REGISTERED PROFESSIONAL ENGINEER
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OKLAHOMA

ODOT

ROADWAY

DESIGN MANUAL

Oklahoma Department of Transportation

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PREFACE

The *ODOT Roadway Design Manual* has been developed with you, the Designer, in mind. It has been over twenty years since we have had a design tool comparable to this *Manual*. I hope that ODOT employees and consultants use this document to provide quality plans for the road users of this State.

The *ODOT Roadway Design Manual* is intended to provide uniform design practices and quality control for the preparation of contract plans for ODOT projects. The roadway designer should attempt to meet all criteria presented in the *Manual*. Where both "desirable" and "minimum" criteria are available, the designer should meet the desirable criteria if practical. However, the *Manual* should not be considered a standard which must be met regardless of impacts. Designers must exercise good judgment on individual projects and, frequently, they must be innovative in their approach to road design. This may require, for example, additional research into the literature.

The *ODOT Roadway Design Manual* was developed by the ODOT Design Division with assistance from the engineering consulting firm of Roy Jorgensen Associates, Inc. I would like to acknowledge and thank the Design Manual Committee for their work and a job well done — Clee Turbyfill, Chairman; Abraham Wong, FHWA; Guy Keith, Grossman-Keith and Associates; Christine Senkowski, Urban Design; Jim Rose, Traffic Engineering; Jim Carson, Rural Design. Special Committee Advisors — Susan Davis, Geometric Design; Jack Stewart, Office Engineer; Bob Rusch, Bridge Division; J. C. Mabry, Chief Traffic Engineer; Larry Hall, Traffic Engineering; Terry McFall, Planning Division; Tim Borg, Pavement Design Engineer; Te Ngo, Hydraulics Engineer; Pat Hernandez, Reproduction Branch; Stacey Trumbo, ARTEMIS; Sherri Hiller, ARTEMIS; and Charles Whittle for his work as the consultant's on-site project coordinator.

R. B. Hankins, P.E.
Assistant Director—Design

The Federal Highway Administration wishes to congratulate the Oklahoma Department of Transportation on the issuance of the *Oklahoma Roadway Design Manual*. The Federal Highway Administration has been pleased to participate in the development of this *Manual* through a representative on the ODOT Design Manual Committee. The *Manual* was based in part on Federal Highway Administration policy and regulation current at the time of the *Manual's* development. However, this *Manual* does not constitute a Federal policy, standard, specification or regulation. The United States Government assumes no liability for its contents or use therefore.

OKLAHOMA ROADWAY DESIGN MANUAL

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Chapter One

ODOT ORGANIZATION

This chapter will be prepared and submitted in the future.

Chapter Two

Project Development Process

Table of Contents

Chapter Two

PROJECT DEVELOPMENT PROCESS
(For Roadway Design)

Introduction

Chapter Two documents the basic approach used by ODOT in its project development process. The chapter presents a flowchart which graphically illustrates the development of a "typical" project. This is followed by a brief description of each activity within the flowchart. In their use of Chapter Two, the users should consider the following:

1. Precedence Activity Network. The flowchart of the project development process is a precedence activity network (see Figure 2A). An "activity" occurs when a significant, discrete event occurs and/or when the responsibility for the project (activity) is transferred from one unit to another. The "precedence" nature of the network implies that an activity cannot occur until all activities preceding that one have been completed.

