	120
HIGHWAY SIGN & BARRICADE DETAILS FOR CONSTRUCTION PROJECTS		TCP-10	259
RURAL DRIVEWAYS		RD-1	271
TYPICAL GRADING TRANSITION BETWEEN CUTS & FILLS		GT-1	272
SIGHT FLARES		SF-1	273
SUPERELEVATION TRANSITION - CASE I ROTATION ABOUT CENTERLINE (2.0% NORMAL SUBGRADE)		SE-2A	276
PIPE CULVERT INSTALLATION		PI-1	300
FLARED END SECTION FOR CONCRETE PIPE		FE-1	328

DESCRIPTION OF SHEET	REVISION DATE	WKG. NO.	SH. NO.
STANDARD DRAWINGS - BRIDGE SHEETS (4)			
BOX CULVERT DRAWING - BARREL JOINT LOCATIONS - NORMAL & SKEWED CULVERTS GROUP I DIAGRAMS		IBJL-1	366.1
COLLAR DETAILS FOR BOX STRUCTURES		ICJ-1	367
BASIC CULVERT DRAWING - SINGLE CELL - HEIGHT 6 FT. - SPANS 6 - 20 FT.		IBS-6-2W	370.1
WINGS WITH 3:1: SLOPE FOR BASIC CULVERT DRAWING - SINGLE CELL - HEIGHTS 6 - 12 FT. - SPANS 6 - 24 FT.		IWS-3	374
SPECIAL DESIGN SHEETS - BRIDGE (5)			
BRIDGE A, RT. LN. STA. 50+35, LT.LN. STA. 50+37.25 (CEDAR LAKE ROAD)			
ELEVATION LAYOUT		A1 OF 5	466
END BENT NO. 1 DETAILS		A2 OF 5	467
END BENT NO. 2 DETAILS		A3 OF 5	468
BOX GIRDER PIER BENT DETAILS		A4 OF 5	469
DECK PLAN		A5 OF 5	470
CROSS SECTIONS (42)			
MAIN FACILITY			901-927
CEDAR LAKE ROAD INTERCHANGE RAMPS			928-935
RELOCATION CEDAR LAKE ROAD			936-942
TOTAL SHEETS			87

GENERAL NOTES:

(1) A SOIL PROFILE HAS BEEN PREPARED FOR THIS PROJECT USING SAMPLES TAKEN FROM HOLES AT THE LOCATIONS INDICATED IN THE TEST REPORTS. THIS SOIL PROFILE IS ON FILE IN THE DISTRICT AND CENTRAL CONSTRUCTION OFFICES AND IS AVAILABLE FOR EXAMINATION. THE DEPARTMENT DOES NOT GUARANTEE THAT THE MATERIALS AS SHOWN IN THE REPORTS ARE NECESSARILY TO BE FOUND OUTSIDE THE TEST HOLES.

(2) THE LOCATION AND SPACING OF SIGNS, SHOWN ON THE TRAFFIC CONTROL PLANS, ARE APPROXIMATE AND MAY BE ADJUSTED AS NECESSARY TO FIT FIELD CONDITIONS.

(3) 25% SHRINKAGE FACTOR USED IN EARTHWORK COMPUTATIONS.

(4) FOR LIST OF UTILITIES, SEE WORKING SHEET # 3.

McCOLLUM

PS & E PLANS-DATE 2-27-97

REVISIONS

DATE	SHEET NO.	BY
3-5-97	5, 8 & 9	DMM

DATE	REVISION	BY

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

DETAILED INDEX AND GENERAL NOTES

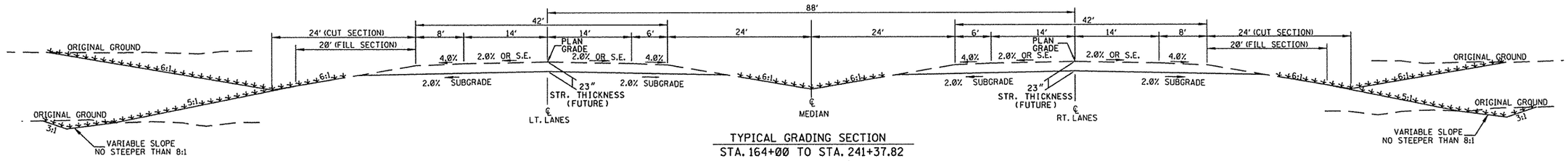
PROJ. NO.: SDP-0000-00(0)
RANKIN COUNTY

FILENAME: MACK/PICK/DI.DGN
DESIGN TEAM PICK CHECKED DATE 2-25-97

WORKING NUMBER DI-1
SHEET NUMBER 2

PLAN
HIGHWAY DESIGN DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

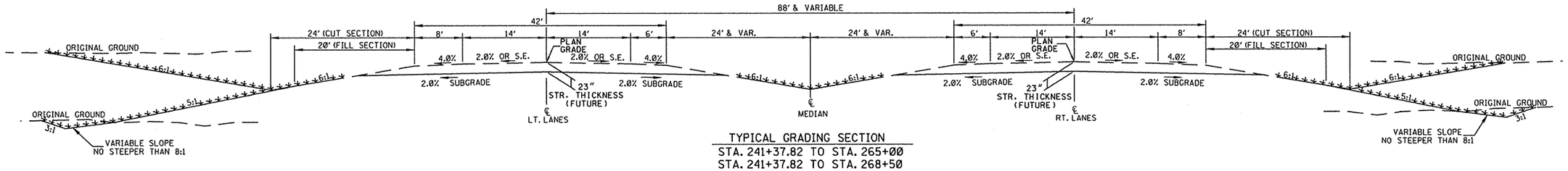
STATE	PROJECT NO.
MISS.	STP-0079-01(025)PH2



TYPICAL GRADING SECTION
STA. 164+00 TO STA. 241+37.82

HIGH VOLUME CHANGE SOIL

NOTES:
NO CONSTRUCTION REQUIRED ABOVE SUBGRADE THIS CONTRACT



TYPICAL GRADING SECTION
STA. 241+37.82 TO STA. 265+00
STA. 241+37.82 TO STA. 268+50

HIGH VOLUME CHANGE SOIL

INDICATES AREA TO BE TREATED IN ACCORDANCE WITH THE VEGETATION SCHEDULE. SEE WK. SH. NO. VS-1

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

TYPICAL SECTION

MAIN FACILITY GRADING
(RURAL GRADING)

STA.164+00 TO STA. 268+50
PROJ. NO.:
STP-0079-01(025)PH2
COUNTY: LOWNDES

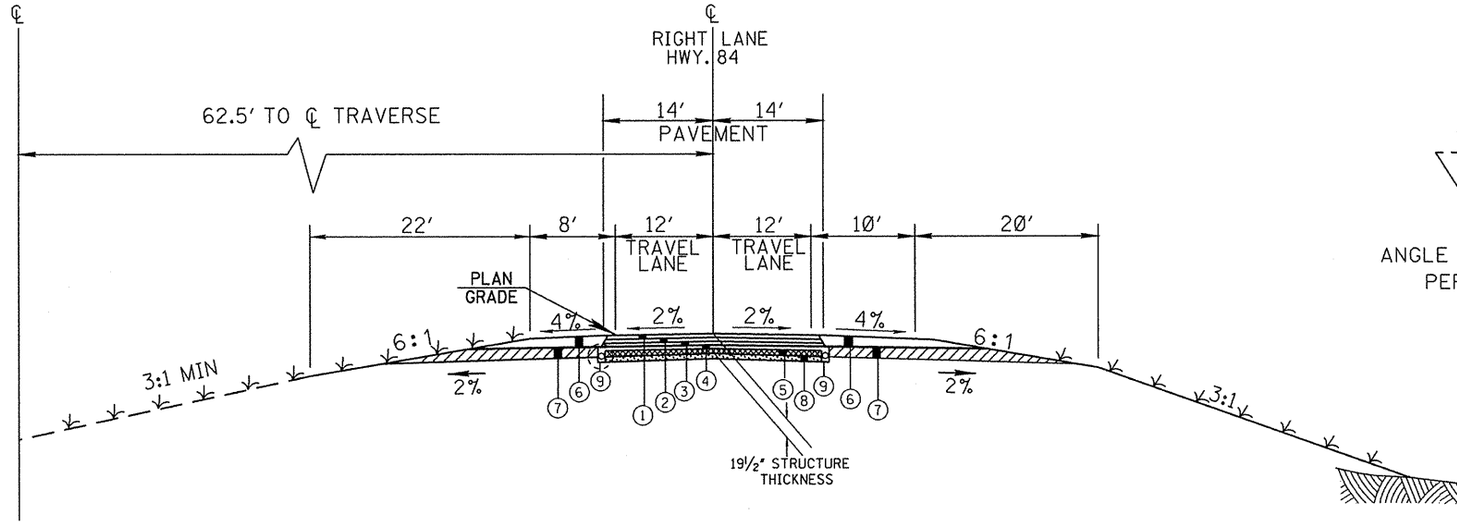
REVISION	BY	DATE

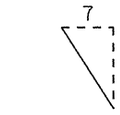
FILENAME: LOWNDES\SOUTH\TYPSEC.DG
DESIGN TEAM _____ CHECKED _____ DATE _____

WORKING NUMBER: TS-2
SHEET NUMBER: 3

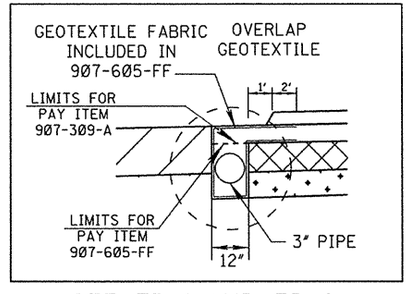
PLAN
HIGHWAY DESIGN DIVISION C.A.D.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	SDP-0015-02(099)





ANGLE OF REPOSE PERMITTED



SEE ED-1 AND ED-2 FOR OTHER DETAILS

TYPICAL SECTION

(B.O.P.) STA. 441+50 - STA. 472+98.53
STA. 0+00 - STA. 2+48.875
STA. 5+01.875 - STA. 32+00

✱ ① 1½″ HOT MIX ASPHALT, HT (9.5 mm MIXTURE), POLYMER MODIFIED REQ'D. (1 @ 1.5″)
② 2″ HOT MIX ASPHALT, HT (12.5 mm MIXTURE) POLYMER MODIFIED REQ'D. (1 @ 2″)
③ 6″ HOT MIX ASPHALT, HT (19 mm MIXTURE) REQ'D. (2 @ 3″)
④ 4″ CRUSHED STONE DRAINAGE LAYER REQ'D.
⑤ 6″ AND VAR. DEPTH GRANULAR MATERIAL (CLASS 9, GROUP C) --- 6″ LIME-FLY ASH TREATED BASE (3% LIME BY WT. & 12% FLY ASH BY WT.) REQ'D.
⑥ 9½″ AND VAR. DEPTH SHOULDER GRANULAR MATERIAL (CLASS 6, GROUP C) REQ'D.
⑦ 10″ AND VAR. DEPTH GRANULAR MATERIAL (9/C) REQ'D.
⑧ 6″ CHEMICALLY TREATED SUBGRADE REQ'D. (SEE NOTE)
⑨ EDGE DRAIN REQ'D. (PER ED-1 & ED-2)

INDICATES AREA TO BE TREATED IN ACCORDANCE WITH THE VEGETATION SCHEDULE. SEE WK. SH. NO. VS-1

NOTE : QUANTITIES FOR THE CHEMICAL TREATMENT OF THE SUBGRADE SHALL BE BASED ON LIME (6% LIME) TREATING 60% OF THE LENGTH OF THE PROJECT AND LIME-FLY ASH (3% LIME, 12% FLY ASH) TREATING THE REMAINING 40% OF THE PROJECT. 30′ WIDE TYPICAL.

✱ TO BE PLACED ON RIGHT LANES BEFORE OPENING TO 4-LANE TRAFFIC. TO BE PLACED ON LEFT LANES AFTER TRAFFIC IS OPENED TO 4-LANES.