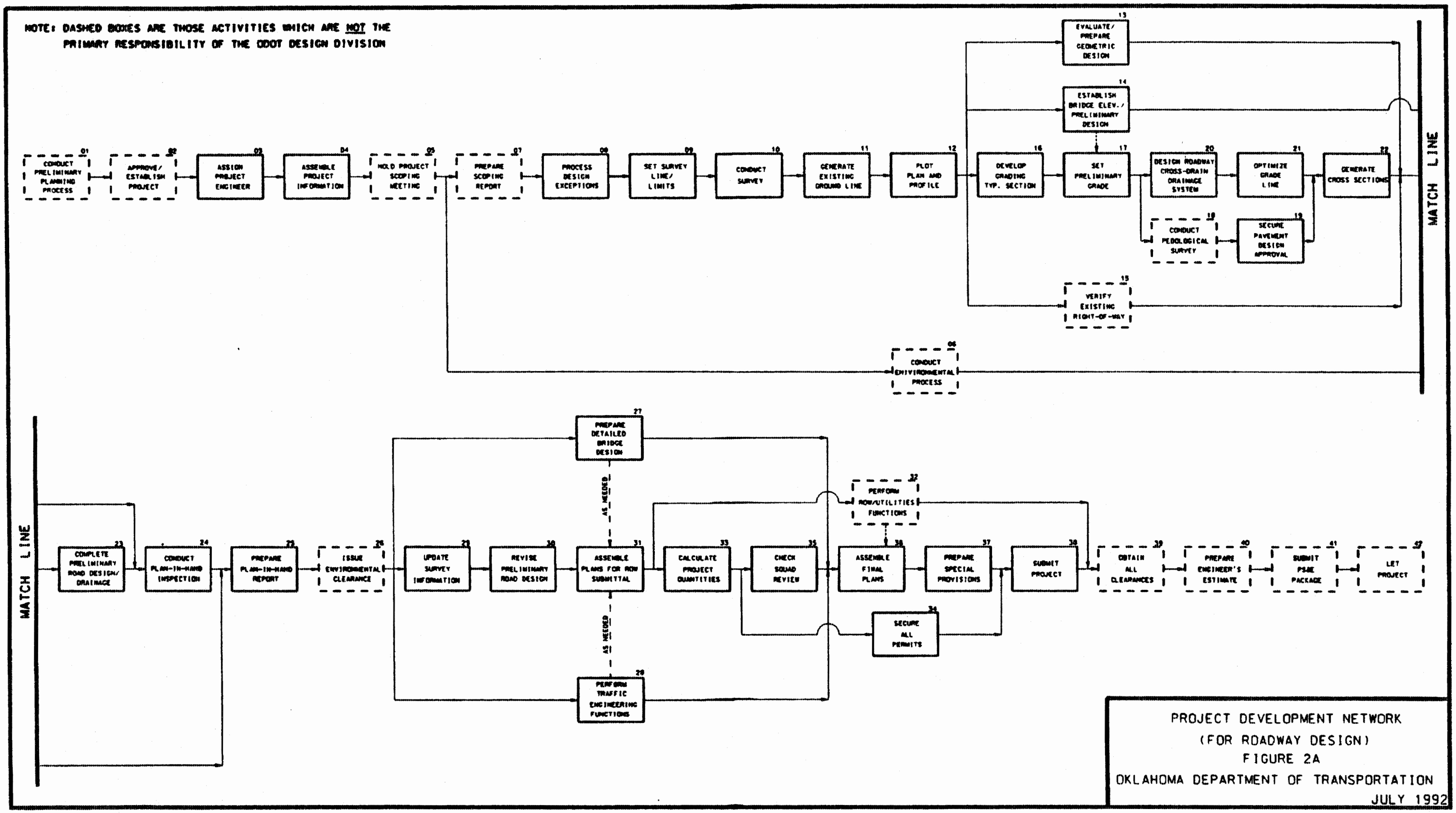
2. Project Application. The flowchart represents an approximate project development process for a relatively complicated project. Not every activity will be applicable to every project; i.e., some activities will represent "zero" time on relatively minor projects. In addition, some major projects are more complex than illustrated in the network. In general, the user should find that projects which are developed according to this process will have fewer management problems.

   The illustrated network assumes a project designed in-house. The process for a consultant-designed project may be similar, except that communication lines exist between ODOT and the consultant for ODOT review and approval.

3. Lines of Communication. The rigid application of the flowchart would lead to predetermined, precise points at which communication occurs between units. This is neither realistic nor preferable. Communications between units must be continuous. This will result in fewer problems and fewer "surprises" in the project development process.

4. Road Design Emphasis. The objective of the flowchart is to illustrate the significant activities for the road design element of project development. Other project development elements (e.g., environment, bridge design, right-of-way) are illustrated as single activities which diverge from and converge with the road design process (i.e., the main spine of the flowchart).