DATA FOR PAVEMENT DETERMINATION

(2003) ADT =	4,200		Current
(2013) ADT =	5,900		
(2023) ADT =	8,300		Design
DHV =	830		
D =	50	% of DHV	
T =	29	% of DHV	
T (total) =	34	% of DHV	
18K (Rigid) =	1515	/ 1000	
18K (Flex) =	985	/ 1000	
Design CBR =	5		

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

TYPICAL SECTION
MAINLINE
(RURAL PAVING)

COUNTY: JEFFERSON DAVIS
PROJ. NO.: SDP-0015-02(099)

WORKING NUMBER TYP-1

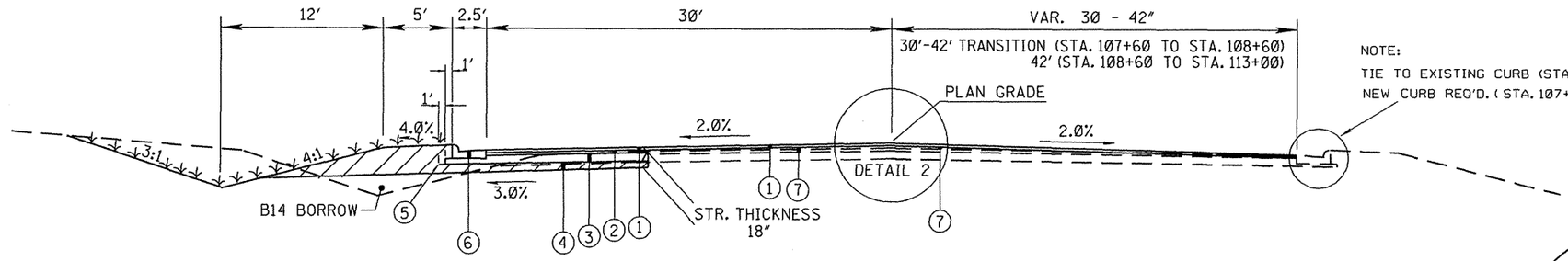
SHEET NUMBER 4

FILENAME: TYP.DGN
DESIGN TEAM WILLIAMS CHECKED DATE

REVISION | BY | DATE

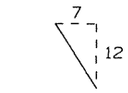
ROADWAY DESIGN DIVISION C.A.D.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION
PLAN

STATE	PROJECT NO.
MISS.	STP-7304-00(004)

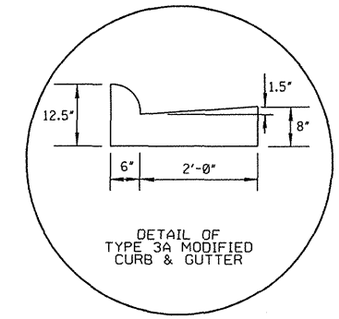


NOTE:
TIE TO EXISTING CURB (STA. 106+50 TO STA. 107+50)
NEW CURB REQ'D. (STA. 107+50 TO STA. 113+00)

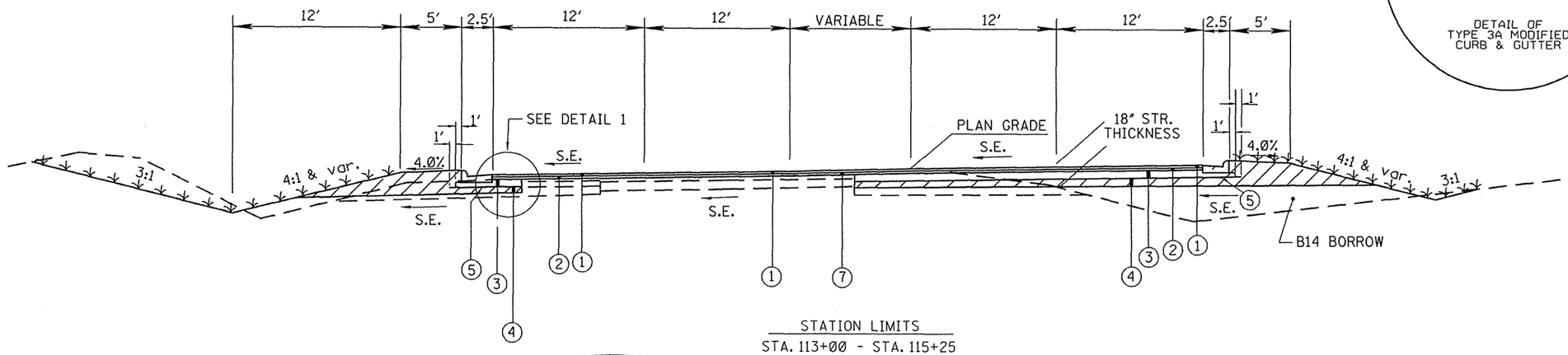
STATION LIMITS
STA. 106+50 - STA. 113+00



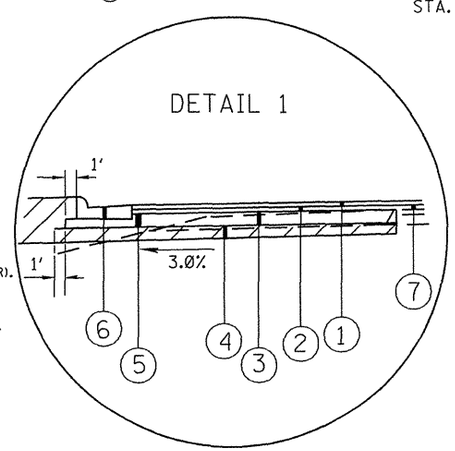
ANGLE OF REPOSE
PERMITTED

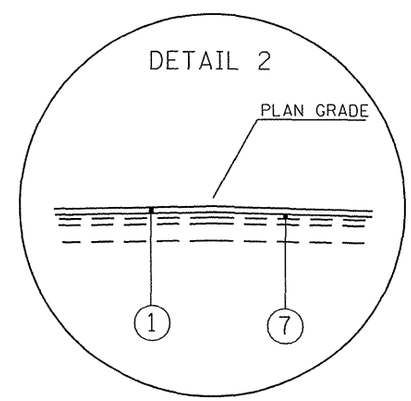


DETAIL OF
TYPE 3A MODIFIED
CURB & GUTTER



STATION LIMITS
STA. 113+00 - STA. 115+25





1. 2¼" HOT MIX ASPHALT MT. (12.5 mm MIXTURE)(1@2¼") REQ'D
2. 2¼" HOT MIX ASPHALT MT. (19 mm MIXTURE)(1@2¼") REQ'D
3. 7½" HOT MIX ASPHALT MT. (25 mm MIXTURE)(1@4", 1@3½" on top) REQ'D
4. 6" & VARIABLE DEPTH GRANULAR MATERIAL (CLASS 5, GROUP "C") REQ'D.
5. PRIME COAT REQ'D (MAY BE DELETED WHEN PERMITTED BY THE ENGINEER).
6. COMBINATION CONCRETE CURB AND GUTTER (TYPE 3A MODIFIED) REQ'D.
7. 2¼" & VAR. HOT MIX ASPHALT, MT. (19 mm MIXTURE) (1@2¼" & VAR.) REQ'D.

INDICATES AREA TO BE TREATED IN ACCORDANCE WITH THE VEGETATION SCHEDULE. SEE WK. SH. NO. VS-1

DATE	REVISION	BY

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

TYPICAL SECTION WIDENING & OVERLAY MAINLINE (URBAN)

PROJECT: NO. STP-7304-00(004)
MADISON COUNTY

FILENAME: typhwy51.dgn
DESIGN TEAM GROBINSON CHECKED DATE

WORKING NUMBER TS-1
SHEET NUMBER 5

PLAN PRODUCT FROM C.A.D. SECTION MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	BR-0010-01(073)

SUMMARY OF QUANTITIES

PAY ITEM NO.	PAY ITEM	UNIT	PRELIMINARY	FINAL	
	***** EARTHWORK ITEMS *****				
201-A	CLEARING AND GRUBBING	LUMP SUM	100%		
202-A	REMOVAL OF OBSTRUCTIONS (TWIN BRIDGES)	LUMP SUM	100%		1
202-B	REMOVAL OF FENCE	LIN.FT.	1100		1
202-B	REMOVAL OF FLARED END SECTIONS (ALL SIZES)	EACH	12		
202-B	REMOVAL OF HEADWALLS (ALL SIZES)	EACH	2		
202-B	REMOVAL OF GUARDRAIL	LIN. FT.	800		3
202-B	REMOVAL OF TRAFFIC STRIPE	LIN. FT.	4000		
202-B	REMOVAL OF CONCRETE PAVEMENT (ALL THICKNESSES)	SQ.YD.	5250		
202-B	REMOVAL OF ASPHALT PAVEMENT (ALL THICKNESSES)	SQ.YD.	29,031		6
202-B	REMOVAL OF CONCRETE LUG ANCHORS	LIN. FT.	192		
202-B	REMOVAL OF CURB (ALL TYPES)	LIN. FT.	200		
202-B	REMOVAL OF BRIDGE PILINGS (CONCRETE)	EACH	30		
203-EX	BORROW EXCAVATION (LVM)(AH)(CLASS B-3)(CONTRACTOR FURNISHED)	CU.YD.	120,585		5
206-A	STRUCTURE EXCAVATION	CU.YD.	47		
206-B	SELECT MATERIAL FOR UNDERCUTS (CONTRACTOR FURNISHED)(FM)	CU.YD.	15		2
	***** ROADSIDE DEVELOPMENT ITEMS *****				
211-A	TOPSOIL FOR SLOPE TREATMENT (FROM RIGHT-OF-WAY)	SQ.YD.	96,600		
212-B	STANDARD GROUND PREPARATION	SQ.YD.	370,754		
907-213-A	AGRICULTURAL LIMESTONE	TON	231		
213-B	COMBINATION FERTILIZER (13-13-13)	TON	39		
213-C	SUPERPHOSPHATE	TON	39		
214-A	SEEDING (BERMUDA GRASS)	LB.	1540		
214-A	SEEDING (TALL FESCUE)	LB.	575		
214-A	SEEDING (CRIMSON CLOVER)	LB.	766		
214-A	SEEDING (BAHIAGRASS)	LB.	1925		
215-A	VEGETATIVE MATERIALS FOR MULCH	TON	154		
216-A	SOLID SODDING	SQ.YD.	356		
219-A	WATERING	M/GALLON	7		
220-A	INSECT PEST CONTROL	ACRE	39		
907-221-A	PORTLAND CEMENT CONCRETE PAVED DITCH	CU.YD.	72		4
907-223-A	MOWING	ACRE	0		
234-A	TEMPORARY SILT FENCE	LIN. FT.	19,260		
235-A	TEMPORARY EROSION CHECKS	BALE	100		2
907-239-A	TEMPORARY SLOPE DRAIN	LIN. FT.	100		2
	***** SUBBASE & BASE ITEMS *****				
304-A	GRANULAR MATERIAL (LVM)(CLASS 9, GROUP C)	CU.YD.	3800		7
907-304-I	CRUSHED STONE BASE (SIZE 825)	TON	24,300		
907-309-A	CRUSHED STONE DRAINAGE LAYER	SQ.YD.	82,700		
907-311-A	PROCESSING LIME AND FLY ASH TREATED COURSE, (6" THICK)	SQ.YD.	82,700		
907-311-B	LIME	TON	670		
907-311-C	FLY ASH (CLASS C OR F)	TON	2700		

1. EASTBOUND (BR. NO. 15.5)
STA. 788+94.96 TO 824+03.90
•40@49';•8@63';••10@340'(100-140-100);•8@63';•5@49'

WESTBOUND (BR. NO. 15.5)
STA. 788+71.85 TO 824+29.74
•39@49';•8@63';••10@340'(100-140-100);•8@63';•7@49'

• PRESTRESSED CONCRETE
•• PLATE GIRDER

2. ESTIMATED TO BE USED AS DIRECTED BY THE ENGINEER

3. INCLUDES POSTS, BRIDGE END SECTIONS & CABLE ANCHORAGE. GUARD RAIL TO BE STOCKPILED AS DIRECTED FOR PICKUP BY STATE FORCES.

4. ESTIMATED QUANTITY. INCLUDES 12 CU. YDS. FOR PAVED FLUMES AT THE BRIDGE ENDS (SEE WK. SH. NO. PF-1) AND 38 CU.YDS. FOR EDGE DRAIN OUTLETS.

5. INCLUDES 17,500 CU.YDS. FOR SHOULDER MATERIAL (50% SHRINKAGE FACTOR ESTIMATED.)

6. INCLUDES 2258 S.Y. OF TEMPORARY WIDENING AT S.E. RAMP AT SR43 INTERCHANGE AND 18,324 S.Y. OF TEMPORARY WIDENING ON THE MAINLINE.

7. INCLUDES 50 % SHRINKAGE FACTOR.

PLAN
ROADWAY DESIGN DIVISION C.A.D.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

DATE	REVISION	BY
4/27/06	ADDED PAY ITEM	AMK
3/8/06	REPLACED PAY ITEM	AMK

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

SUMMARY OF QUANTITIES (ROADWAY)

PROJECT NO.: BR-0010-01(073)
COUNTY: HANCOCK
FILENAME: SQ.DGN
DESIGN TEAM KOPF CHECKED DATE

WORKING NUMBER SQ-1
SHEET NUMBER 6

STATE	PROJECT NO.
MISS.	NH-0021-01(104)PH2

PAY ITEM NO.	PAY ITEM	UNIT	QUANTITIES PRELIMINARY	QUANTITIES FINAL
	BRIDGE SUMMARY			
907-501-K	Transverse Grooving	S.Y.	5565	
801-A	Foundation Excavation For Bridges	C.Y.	2070	
803-A	Test Piles	Each	10	
803-B	Loading Test	Each	10	
803-G	HP14X73 Steel Piling	L.F.	7510	
804-A	Bridge Concrete Class "AA"	C.Y.	2921	
805-A	Reinforcement	Lb.	544,752	
907-810-A	Structural Steel (A709)	Lb.	3,244,062	
813-A	Concrete Railing	L.F.	2891	
815-D	Concrete Slope Paving	C.Y.	1120	
907-854-A	Floating (Pot Type) Bearings Assemblies, Bridge "A"	L.S.	L.S.	