5. Other *Manual* Chapters. The *ODOT Roadway Design Manual* contains several other chapters which provide complementary information to Chapter Two. The designer should review these chapters for more information on the project development process. In particular, Chapter Two should be used in combination with Chapter Four "Plan Preparation," which describes the specific content of individual plan sheets.



PROJECT DEVELOPMENT NETWORK
(FOR ROADWAY DESIGN)
FIGURE 2A
OKLAHOMA DEPARTMENT OF TRANSPORTATION
JULY 1992

ROAD DESIGN ACTIVITY

Activity Title: Conduct Preliminary Planning Process

Activity No.: 01

Responsible Unit: ODOT Planning Division

Activity Description:

Before most proposed projects are presented to the State Transportation Commission, the Planning Division performs basic preliminary planning functions on each project. Depending upon the nature of the project, the planning and programming activities may include:

1. evaluating the project initiation request (e.g., from a legislator, private citizen or ODOT's normal planning process);

2. investigating the project relative to other needs and planned improvements in the same geographic area;

3. establishing a preliminary scope of work for the project (e.g, reconstruction, 3R non-freeway, spot improvement);

4. conducting a priority analysis for the project;

5. developing an estimate of construction costs and determining funding availability; and

6. estimating the likely environmental documents required for the project.

ROAD DESIGN ACTIVITY

Activity Title: Approve/Establish Project

Activity No.: 02

Responsible Unit: State Transportation Commission/Deputy Director/ODOT Programs Division

Activity Description:

After the preliminary planning process has been completed, the ODOT Deputy Director formally requests project approval from the Oklahoma State Transportation Commission. Commission approval is needed before ODOT can expend preliminary engineering funds on project development. The Commission may also assign the project to a specific fiscal year construction program.

After project approval, the ODOT Programs Division:

1. assigns a project number,
2. inserts the project into the ODOT Project Management System for tracking; and
3. transfers the project to the ODOT Planning Division, Transportation Planning Branch and/or the Assistant Director -- Design for further project development.

ROAD DESIGN ACTIVITY

Activity Title: Assign Project Engineer

Activity No.: 03

Responsible Unit: Applicable Design Division Engineer

Activity Description:

The Design Division Engineer assigns the project to a Project Engineer for development. The Project Engineer has the overall day-to-day responsibility for advancing the project through the design process. The Project Engineer:

1. coordinates directly with other units in ODOT;
2. attends all internal meetings and field inspections;
3. is responsible for ensuring that the project design meets all ODOT criteria and procedures;
4. reports directly to the Assistant Design Division Engineer on all significant project activities, problems and developments; and
5. participates in the public involvement process.

ROAD DESIGN ACTIVITY

Activity Title: Assemble Project Information

Activity No.: 04

Responsible Unit: Project Engineer

Activity Description:

The Project Engineer accumulates and reviews all available information on the proposed project and requests additional information as needed. As appropriate, the information may include (but not be limited to):

1. planning reports or studies;
2. record plans (as-built);
3. letters/correspondence on the project;
4. accident data;
5. traffic data and turning movements;
6. documentation on any public or private meetings;
7. original surveys;
8. aerial photos;
9. USGS quadrangle sheets; and
10. information and comments from other ODOT units, FHWA or other agencies.

This activity is conducted to prepare for the Project Scoping Meeting (Activity 05), as outlined in the ODOT Scoping Procedure(1). Depending upon the evaluation of the existing project information, the Project Engineer may determine that informal meetings or contacts with units internal or external to ODOT are appropriate before the Scoping Meeting. The Project Engineer submits a written request to the ODOT Planning Division, Transportation Planning Branch, for scheduling the scoping meeting.

ROAD DESIGN ACTIVITY

Activity Title: Hold Project Scoping Meeting

Activity No.: 05

Responsible Unit: ODOT Planning Division/Transportation Planning Branch

Activity Description:

The Environmental Coordinator schedules the Project Scoping Meeting to firmly establish the overall scope of the project design. The Project Scoping Team may include representatives from any or all of the following as applicable to the project:

1. Rural Design Division (Project Engineer, Drainage Engineer, Pavement Engineer);
2. Urban Design Division (Project Engineer, Geometric Design Engineer);
3. Bridge Division;
4. Traffic Division;
5. Right-of-Way Division (including Utilities Branch);
6. Materials Division;
7. Planning Division (Rail Planning Branch);
8. Field Division;
9. Federal Highway Administration; and
10. local government entity.

The basic objective of the Project Scoping Meeting is to evaluate the project objective and determine if the envisioned scope of the project design is consistent with the available funds. The Project Scoping Team, based on an evaluation of the assembled project information (Activity 04), will identify the basic project design parameters (e.g., scope of work, design speed, capacity improvements, pavement improvements, safety upgrading), major design exceptions and environmental requirements. The Team will usually conduct an on-site field review of the proposed project. The Team may also conclude that functional plans are required.

The estimated project funds, as approved by the Commission, may be inconsistent with the Scoping Team's judgment on the needed level of improvement. If so, the project scope of work must be redefined, or additional funds approved by the Commission, or the project dropped by the Commission.

ROAD DESIGN ACTIVITY

Activity Title: Conduct Environmental Process

Activity No.: 06

Responsible Unit: ODOT Planning Division/Transportation Planning Branch

Activity Description:

Based on the results of the Project Scoping Meeting (Activity 05), the Transportation Planning Branch completes ODOT's environmental process. Depending upon the nature of the project, the environmental process may include any or all of the following activities:

1. an assessment of early coordination needs;
2. a determination of the environmental class of action;
3. the preparation of the necessary environmental documents (e.g., EIS);
4. the administration of the public involvement process (e.g., public hearings); and
5. the identification of all necessary permits (see Activity 34).

Design activities may not proceed beyond the plan-in-hand field inspection until the environmental process has been completed. The final environmental clearance is discussed in Activity 26.

The *ODOT Action Plan* (2) describes ODOT's environmental involvement and presents the necessary step-by-step procedures for all projects to fulfill environmental requirements.

ROAD DESIGN ACTIVITY

Activity Title: Prepare Scoping Report

Activity No.: 07

Responsible Unit: ODOT Planning Division/Transportation Planning Branch

Activity Description:

If the Project Scoping Team determines that the envisioned project scope of work is consistent with the available funds (Activity 05), the Environmental Coordinator prepares a Scoping Report for the project based on information received from the Scoping Team members. The Report briefly discusses the following:

1. basic project geometrics, including:
    a. design speed,
    b. typical section (number and arrangement of lanes),
    c. basic vertical/horizontal alignment,
    d. interchange and intersection improvements, and
    e. level of roadside safety improvements;
2. any geometric design exceptions;
3. constructability/maintenance of traffic;
4. updated cost estimate;
5. right-of-way/utility impacts;
6. environmental impacts;
7. needed permits; and
8. a recommendation on the need for value engineering studies.

The Environmental Coordinator will submit the Project Scoping Report to the Team members for review and comment. After any needed changes have been made, the Report will be submitted for formal approval.

ROAD DESIGN ACTIVITY

Activity Title: Process Design Exceptions

Activity No.: 08

Responsible Unit: Project Engineer

Activity Description:

The Project Scoping Team in the Scoping Report will attempt to identify all design elements in the proposed project design which do not meet ODOT's criteria in the *ODOT Roadway Design Manual*. The Project Engineer is responsible for processing any design exceptions according to the procedures in Section 5.8 of the *Manual*. The process may include the need to secure FHWA approval of the design exception, depending upon several factors as discussed in Section 5.8. This step is important early in project development because it will ensure ODOT and FHWA acceptance of all critical project design features before ODOT resources are devoted to detailed project design.