PLAN
HIGHWAY DESIGN DIVISION C.A.D.D. SECTION
HIGHWAY DESIGN DIVISION TRANSPORTATION

DATE	REVISION	BY
05/09	TRANSVERSE GROOVING PAY QTY.	CM

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

SUMMARY OF QUANTITIES
(BRIDGE ITEMS)

102650
NH-0021-01(104)PH2

MARSHALL COUNTY

FILENAME:
DESIGN TEAM CHECKED RHM DATE

WORKING NUMBER SQ-5
SHEET NUMBER 7

STATE	PROJECT NO.
MISS.	SDP-0002-02(042)

PIPE CULVERT DRAINAGE STRUCTURES

WK. SH. NO.	STATION	ID NUMBER	TYPE	PIPE CULVERTS 18"	24"	30"	36"	48"	22"x13"	51"x31"	END SECTIONS 18"	24"	30"	36"	48"	22"x13"	51"x31"	SIZE L	W	H	OPENINGS	DRAWINGS REQ'D.	CLASS "B" CONC.	REINF. STEEL	GRATING	CASTINGS	PAVED APRON	STRUCTURE EXCAVATION CU YDS EXCAV	EST. DEPTH	GEOTEXTILE	FILTER MATERIAL TYPE "A"	TYPE "B"	COVER	REMARKS
7	240+50		CONC.	112							1									8'	1 @ 18"	MI-1,FES	1.894	103	250		0.532	29	2				10'	9 TON RIPRAP LT
7	243+50		CONC.					408							4					10'	2 @ 48"	MI-1A,FES	2.775	329	250		0.532	94	1				7.5'	DBL LN (204 EA.)
9	271+00		CONC.						80							2						FES	0.126					27	2				12'	
9	276+00		CONC.							200							2					FES,BC-1	0.268					49	1				11.4'	
9	276+00		Ⓑ						12											MIN.	1@22"x13"	MI-4	0.582	50	500		1.203							STUB IN 51x31 BK
UNIT				L.F.	L.F.	L.F.	L.F.	L.F.	L.F.	L.F.	EA.	EA.	EA.	EA.	EA.	EA.	EA.						C.Y.	LB.	LB.	LB.	C.Y.	C.Y.						
CONCRETE THIS SHEET				324	312	296	80	496	80	200	3	8	4	2	6	2	2						13.522	934	1750	79	3.863	563						
ALTERNATE Ⓐ THIS SHEET				40	40																													
ALTERNATE Ⓑ THIS SHEET									12																									
UNITS				L.F.	L.F.	L.F.	L.F.	L.F.	L.F.	L.F.	EA.	EA.	EA.	EA.	EA.	EA.	EA.						C.Y.	LB.	LB.	LB.	C.Y.	C.Y.						
CONCRETE TOTALS				444	460	536	80	496	80	200	4	9	4	2	6	2	2						40.830	3214	1877	790	4.395	1090						
ALTERNATE Ⓐ TOTALS				64	64	272																												
ALTERNATE Ⓑ TOTALS									12																									

BOX CULVERTS REQUIRED

WORK. NO.	STATION	SIZE	LENGTH	STANDARD DRAWINGS REQUIRED	CLASS "B" CONCRETE	REINF. STEEL	STRUC. EXCAV. EST. DEPTH	CUBIC YARDS	COVER	SELECT MAT'L.	REMARKS
7	248+25	4'X4'	16'	IBJL-1,ICJ-1,IBS-4-2W	9.45	1,485	1.5	17.80	3.25'	4.81	
8	255+75	6'X6'	14'	IBJL-1,ICJ-1,IBS-6-2W	18.66	2,625	1.5	23.97	5.78'	7.82	
9	275+50	10'X6'	86'	IBJL-1,ICJ-1,IBS-6-2W	112.95	23,636	1.5	105.38	2.50'	34.68	
UNITS					CU. YD	LB.		CU. YD		CU. YD	
TOTALS					141.06	27,746		147.15		47.31	

GUARD RAIL REQUIRED

WORK NO.	STATION	W-BEAM	STATE STD. (INSTAL.)	TERMINAL SECTION	TYPE "1" CABLE ANCHOR.	TYPE "I" BR. END SECTION	TYPE "H" BR. END SECTION	WHITE DELIN.	YELLOW DELIN.	REMARKS
7	241+45.95	175	GR-4C	1		1		8		BR. ABUT. RT.
8	252+31.27	175	GR-4C	1		1		8		BR. ABUT. RT.
UNITS		LF		EACH		EACH		EACH		
TOTALS		350		2		2		16		

ESTIMATED EARTHWORK QUANTITIES

WK. SH. NUMBER	CUT	FILL	BORROW (LVM)	BORROW (FME)	UNCLASSIFIED EXCAVATION	EXCAVATION EXCESS	SURPLUS	REMARKS
7	12,942	38,148			12,942			
8			975	4,473		4,473		
9	45,145	3,253			45,145			
UNITS	CU. YD	CU. YD	CU. YD	CU. YD	CU. YD	CU. YD		
TOTALS	58,087	41,401	975	4,473	58,087	4,473		

BOX BRIDGES REQUIRED

WORK. NO.	STATION	SIZE	LENGTH ALONG ℄	STANDARD DRAWINGS REQUIRED	CLASS "B" CONCRETE	LENGTH	REINF. STEEL	STRUC. EXCAV. EST. DEPTH	CUBIC YARDS	COVER	SELECT MAT'L.	REMARKS
7	250+50	DBL. 10'X8'	22.1250'	IBJL-1, ICJ-1, IBD-8-2W & IWD-3	192.06	73'	22,825	1.5	145.05	2.79	47.63	EXTEND BOX
8	257+25	DBL. 8'X8'	25.2791'	IBJL-1, ICJS-1, IBD-8-2W, IWD-3, ISK-45-3-W, & IBDM-3W	214.78	118'	38,885	1.5	185.73	1.95	61.03	EXTEND BOX SKEWED 45 DEG. RT. FWD. (1)
9	271+75	DBL. 12'X10'	26.5280'	IBJL-1, ICJS-1, IBD-10-2W IWD-3, ISK-15-3W, & IBDM-3W	279.11	92'	50,681	1.5	208.72	1.13	67.44	EXTEND BOX SKEWED 10 DEG. LT. FWD. (1)
UNITS					CU. YD		LB.		CU. YD		CU. YD	
TOTALS					685.95		112,391		539.50		176.10	

(1) MODIFIED FOR LOW COVER.

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

ESTIMATED QUANTITIES (ROADWAY)

PROJECT NO. SDP-0002-02(042)

COUNTY : JEFFERSON DAVIS

FILENAME:

DESIGN TEAM CHECKED DATE

REVISION BY DATE

WORKING NUMBER EQ-1

SHEET NUMBER 8

PLAN ROADWAY DESIGN DIVISION C.A.D.D. SECTION MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	NH-0021-01(104)PH2

BRIDGE	BEGINNING STATION	SPANS-SIZE	OVERALL LENGTH	ITEM	Transverse Grooving		Foundation Excavation		Test Piles		Loading Tests		HP14X73 Steel Piling		Class "AA" Bridge Concrete		Reinforce-ment		Structural Steel (A709)		Concrete Railing		Concrete Slope Paving		Floating Pot Bearings	
					S.Y.		C.Y.		Each		Each		L.F.		C.Y.		Lb.		Lb.		L.F.		C.Y.		L.S.	
"A" WEST	580+40.32	170'-224'-200'- 140' WPG	737'-5"	Spans	2438	3									661	04	142844		1475856		1477	62	560	0	L.S.	
				End Bents			800	0	2		2		1105	0	92	36	13842									
				Int. Bents					3		3		2070	0	499	22	79400									
				Total	2438	3	800	0	5		5		3175	0	1252	62	236086		1475856		1477	62	560	0	L.S.	
"A" EAST	582+32.53	155'-222'-194'- 135' WPG	709'-5"	Spans	3126	7									824	27	178200		1768206		1413	66	560	0	L.S.	
				End Bents			1270	0	2		2		1275	0	109	36	15700									
				Int. Bents					3		3		3060	0	735	10	114766									
				Total	3126	7	1270	0	5		5		4335	0	1668	73	308666		1768206		1413	66	560	0	L.S.	
				Totals	5565.0		2070.0		10		10		7510.0		2921.35		544,752		3,244,062		2891.28		1120.0		LS	

STRUCTURAL STEEL (GRADE 36 - 294,188 Lb.)
(GRADE 50 - 2,949,874 Lb.)

PLAN
HIGHWAY DESIGN DIVISION G.A.D.B. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

DATE	REVISION	BY

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

ESTIMATED QUANTITIES
(BRIDGE ITEMS)

102650
NH-0021-01(104)PH2

MARSHALL COUNTY

WORKING NUMBER EQ-2

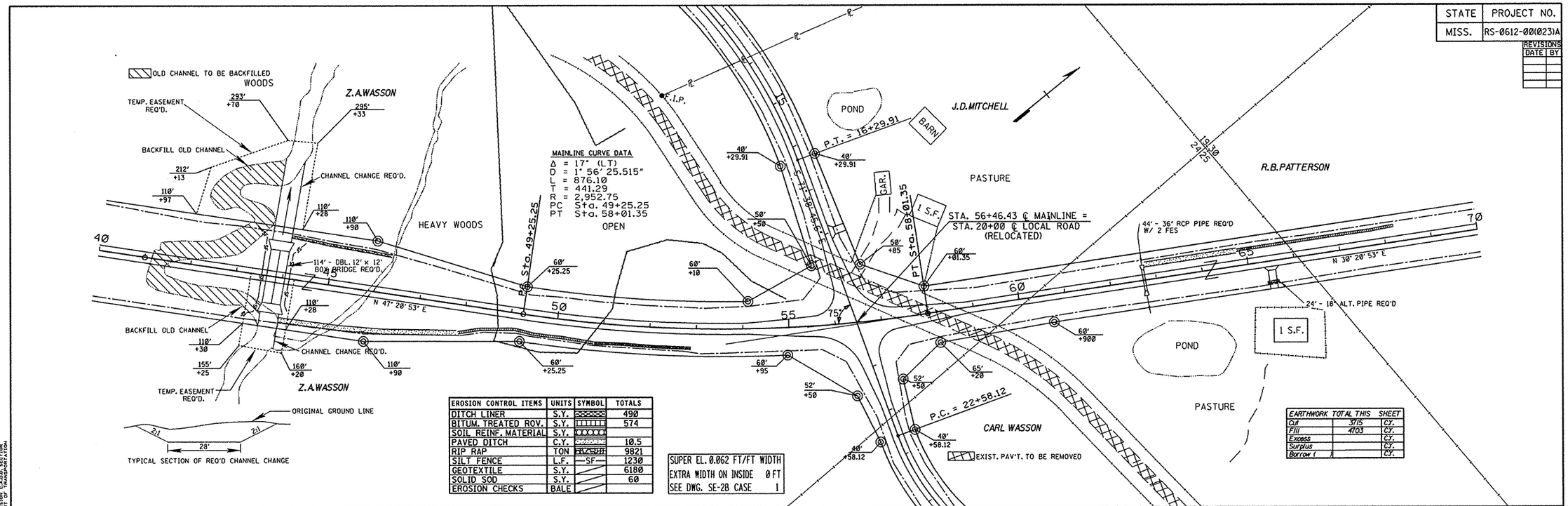
FILENAME:

DESIGN TEAM CHECKED RHM DATE

SHEET NUMBER 9

STATE	PROJECT NO.
MISS.	RS-0612-00(023)A

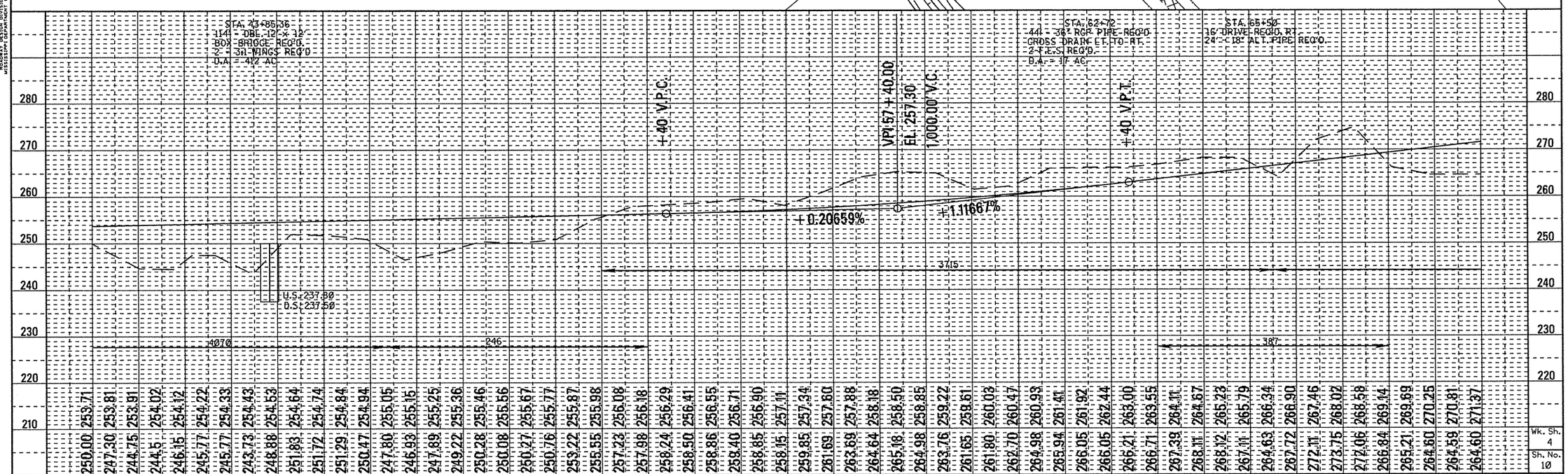
REVISIONS	
DATE	BY



EROSION CONTROL ITEMS	UNITS	SYMBOL	TOTALS
DITCH LINER	S.Y.		490
BITUM. TREATED ROV.	S.Y.		574
SOIL REINF. MATERIAL	S.Y.		
PAVED DITCH	C.Y.		10.5
RIP RAP	TON		982
SILT FENCE	L.F.	SF	1230
GEOTEXTILE	S.Y.		6180
SOLID SOD	S.Y.		60
EROSION CHECKS	BALE		