ROAD DESIGN ACTIVITY

Activity Title: Set Survey Line/Limits

Activity No.: 09

Responsible Unit: Project Engineer

Activity Description:

In anticipation of the project survey (Activity 10), the Project Engineer in cooperation with Survey Division Engineers determines the project survey line and limits. This information will be essential before the Survey Division can survey the project. Depending upon the nature of the proposed project, the Project Engineer may request any or all of the following survey information:

1. existing field conditions (topography, vegetation, existing structures and road design features, etc.);

2. drainage features (bodies of water, open channels, channel slopes and cross sections, existing drainage appurtenances, etc.);

3. existing field landmarks;

4. existing utilities (above and below ground);

5. existing right-of-way markers and property lines;

6. proposed survey line for new project (plan and profile data); and

7. alignment and cross section of existing intersecting roads and driveways.

The Project Engineer may also compute the horizontal curvature data for the proposed alignment according to ODOT criteria presented in Chapters Six and Seven of the *ODOT Roadway Design Manual*. The objective is to select a trial horizontal alignment (with curvature rates, deflections and lengths) as soon as practical in project development. This information is plotted on the aerial photo.

ROAD DESIGN ACTIVITY

Activity Title: Conduct Survey

Activity No.: 10

Responsible Unit: ODOT Survey Division

Activity Description:

Based on the survey request from the Project Engineer (Activity 09), the Survey Division performs the project survey (conventional or aerial). The survey data will be compiled by the Survey Division, which performs several functions as part of the preparation of survey plans. The Division will, as appropriate:

1. make alignment ties to land corners or monuments;

2. locate all applicable new items in the field;

3. locate all utilities;

4. obtain conventional cross sections (station offset/elevation) or surface features (x, y, z coordinates); and

5. check field notes for accuracy and completeness.

Before the existing ground line can be produced, the Survey Division must edit the aerial survey data before it can be plotted.

The survey plans are submitted to the Project Engineer.

ROAD DESIGN ACTIVITY

Activity Title: Generate Existing Ground Line

Activity No.: 11

Responsible Unit: Project Engineer

Activity Description:

Under the direction of the Project Engineer, the existing ground line is generated either based on aerial, computer-generated digital terrain data or by reducing survey level notes. This may be accomplished either longitudinally or laterally along a survey or construction centerline.

ROAD DESIGN ACTIVITY

Activity Title: Plot Plan and Profile

Activity No.: 12

Responsible Unit: Project Engineer

Activity Description:

The road design squad, under the direction of the Project Engineer, prepares the plan and profile sheets for the project. These sheets will become the primary working drawings for all succeeding work on the project. As appropriate for the project, the plan and profile sheets may include:

1. all topographical data from the aerial survey, as checked by the field survey;

2. all relevant existing on-the-ground survey information;

3. final horizontal alignment of the new facility based on computed survey and curve data;

4. plan views of underground facilities;

5. in profile view, crossing elevations of underground facilities; and

6. existing centerline elevations.

At this stage, the proposed profile of the new facility has not normally been determined for presentation on the plan and profile sheets.

ROAD DESIGN ACTIVITY

Activity Title: Evaluate/Prepare Geometric Design

Activity No.: 13

Responsible Unit: ODOT Urban Design Division/Geometric Design Engineering Branch

Activity Description:

Upon receiving the plan and profile sheets from the Project Engineer on a project-by-project basis, the Geometric Design Branch evaluates the geometric design of the project and, for some items, selects the design details. The Branch is responsible for:

1. evaluating and commenting on the proposed horizontal and vertical alignment;

2. reviewing and sometimes determining intersection design details (e.g., auxiliary turn lanes, intersection sight distance, turning treatments);

3. determining interchange design details (e.g., lengths of acceleration and deceleration lanes, ramp widths, ramp alignment); and

4. providing sketches to the Project Engineer with sufficient detail to allow the preparation of the detailed sheets for the preliminary road design plans.

The Geometric Design Engineer will review all interchanges; the need for any other roadway review will be determined on a project-by-project basis.

ROAD DESIGN ACTIVITY

Activity Title: Establish Bridge Elevations/Preliminary Design

Activity No.: 14

Responsible Unit: ODOT Bridge Division

Activity Description:

If a bridge is within the limits of the proposed project, the Project Engineer submits a set of the plan and profile sheets with tentative roadway grades to the Bridge Division. The first objective of the bridge design is to establish the preliminary bridge elevations. This will be based upon ODOT requirements for vertical clearances and an estimate of the structure depth required at the site. The Bridge Division transmits the preliminary bridge elevation to the Project Engineer to allow the establishment of the preliminary roadway grade (Activity 17).