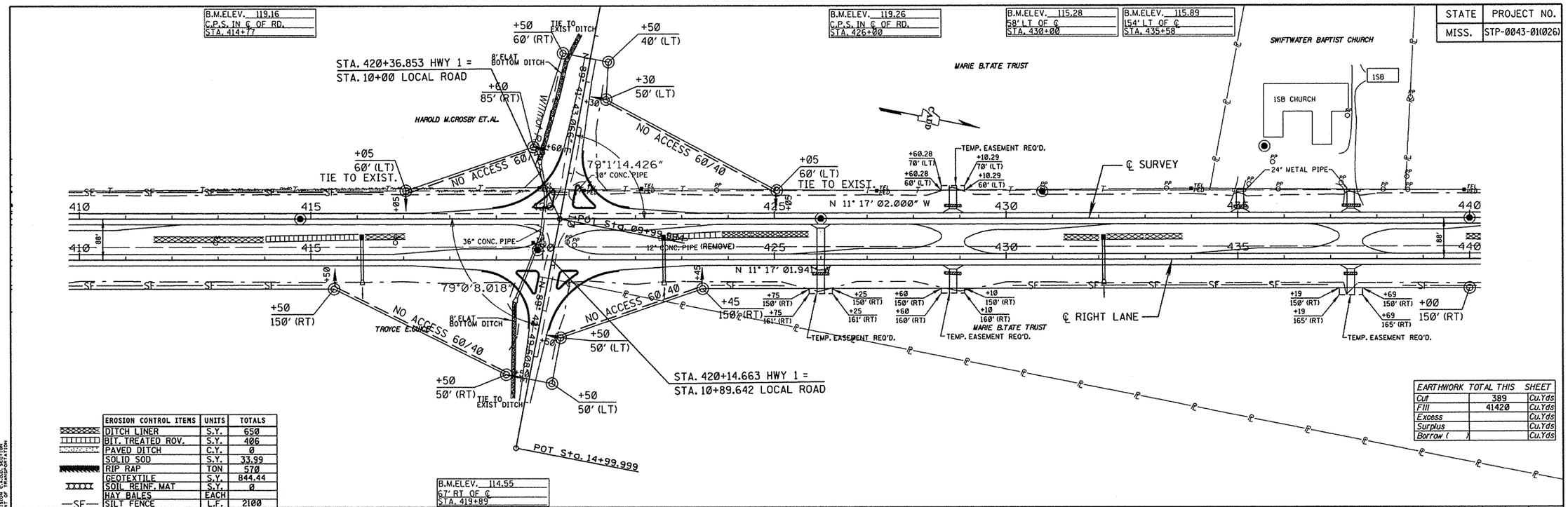
SUPER EL. 0.062 FT/FT WIDTH	
EXTRA WIDTH ON INSIDE	0 FT
SEE DWG. SE-2B CASE	1

EARTHWORK	TOTAL THIS	SHEET
Cut	3715	C.Y.
Fill	4703	C.Y.
Excess		C.Y.
Surplus		C.Y.
Borrow ( )		C.Y.



PLAN-PROFILE
PROPERTY OF THE STATE OF MISSISSIPPI
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	STP-0043-01(026)



EROSION CONTROL ITEMS	UNITS	TOTALS
DITCH LINER	S.Y.	650
BIT. TREATED ROV.	S.Y.	406
PAVED DITCH	C.Y.	0
SOLID SOD	S.Y.	33.99
RIP RAP	TON	570
GEOTEXTILE	S.Y.	844.44
SOIL REINF. MAT	S.Y.	0
HAY BALES	EACH	0
SILT FENCE	L.F.	2100

EARTHWORK	TOTAL THIS	SHEET
Cut	389	Cu.Yds
Fill	41420	Cu.Yds
Excess		Cu.Yds
Surplus		Cu.Yds
Borrow ( )		Cu.Yds

B.M.ELEV. 114.55
67' RT OF ℄
STA. 419+89

STA. 416+00 RT. SD.
80' - 18" RCP - REQ'D.
1 - 18" FES REQ'D.
1 - MI-4 REQ'D.

STA. 420+26
CROSSOVER REQ'D.

STA. 420+05
EXTEND EXISTING 36" RCP W/
72'-36" RCP LT. 20° SKEW RT. FWD.
120'-36" RCP RT. 10° SKEW LT. FWD.
2 - PC-1 REQ'D.
8A.13 AC.
2 - 36" FE-1 REQ'D.

STA. 422+50 RT. SD.
80' - 18" RCP - REQ'D.
1 - 18" FES REQ'D.
1 - MI-4 REQ'D.

STA. 426+00
RAMP REQ'D. RT.
36" - 24" ALT. PIPE REQ'D.

STA. 426+00
TEMP. RAMP REQ'D. RT. (DRY)

STA. 428+85
CROSSOVER REQ'D.

STA. 428+85
RAMP REQ'D. RT.
36" - 24" ALT. PIPE REQ'D.

STA. 428+85
RAMP REQ'D. LT.
36" - 24" ALT. PIPE REQ'D.

STA. 432+00 RT. SD.
84" - 18" RCP - REQ'D.
1 - 18" FES REQ'D.
1 - MI-4 REQ'D.

STA. 435+05
RAMP REQ'D. LT.
28" - 24" ALT. PIPE REQ'D.

STA. 437+44
RAMP REQ'D. RT.
40" - 24" ALT. PIPE REQ'D.

STA. 437+44
RAMP REQ'D. LT.
28" - 24" ALT. PIPE REQ'D.

STA. 437+44
CROSSOVER REQ'D.

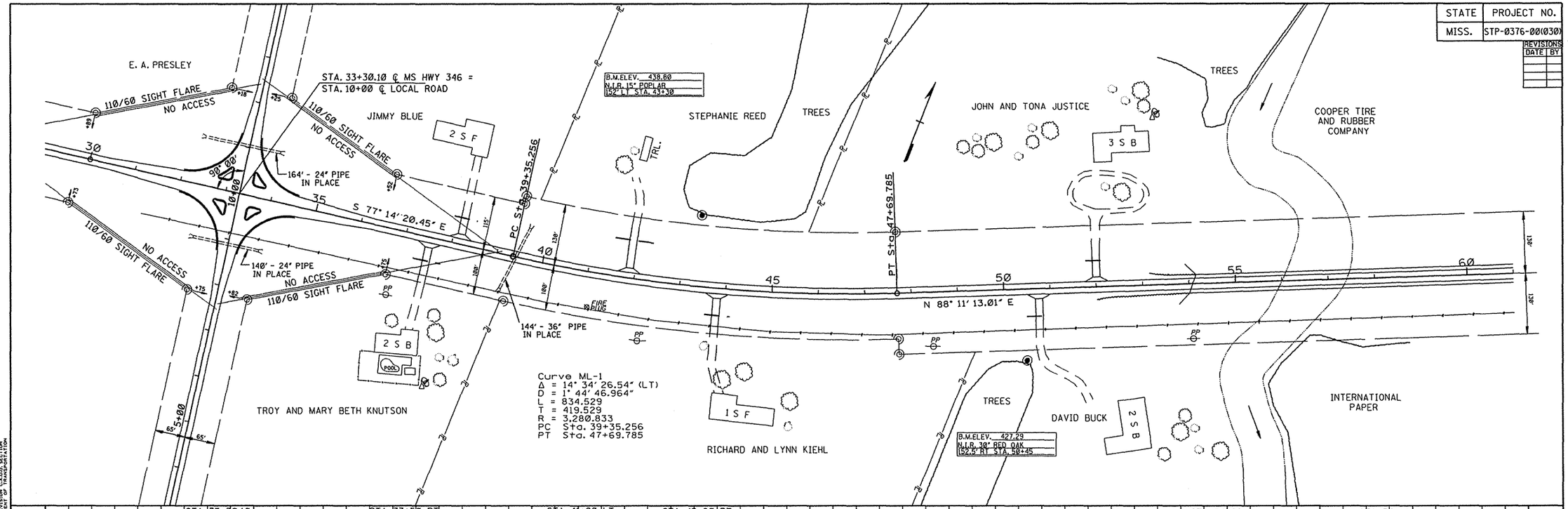
VPI 418+00
VPI 420+00
EL. 119.600
200' V.C.
VPI 421+00
+0.01769%
+0.05404%
USFL 113.548
DSFL 113.428
USFL 114.238
DSFL 113.878
USFL 113.452
DSFL 113.810
PROP. RIGHT LANE
EXIST. LEFT LANE
EXIST. RIGHT LANE
CUT
FILL
102 C.Y.
13584 C.Y.
246 C.Y.
11232 C.Y.
39 C.Y.
18604 C.Y.
EXIST. PROP. RIGHT LANE

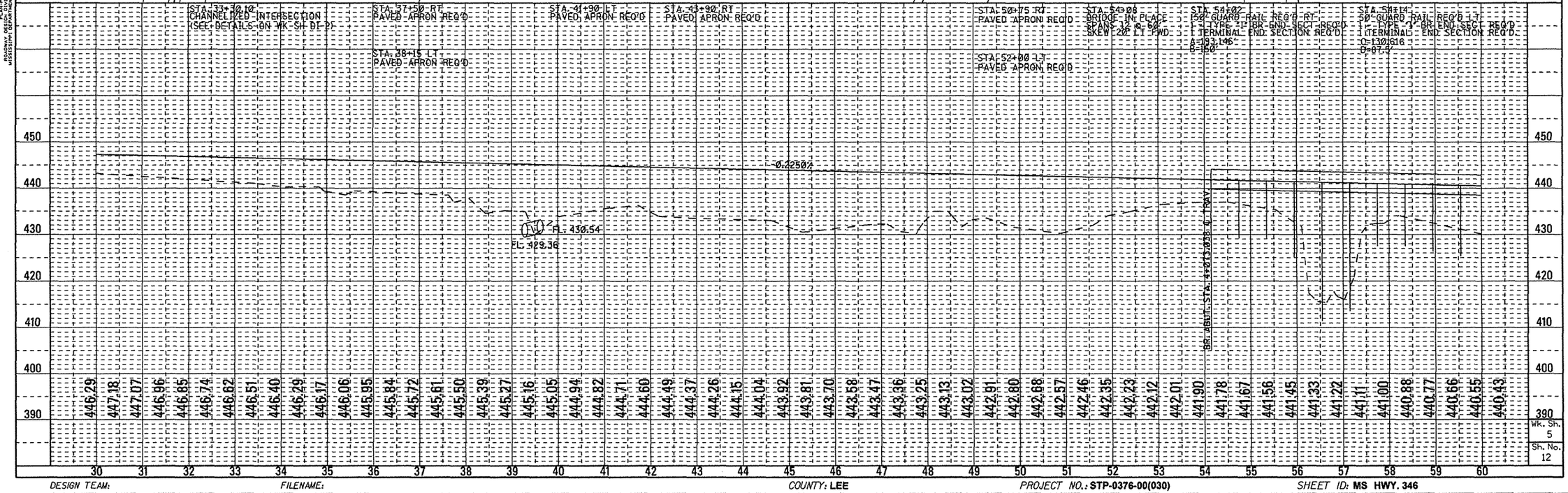
Wk. Sh. 7
Sh. No. 11

DESIGN TEAM: TURNER FILENAME: WK7.DGN COUNTY: WASHINGTON PROJECT NO.: STP-0043-01(026) SHEET ID: HIGHWAY 1

STATE	PROJECT NO.
MISS.	STP-0376-00(030)

REVISIONS	
DATE	BY



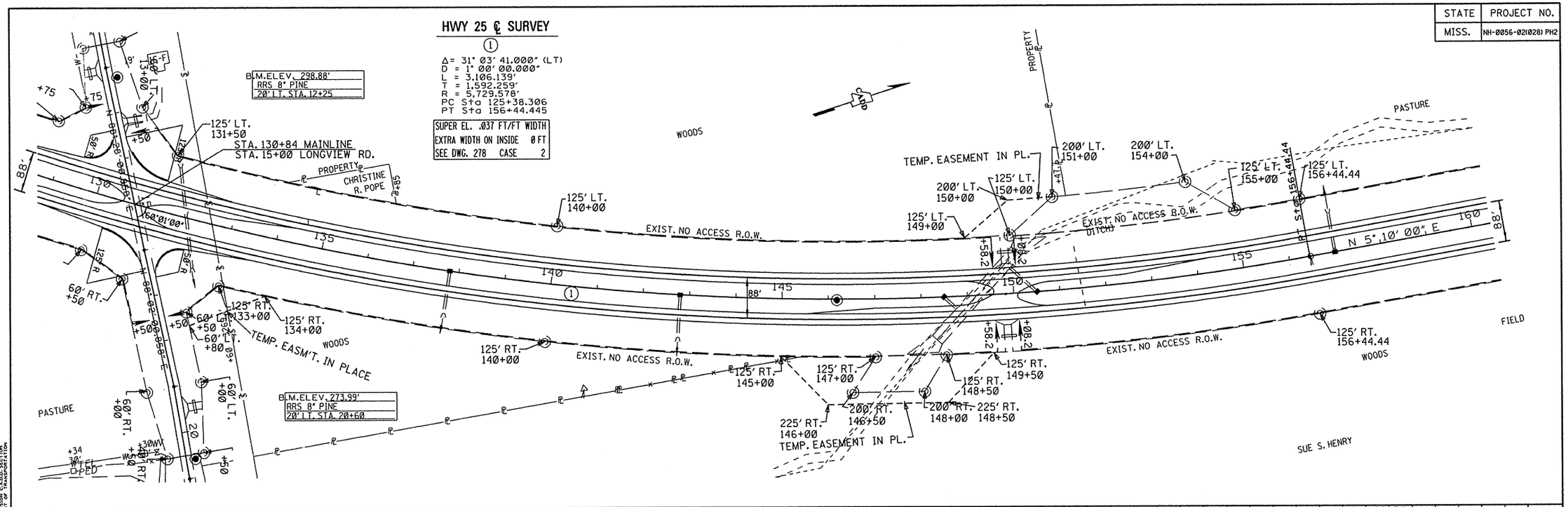


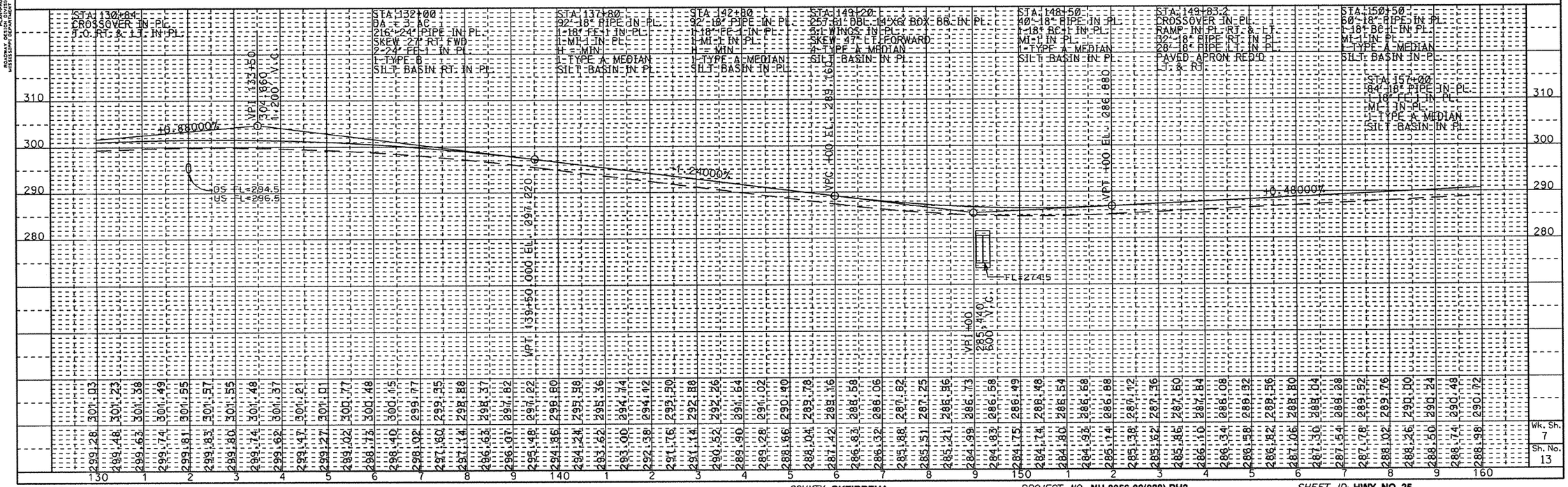
Wk. Sh. 5
Sh. No. 12

PLAN-PROFILE
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

DESIGN TEAM: FILENAME: COUNTY: LEE PROJECT NO.: STP-0376-00(030) SHEET ID: MS HWY. 346

STATE	PROJECT NO.
MISS.	NH-0056-02(028) PH2

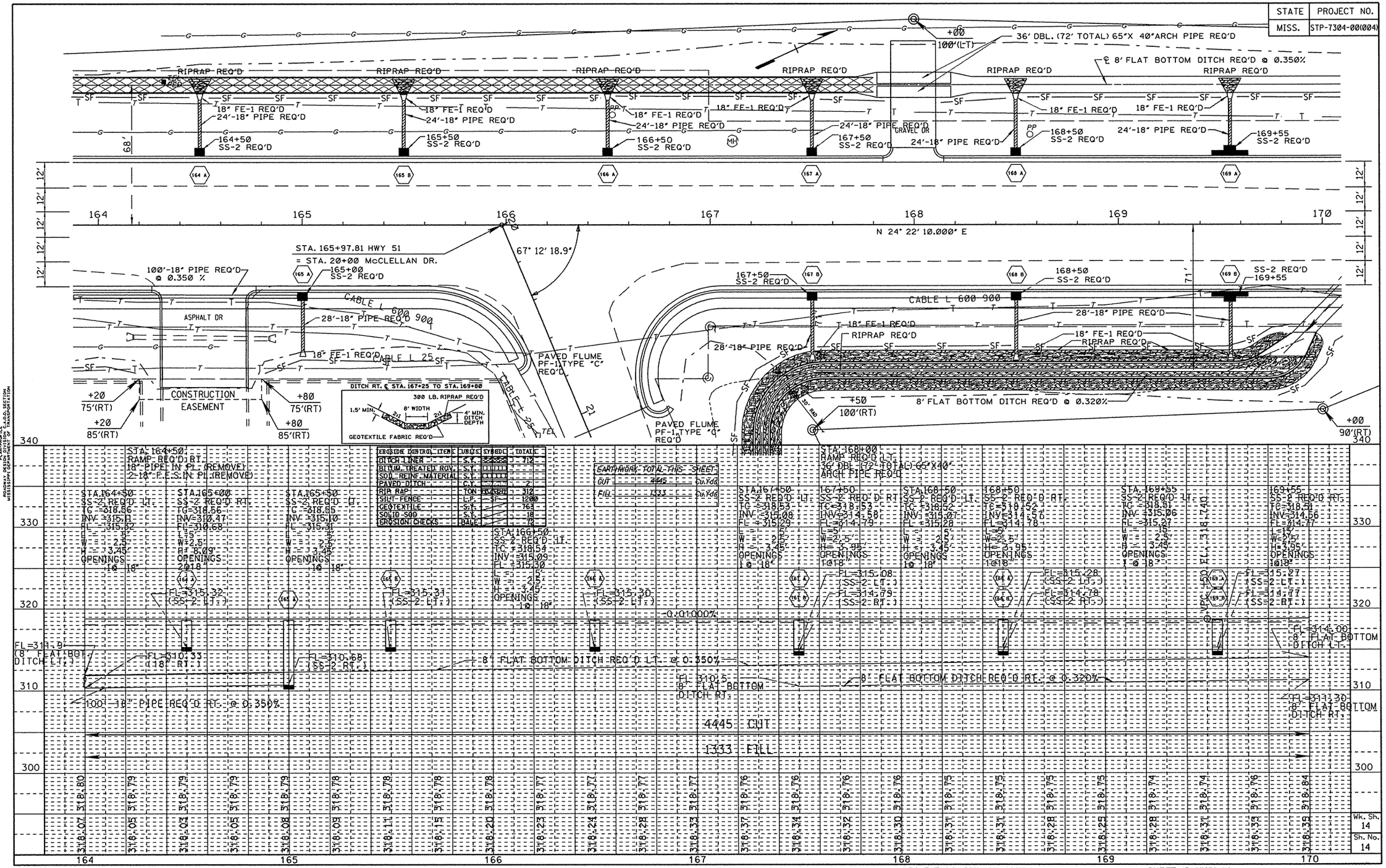




Wk. Sh.	Sh. No.
7	13

DESIGN TEAM: McLendon FILENAME: SH7.DGN COUNTY: OKTIBBEHA PROJECT NO.: NH-0056-02(028) PH2 SHEET ID: HWY. NO. 25

STATE	PROJECT NO.
MISS.	STP-7304-00(004)



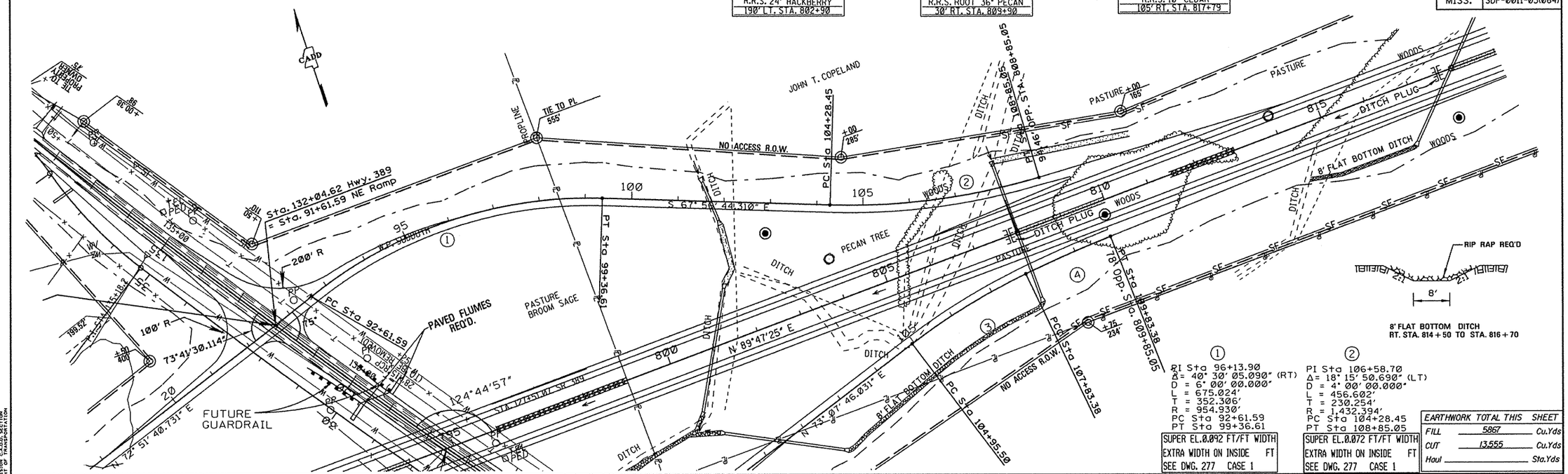
DESIGN TEAM: FILENAME: COUNTY: MADISON PROJECT NO.: STP-7304-00(004) SHEET ID: HWY 51

STATE	PROJECT NO.
MISS.	SDP-0011-03(064)