After receiving the plan and profile sheets, the Bridge Division also initiates the preliminary design of the bridge. The basic objective of the preliminary structure design is to determine the most appropriate bridge type and configuration for the given site conditions or, for existing bridges, the most appropriate level of bridge rehabilitation. The preliminary bridge design analysis is based on the evaluation of many factors, including geometrics, hydraulics, structural loads, foundation conditions, environmental and right-of-way impacts, aesthetics and construction costs. Constructability, detours, and maintenance of traffic during construction are also considered in the preliminary design. During this Activity, the Bridge Division should submit a request to the Materials Division for obtaining sufficient sounding data for design.

ROAD DESIGN ACTIVITY

Activity Title: Verify Existing Right-of-Way

Activity No.: 15

Responsible Unit: ODOT Right-of-Way (ROW) Division

Activity Description:

The Project Engineer submits the plan and profile sheets to the ROW Division to verify the existing ROW. The ROW Division researches the existing ROW arrangements within the project limits, including:

1. ROW titles and deeds,
2. permanent easements,
3. property lines and owners,
4. existing utility accommodation, and
5. existing limits of access.

The ROW Division conveys its findings to the Project Engineer before the preliminary roadway plans are prepared (Activity 23).

ROAD DESIGN ACTIVITY

Activity Title: Develop Grading Typical Section

Activity No.: 16

Responsible Unit: Project Engineer

Activity Description:

The road design squad, under the direction of the Project Engineer, develops the grading typical section for the proposed project. The section dimensions are based on Chapters Eight, Twelve and Thirteen of the *ODOT Roadway Design Manual*, which presents criteria for fill slopes, earth cuts, rock cuts, roadway widths, etc. The road design squad also selects a reasonable depth for an assumed pavement design.

In general, the side slope configuration should be as flat as practical. This will provide a safer design, decrease erosion problems and decrease maintenance problems.

ROAD DESIGN ACTIVITY

Activity Title: Set Preliminary Grade

Activity No.: 17

Responsible Unit: Project Engineer

Activity Description:

The road design squad, under the direction of the Project Engineer, sets a preliminary grade (including the proposed vertical curvature) based on the criteria in Chapter Seven of the *ODOT Roadway Design Manual* and based on the preliminary bridge elevations provided by the Bridge Division (Activity 14). The grade is not computed, nor are there any earthwork calculations. A major objective of setting the preliminary grade is to provide the Materials Division with a general estimate of the cuts and fills which will be encountered. The grade should, however, be reasonably close to the final grade to allow the pedological soils survey to be as accurate as practical.

ROAD DESIGN ACTIVITY

Activity Title: Conduct Pedological Survey

Activity No.: 18

Responsible Unit: ODOT Materials Division

Activity Description:

The Project Engineer submits the preliminary grade (Activity 17) to the Materials Division to conduct the pedological survey. The objective of the survey is to investigate the types of soils, potential slides or faults within the project limits and other relevant soils information. Upon request from the Project Engineer, the Materials Division includes relevant soils information such as location of roadway borrow areas. The Materials Division prepares a Pedological Report with its findings and recommendations. The Report is submitted to the Project Engineer to develop the preliminary cross sections (Activity 22). The Report is also submitted to the Project Engineer and the Rural Design Division/Pavement Design Engineer for the pavement design analysis (Activity 19).

ROAD DESIGN ACTIVITY

Activity Title: Secure Pavement Design Approval

Activity No.: 19

Responsible Unit: ODOT Rural Design Division/Pavement Design Engineer

Activity Description:

Based on the results of the pedological survey (Activity 18) and the typical sections for grading (Activity 16), the Pavement Design Engineer performs the pavement design analysis. Chapter Sixteen of the *ODOT Roadway Design Manual* discusses the ODOT pavement design procedures and criteria. Many factors impact the analysis, including:

1. the functional class of roadway;
2. the structural support of the subgrade;
3. the projected traffic volumes, especially the truck volumes;
4. the design analysis period; and
5. the selected terminal pavement serviceability.