B.M.ELEV. 292.07'
R.R.S. 24" HACKBERRY
190' LT. STA. 802+90

B.M.ELEV. 297.93'
R.R.S. ROOT 36" PECAN
30' RT. STA. 809+90

B.M.ELEV. 327.63'
R.R.S. 10" CEDAR
105' RT. STA. 817+79



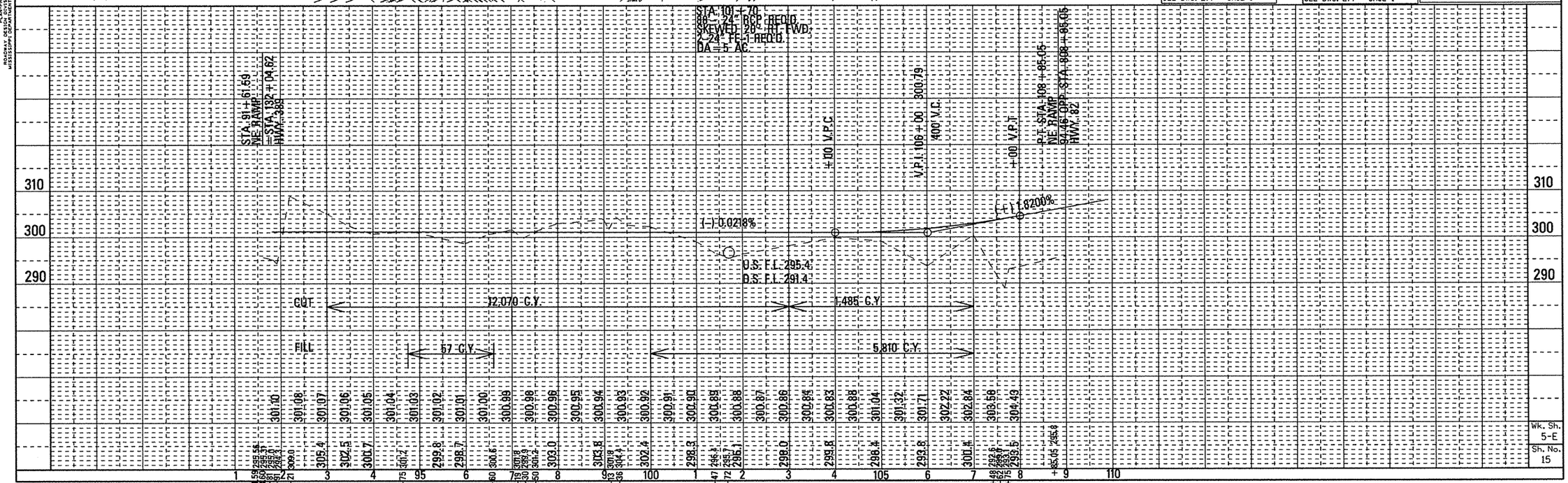
① PI Sta 96+13.90
Δ = 40° 30' 05.090" (RT)
D = 6° 00' 00.000"
L = 675.024'
T = 352.306'
R = 954.930'
PC Sta 92+61.59
PT Sta 99+36.61

SUPER EL.0.092 FT/FT WIDTH
EXTRA WIDTH ON INSIDE FT
SEE DWG. 277 CASE 1

② PI Sta 106+58.70
Δ = 18° 15' 50.690" (LT)
D = 4° 00' 00.000"
L = 456.602'
T = 230.254'
R = 1,432.394'
PC Sta 104+28.45
PT Sta 108+85.05

SUPER EL.0.072 FT/FT WIDTH
EXTRA WIDTH ON INSIDE FT
SEE DWG. 277 CASE 1

EARTHWORK TOTAL THIS SHEET		
FILL	5867	Cu.Yds
CUT	13,555	Cu.Yds
Haul		Sta.Yds



Wk. Sh.	Sh. No.
5-E	15

DESIGN TEAM: SEAL FILENAME: WK5E.DGN COUNTY: OKTIBBEHA PROJECT NO.: SDP-0011-03(064) SHEET ID: NE RAMP HWY. 389

PLAN-PROFILE
PROJECT DEVELOPMENT & R.O.W. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	ISP-0055-02(178)

Curve SWRP-1
Δ = 6° 11' 39.186" (LT)
D = 2° 00' 00.000"
L = 309.711
T = 155.006
R = 2,864.789
PC Sta. 744+33.155
PCC Sta. 747+42.866

Curve SWRP-2
Δ = 25° 12' 45.786" (LT)
D = 3° 58' 29.237"
L = 634.316
T = 322.377
R = 1,441.480
PCC Sta. 747+42.866
PT Sta. 753+77.182

Curve SWRP-3
Δ = 59° 09' 08.024" (LT)
D = 13° 19' 28.560"
L = 443.932
T = 244.037
R = 430.000
PC Sta. 765+57.269
PT Sta. 770+01.201

Curve I55RT-7
Δ = 1° 51' 46.204" (RT)
D = 0° 20' 00.000"
L = 558.850
T = 279.450
R = 17,188.734
PC Sta. 745+81.066
PT Sta. 751+39.916

Curve SWLP-1
Δ = 52° 25' 25.010" (RT)
D = 14° 19' 26.202"
L = 365.986
T = 196.927
R = 400.000
PC Sta. 763+30.610
PCC Sta. 766+96.596

Curve SWLP-2
Δ = 160° 04' 43.430" (RT)
D = 20° 27' 46.002"
L = 782.292
T = 1,594.364
R = 280.000
PCC Sta. 766+96.596
PCC Sta. 774+78.888

Curve SWLP-3
Δ = 59° 47' 36.300" (RT)
D = 14° 19' 26.203"
L = 417.437
T = 229.980
R = 400.000
PCC Sta. 774+78.888
PT Sta. 778+96.325

Curve NWRP-1
Δ = 60° 54' 35.716" (LT)
D = 13° 19' 28.560"
L = 457.124
T = 252.834
R = 430.000
PC Sta. 763+53.557
PT Sta. 768+10.681

Curve NWRP-2
Δ = 36° 46' 39.102" (LT)
D = 5° 59' 59.992"
L = 612.959
T = 317.455
R = 954.930
PC Sta. 776+89.224
PT Sta. 783+02.183

Curve NWLP-1
Δ = 57° 09' 30.110" (RT)
D = 14° 19' 26.202"
L = 399.041
T = 217.899
R = 400.000
PC Sta. 754+96.318
PCC Sta. 758+95.358

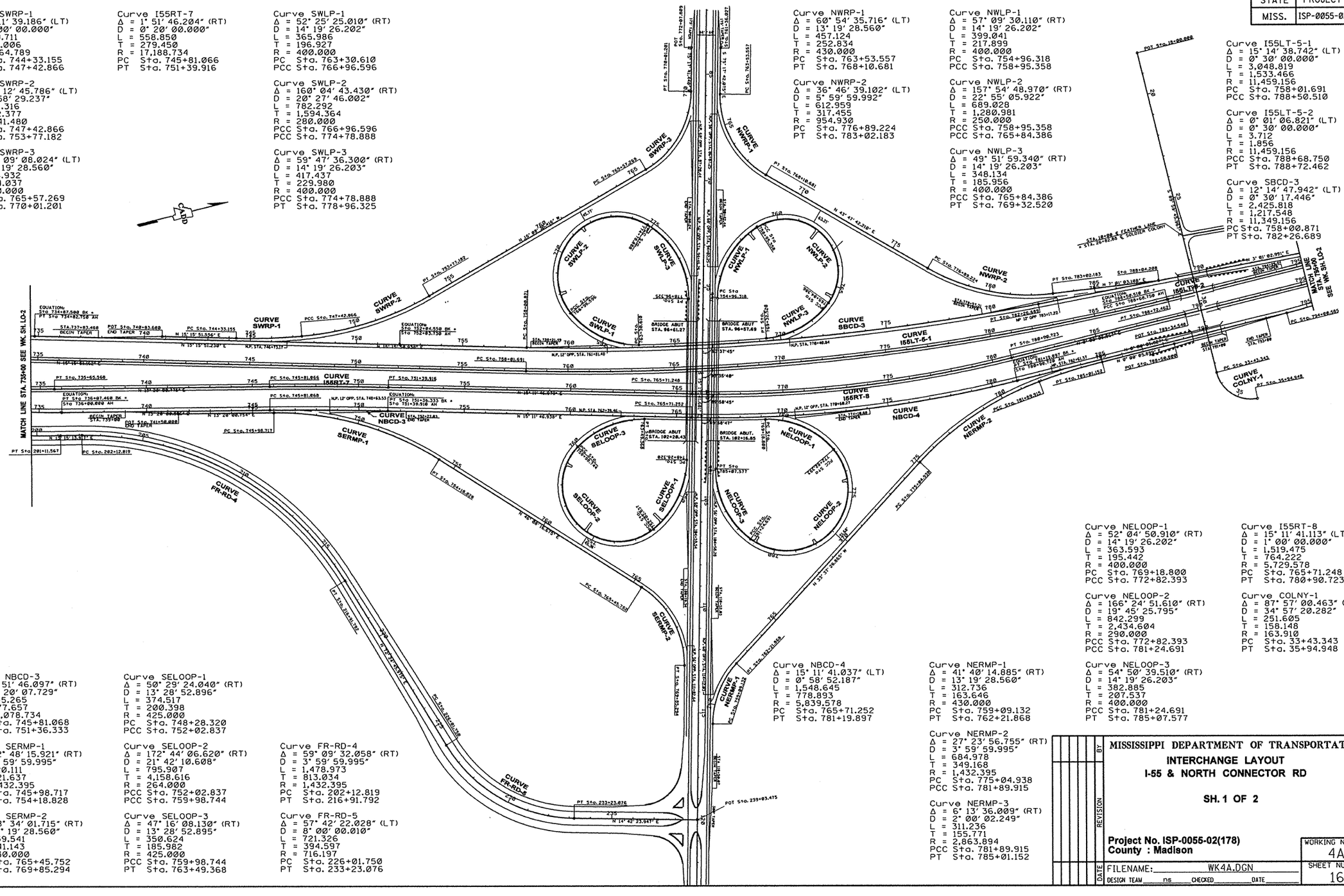
Curve NWLP-2
Δ = 157° 54' 48.970" (RT)
D = 22° 55' 05.922"
L = 689.028
T = 1,280.981
R = 250.000
PCC Sta. 758+95.358
PCC Sta. 765+84.386

Curve NWLP-3
Δ = 49° 51' 59.340" (RT)
D = 14° 19' 26.203"
L = 348.134
T = 185.956
R = 400.000
PCC Sta. 765+84.386
PT Sta. 769+32.520

Curve I55LT-5-1
Δ = 15° 14' 38.742" (LT)
D = 0° 30' 00.000"
L = 3,048.819
T = 1,533.466
R = 11,459.156
PC Sta. 758+01.691
PCC Sta. 788+50.510

Curve I55LT-5-2
Δ = 0° 01' 06.821" (LT)
D = 0° 30' 00.000"
L = 3.712
T = 1.856
R = 11,459.156
PCC Sta. 788+68.750
PT Sta. 788+72.462

Curve SBCD-3
Δ = 12° 14' 47.942" (LT)
D = 0° 30' 17.446"
L = 2,425.818
T = 1,217.548
R = 11,349.156
PC Sta. 758+00.871
PT Sta. 782+26.689



Curve NELOOP-1
Δ = 52° 04' 50.910" (RT)
D = 14° 19' 26.202"
L = 363.593
T = 195.442
R = 400.000
PC Sta. 769+18.800
PCC Sta. 772+82.393

Curve NELOOP-2
Δ = 166° 24' 51.610" (RT)
D = 19° 45' 25.795"
L = 842.299
T = 2,434.604
R = 290.000
PCC Sta. 772+82.393
PCC Sta. 781+24.691

Curve I55RT-8
Δ = 15° 11' 41.113" (LT)
D = 1° 00' 00.000"
L = 1,519.475
T = 764.222
R = 5,729.578
PC Sta. 765+71.248
PT Sta. 780+90.723

Curve COLNY-1
Δ = 87° 57' 00.463" (LT)
D = 34° 57' 20.282"
L = 251.605
T = 158.148
R = 163.910
PC Sta. 33+43.343
PT Sta. 35+94.948

Curve NBCD-3
Δ = 1° 51' 46.097" (RT)
D = 0° 20' 07.729"
L = 555.265
T = 277.657
R = 17,078.734
PC Sta. 745+81.068
PT Sta. 751+36.333

Curve SERMP-1
Δ = 32° 48' 15.921" (RT)
D = 3° 59' 59.995"
L = 820.111
T = 421.637
R = 1,432.395
PC Sta. 745+98.717
PT Sta. 754+18.828

Curve SERMP-2
Δ = 58° 34' 01.715" (RT)
D = 13° 19' 28.560"
L = 439.541
T = 241.143
R = 430.000
PC Sta. 765+45.752
PT Sta. 769+85.294

Curve SELOOP-1
Δ = 50° 29' 24.040" (RT)
D = 13° 28' 52.896"
L = 374.517
T = 200.398
R = 425.000
PC Sta. 748+28.320
PCC Sta. 752+02.837

Curve SELOOP-2
Δ = 172° 44' 06.620" (RT)
D = 21° 42' 10.608"
L = 795.907
T = 4,158.616
R = 264.000
PCC Sta. 752+02.837
PCC Sta. 759+98.744

Curve SELOOP-3
Δ = 47° 16' 08.130" (RT)
D = 13° 28' 52.895"
L = 350.624
T = 185.982
R = 425.000
PCC Sta. 759+98.744
PT Sta. 763+49.368

Curve FR-RD-4
Δ = 59° 09' 32.058" (RT)
D = 3° 59' 59.995"
L = 1,478.973
T = 813.034
R = 1,432.395
PC Sta. 202+12.819
PT Sta. 216+91.792

Curve FR-RD-5
Δ = 57° 42' 22.028" (LT)
D = 8° 00' 00.010"
L = 721.326
T = 394.597
R = 716.197
PC Sta. 226+01.750
PT Sta. 233+23.076

Curve NBCD-4
Δ = 15° 11' 41.037" (LT)
D = 0° 58' 52.187"
L = 1,548.645
T = 778.893
R = 5,839.578
PC Sta. 765+71.252
PT Sta. 781+19.897