The Pavement Design Engineer secures approval of the proposed pavement design from the ODOT Pavement Design Committee. Once approved, the Pavement Design Engineer submits the pavement design to the Project Engineer for incorporation in the project plans (Activity 22).

ROAD DESIGN ACTIVITY

Activity Title: Design Major Roadway Cross-Drain Drainage System

Activity No.: 20

Responsible Unit: Project Engineer

Activity Description:

The road design squad, under the direction of the Project Engineer, develops the major roadway drainage system cross drains. This involves, where applicable:

1. conducting the hydrological calculations (based on rainfall data, drainage basin characteristics, etc.);

2. designing the open cross drainage system (channel size for creek or stream crossing); and

3. designing all culverts which cross the roadway and are not classified as bridges (culvert size and slope, inlet configuration and energy dissipators to determine the head water requirements for roadway elevations and overtopping considerations if any.).

After the drainage design is completed, the Project Engineer determines the need to revise any existing 100-year flood maps prepared by the Federal Emergency Management Agency (FEMA). If revisions are necessary, the Project Engineer initiates the process.

The *ODOT Drainage Manual* (3) and Chapter Fifteen of the *ODOT Roadway Design Manual* present the acceptable criteria and methodologies to design the roadway drainage system.

ROAD DESIGN ACTIVITY

Activity Title: Optimize Grade Line

Activity No.: 21

Responsible Unit: Project Engineer

Activity Description:

At this stage of project development, the following have been prepared:

1. plan and profile sheets (Activity 12),
2. grading typical section (Activity 16),
3. preliminary grade (Activity 17), and
4. major roadway cross-drain drainage design (Activity 20).

Based on this information, the road design squad (under the direction of the Project Engineer) determines the optimum grade line and performs the earthwork calculations for the preliminary road design plans. Several objectives must be met to optimize the grade line, including:

5. meeting critical field controls (e.g., railroad crossings, bridge elevations, ramp connections, topography, utilities, driveways);

6. meeting critical geometric controls (e.g., vertical clearances, grades);

7. providing a grade consistent with the major roadway cross-drain drainage design;

8. providing a rough balance of earthwork (i.e., minimize borrow or waste); and

9. minimizing environmental impacts (e.g., erosion, encroachment into streams and wetlands).

Note that optimization is a repetitive or iterative process and may be impacted by the development of roadway cross sections (Activity 22).

ROAD DESIGN ACTIVITY

Activity Title: Generate Cross Sections

Activity No.: 22

Responsible Unit: Project Engineer

Activity Description:

After the desired grade line and earthwork balance have been achieved (Activity 21), the road design squad generates the roadway cross sections. These are scanned to ensure that all controls are met, as listed in Activity 21. The design squad should also ensure that, for example, there are no short extents of slope changes which would be unsightly. The cross sections will incorporate and present the approved pavement design structure (Activity 19).

Any minor changes are noted on the cross sections, and they are regenerated to produce the final cross sections for the preliminary road design plans.

ROAD DESIGN ACTIVITY

Activity Title: Complete Preliminary Road Design/Drainage

Activity No.: 23

Responsible Unit: Project Engineer

Activity Description:

After the major project design features have been completed, the road design squad can now assemble all developed plan sheets, and the squad can complete the preparation of the remaining plan sheets for the preliminary road design. The overall objectives of the preliminary road design plans are 1) to prepare for the plan-in-hand field inspection (Activity 24), and 2) to minimize the necessary work on the final plans, assuming relatively minor changes result from the field inspection. The final compilation of the preliminary road design plans includes the following activities:

1. completing drainage design for inlets and storm sewers on curbed facilities;
2. plotting the toe of slopes on the plan and profile sheets;
3. presenting the necessary details on the title sheet (e.g., project numbers, project length, traffic data, location);
4. presenting the necessary details on the typical sections (e.g., guardrail widening, topsoil notes, ditch sections);
5. presenting the necessary details on the plan and profile sheets (e.g., grade percents, vertical and horizontal curvature data, structure sizes, edge of pavement, property lines, stationing);
6. preparing tentative construction sequence and construction traffic control;
7. developing plan sheets for intersections and interchanges;
8. preparing a preliminary summary of pay quantities;
9. developing the erosion control plan;
10. addressing special design considerations; and
11. reviewing the checklist of plan requirements for plan-in-hand.