Curve NERMP-1
Δ = 41° 40' 14.885" (RT)
D = 13° 19' 28.560"
L = 312.736
T = 163.646
R = 430.000
PC Sta. 759+09.132
PT Sta. 762+21.868

Curve NERMP-2
Δ = 27° 23' 56.755" (RT)
D = 3° 59' 59.995"
L = 684.978
T = 349.168
R = 1,432.395
PC Sta. 775+04.938
PCC Sta. 781+89.915

Curve NERMP-3
Δ = 6° 13' 36.009" (RT)
D = 2° 00' 02.249"
L = 311.236
T = 155.771
R = 2,863.894
PCC Sta. 781+89.915
PT Sta. 785+01.152

DATE	REVISION	BY

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

INTERCHANGE LAYOUT

I-55 & NORTH CONNECTOR RD

SH. 1 OF 2

Project No. ISP-0055-02(178)
County : Madison

WORKING NUMBER 4A

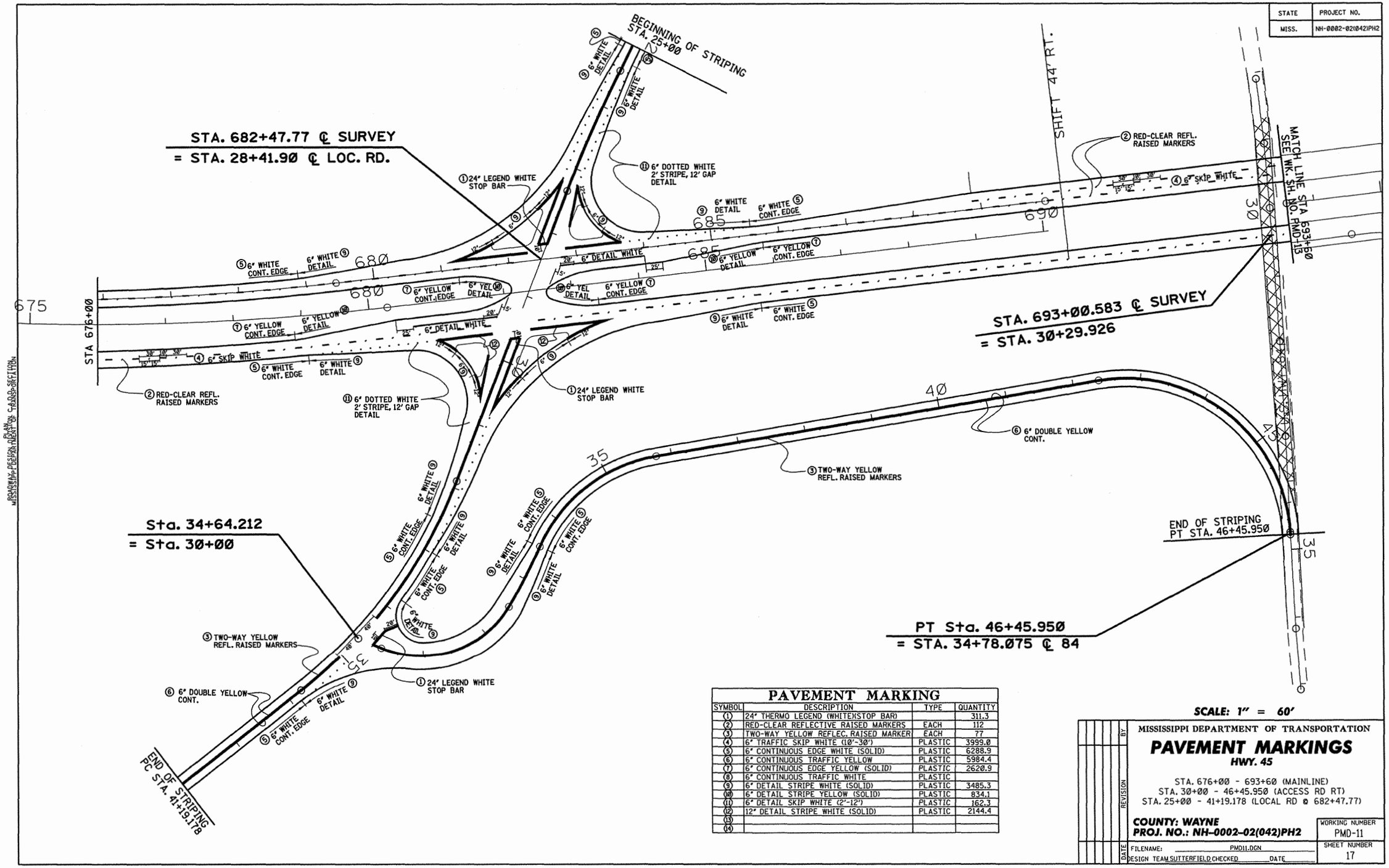
FILENAME: WK4A.DGN

DESIGN TEAM ns CHECKED DATE

SHEET NUMBER 16

PROJECT DESIGN DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	NH-0002-02(042)PH2



PAVEMENT MARKING

SYMBOL	DESCRIPTION	TYPE	QUANTITY
①	24" THERMO LEGEND (WHITE)(STOP BAR)		311.3
②	RED-CLEAR REFLECTIVE RAISED MARKERS	EACH	112
③	TWO-WAY YELLOW REFLEC. RAISED MARKER	EACH	77
④	6" TRAFFIC SKIP WHITE (10'-30')	PLASTIC	3999.0
⑤	6" CONTINUOUS EDGE WHITE (SOLID)	PLASTIC	6288.9
⑥	6" CONTINUOUS TRAFFIC YELLOW	PLASTIC	5984.4
⑦	6" CONTINUOUS EDGE YELLOW (SOLID)	PLASTIC	2620.9
⑧	6" CONTINUOUS TRAFFIC WHITE	PLASTIC	
⑨	6" DETAIL STRIPE WHITE (SOLID)	PLASTIC	3485.3
⑩	6" DETAIL STRIPE YELLOW (SOLID)	PLASTIC	834.1
⑪	6" DETAIL SKIP WHITE (2'-12')	PLASTIC	162.3
⑫	12" DETAIL STRIPE WHITE (SOLID)	PLASTIC	2144.4
⑬			
⑭			

SCALE: 1" = 60'

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

PAVEMENT MARKINGS

HWY. 45

STA. 676+00 - 693+60 (MAINLINE)
STA. 30+00 - 46+45.950 (ACCESS RD RT)
STA. 25+00 - 41+19.178 (LOCAL RD ⊗ 682+47.77)

COUNTY: WAYNE
PROJ. NO.: NH-0002-02(042)PH2

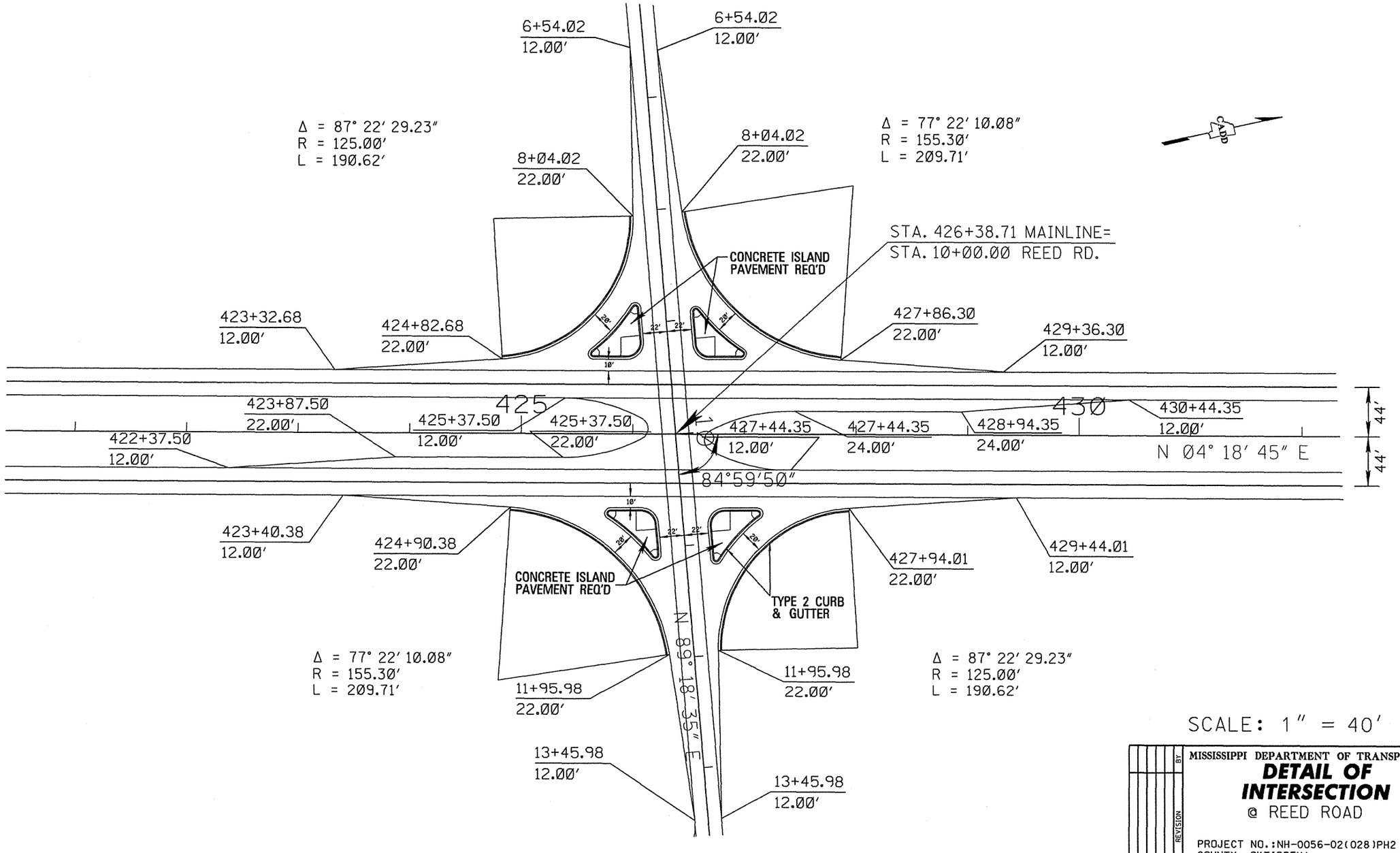
DATE	REVISION	BY

FILENAME: PMD11.DGN
DESIGN TEAM SUTTERFIELD CHECKED _____ DATE _____

WORKING NUMBER: PMD-11
SHEET NUMBER: 17

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	NH-0056-(028)PH2



SCALE: 1" = 40'

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

DETAIL OF INTERSECTION

@ REED ROAD

PROJECT NO.:NH-0056-02(028)PH2
COUNTY: OKTIBBEHA

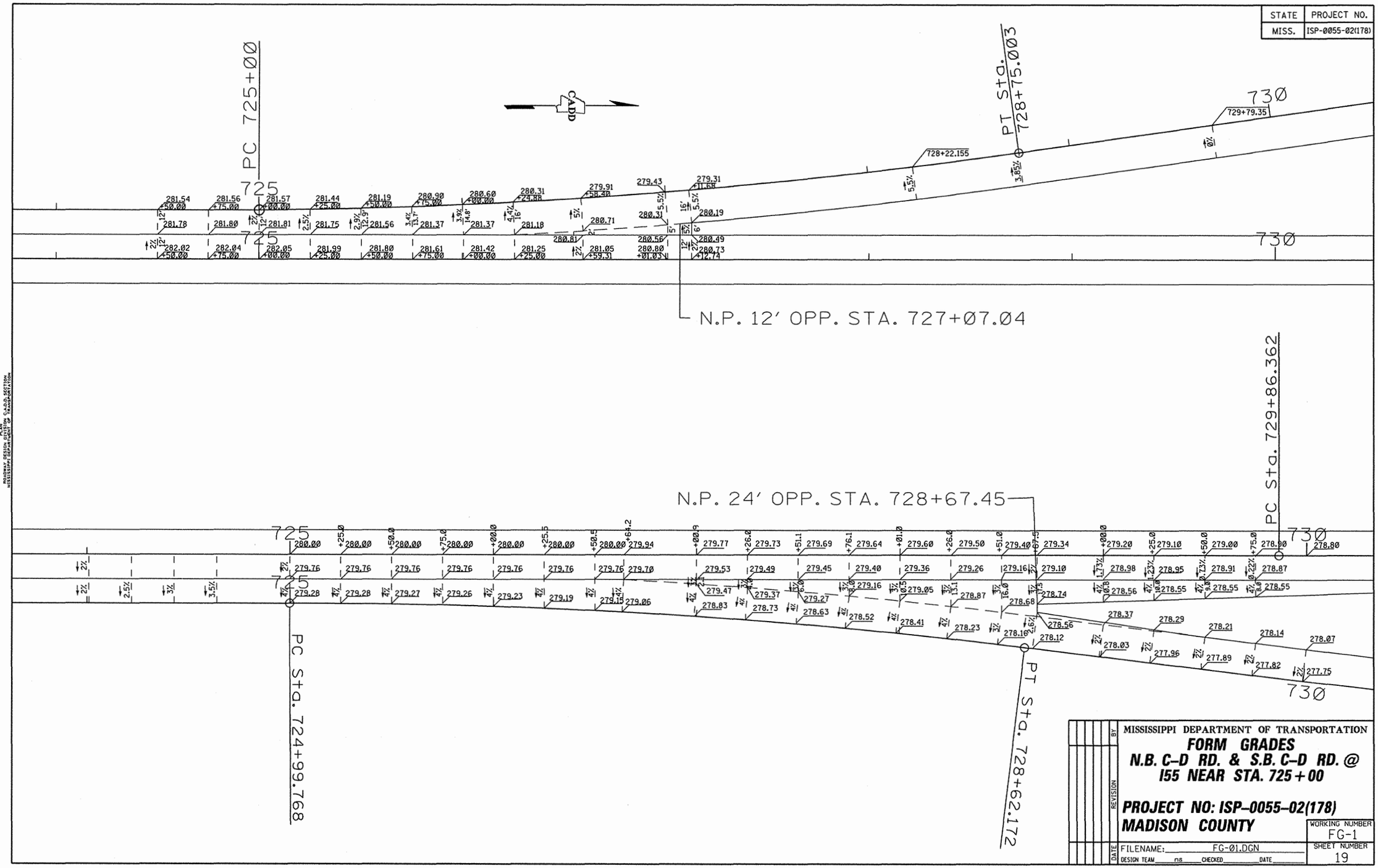
DATE	REVISION	BY

FILENAME: INTREED.DGN
DESIGN TEAM GW CHECKED DATE

WORKING NUMBER 24
SHEET NUMBER 18

PLAN
HIGHWAY DESIGN DIVISION C.A.D.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	ISP-0055-02(178)



MISSISSIPPI DEPARTMENT OF TRANSPORTATION

FORM GRADES

N.B. C-D RD. & S.B. C-D RD. @ I55 NEAR STA. 725+00

PROJECT NO: ISP-0055-02(178)

MADISON COUNTY

REVISION	BY	DATE

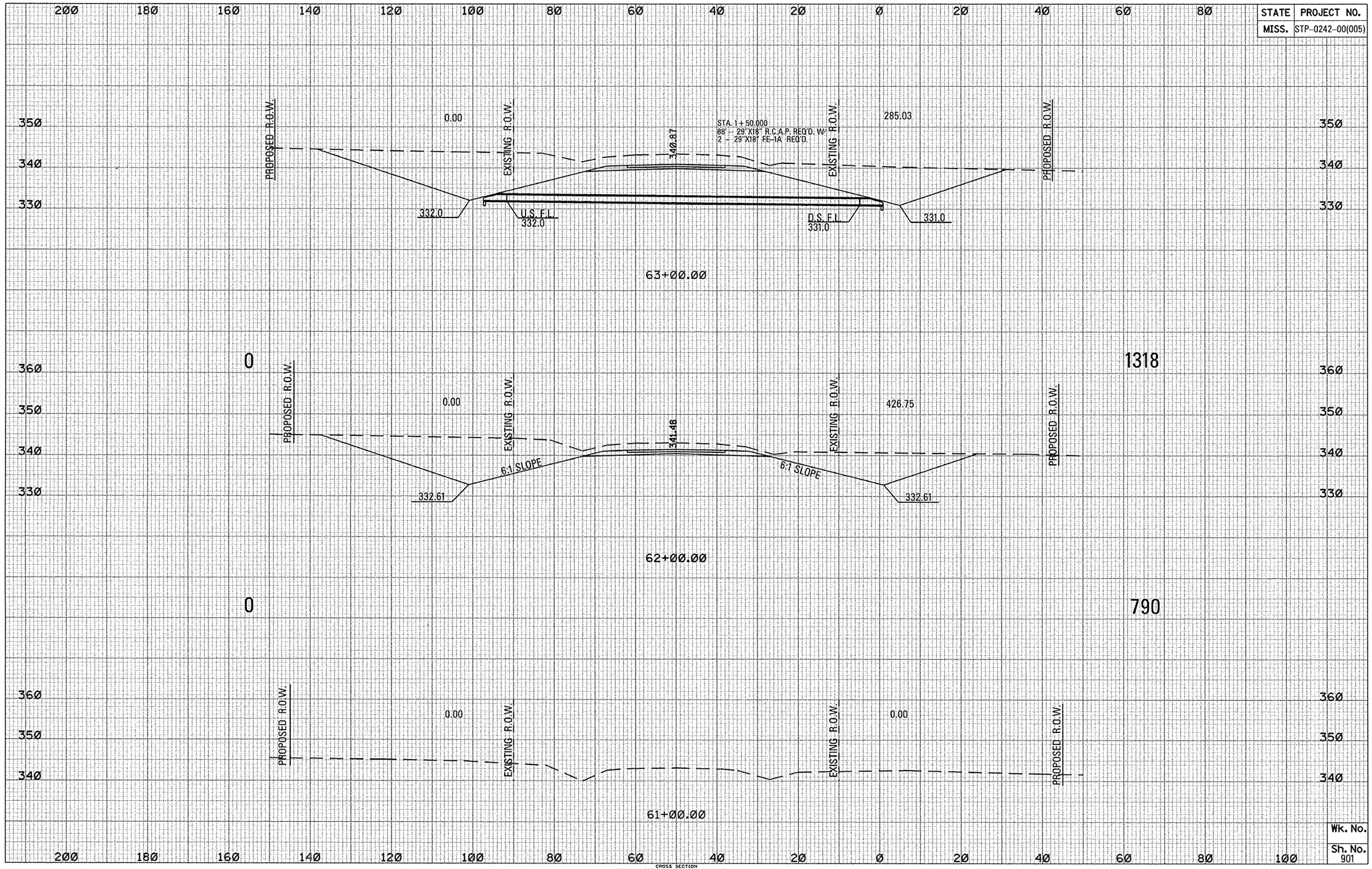
FILENAME: FG-01.DGN

DESIGN TEAM ____ DB ____ CHECKED ____ DATE ____

WORKING NUMBER: FG-1

SHEET NUMBER: 19

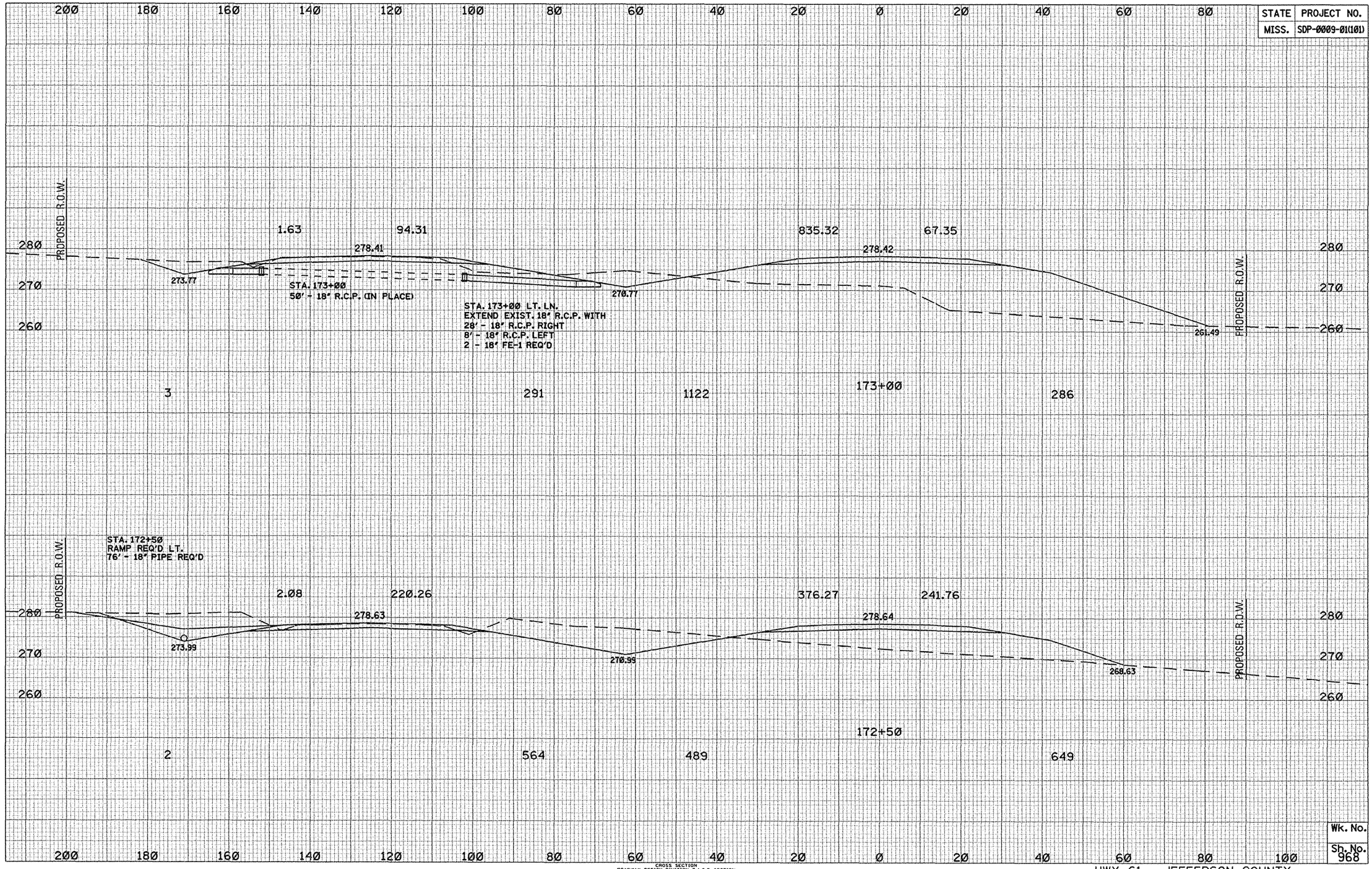
STATE	PROJECT NO.
MISS.	STP-0242-00(005)



Wk. No.
Sh. No. 901

CROSS SECTION
ROADWAY DESIGN DIVISION C.A.O.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	SDP-0009-01(101)



Wk. No.
Sh. No. 968

CROSS SECTION
ROADWAY DESIGN DIVISION C.I.A.D. SECTION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

HWY. 61 JEFFERSON COUNTY

STANDARD SYMBOLS

SOME OF THE STANDARD SYMBOLS WILL BE SHOWN HERE IN ORDER TO ASSIST IN UNIFORMITY OF SIZE, LINE WEIGHT, AND SYMBOL. THE CADD EQUIVALENT (CELLS, LINE STYLES) OF THESE SYMBOLS ARE AVAILABLE IN ROADWAY DESIGN'S CELL LIBRARIES AND LINE STYLE RESOURCE FILES.

ITEM	SYMBOL	MICROSTATION WEIGHT
COUNTY LINE		3
TOWNSHIP & RANGE LINE		3
SECTION LINE & CORNER	6 5 / 7 8	1
CENTERLINE OF CONSTRUCTION	65 70 ℄ CONSTR.	4
CENTERLINE OF (LABEL) (UNSTATIONED)	℄ LT.LN.	4
CENTERLINE OF TRAVERSE	65 70 ℄ TRAVERSE	4
CONSTRUCTION SLOPE LIMIT		2
RIGHT OF WAY & MARKERS (EXISTING)		0
RIGHT OF WAY & MARKERS (PROPOSED)		2
NO ACCESS LIMIT		4
EASEMENT LINE		3

ITEM	SYMBOL	MICROSTATION WEIGHT
PROPERTY LINE	PL PL	0
GRADE LINE PROFILE		3
ORIGINAL GROUND LINE (GRADING)		3
ORIGINAL GROUND LINE (PAVING)		3
ANGLE	40°	2
BENCHMARK		1
BRIDGE (EXISTING)		0
BRIDGE (PROPOSED)		2
BUILDING		0
CULVERT (EXISTING)		0
CULVERT (PROPOSED)		2

ITEM	SYMBOL	MICROSTATION WEIGHT
DITCH PLUG		1
FENCE	—×——×—	0
FIRE PLUG	FIRE PLUG	0
GUARDRAIL (PROPOSED)		3
INLET		2
MANHOLE	MH	0
RAILROAD		0
ROAD (EXISTING)	20' D.B.S.T.	0
STREAM OR CREEK	MUDDY CR	0
POWER LINE ABOVE GROUND	PP —P— PP	0
TELEGRAPH LINE ABOVE GROUND	—TG—	0
TELEPHONE LINE ABOVE GROUND	TP —T— TP	0

/standard/manual.dgn

ITEM	SYMBOL	MICROSTATION WEIGHT
GAS SERVICE LINE	—G— G —G—	0
HIGH PRESSURE GAS LINE	—HG—	0
POWER LINE UNDERGROUND	—P—UG—	0
SANITARY SEWER	—SS—	0
TELEPHONE LINE UNDERGROUND	—T—UG—	0
WATER LINE	—W—	0
WELL	W WELL	0
TANK UNDERGROUND	UG-TANK	0
NORTH ARROW		1

	MICROSTATION WEIGHT
	0
	1
	2
	3
	4
	5