Road design plans are distributed to all parties who need to have input into the project design. Table 2A indicates the typical plan distribution.

Table 2A

DISTRIBUTION OF PLANS

Organization/Unit	Project Stage			
	Scoping Plans*	Functional Plans	Plan-in-Hand Plans	Review Plans
FHWA	1	2	2	2
ROW Division	1	1	1	0
Field Division	2	2	2	2
Traffic Engineering Division	1	1	1	1
Roadway Design Division	1	1	1	1
Bridge Division	1	1	1	1
Planning Division/Rail Planning	1	1	1	1
Local Entity	1	1	1	1
Planning Division/ Transportation Planning Branch	1	1	1	0
Geometric Design Branch	1	1	1	0

* Include a print of the aerial roll for each set of plan-in-hand prints.

<u>Note</u>: Preliminary plans should be submitted to other organizations/units two weeks before plan-in-hand date.

ROAD DESIGN ACTIVITY

Activity Title: Conduct Plan-in-Hand Field Inspection

Activity No.: 24

Responsible Unit: Project Engineer

Activity Description:

The Project Engineer coordinates with all units who will attend the plan-in-hand field inspection to identify a mutually acceptable date. Depending upon the nature of the project, the attendees at the inspection may include representatives from any or all of the following:

1. Urban/Rural Design Division;
2. Bridge Division;
3. Traffic Engineering Division;
4. Urban Design Division, Geometric Design Branch;
5. Right-of-Way Division (including Utilities Branch);
6. Planning Division (Transportation Planning Branch and Rail Planning Branch);
7. Field Division;
8. FHWA; and
9. local entity.

The plan-in-hand field inspection is a multi-purpose function during which the Project Engineer presents his proposals for consideration by others, checks for compatibility with existing features, assures completeness of survey, determines the maintenance of traffic during construction, discusses the new facility with the local political subdivision and advises of any road or street closures and checks the compatibility of plans.

Any changes decided upon during the plan-in-hand will be marked with permanent marking on the prints, including the date of inspection and the names of all participants. These prints will be retained and checked to ensure that all items are placed on the plans. After construction is completed, these plan-in-hand prints may be destroyed.

ROAD DESIGN ACTIVITY

Activity Title: Prepare Plan-in-Hand Report

Activity No.: 25

Responsible Unit: Project Engineer/Various Units

Activity Description:

The Project Engineer has the overall responsibility to develop the Plan-in-Hand (PIH) Report, which documents the decisions and understandings reached during the plan-in-hand field inspection. Normally, the PIH report is generated through a process in which each attendee at the field inspection independently prepares a memorandum documenting his understanding of the agreements made. For example, the FHWA representative may document the need to obtain a design exception for a geometric design element which was not identified in Activity 08. The Project Engineer is also responsible for ensuring that all necessary changes discussed in the PIH Report are incorporated into the final plans.

The Project Engineer also updates the preliminary construction cost estimate at this stage of project development. If necessary, the Project Engineer notifies the ODOT Programs Division of the need to revise the allocated construction funds in the State construction program.

ROAD DESIGN ACTIVITY

Activity Title: Issue Environmental Clearance

Activity No.: 26

Responsible Unit: ODOT Planning Division/Transportation Planning Branch

Activity Description:

Activity 06 discusses the ODOT environmental process. The Transportation Planning Branch obtains FHWA approval and formally certifies that the process has been completed. The Branch issues the environmental clearance and submits the clearance to the Project Engineer. The environmental clearance must be obtained prior to submission of ROW plans. Any work on the plans beyond P-I-H stage prior to receipt of environmental clearances is subject to revision.

It is possible that the scope of work can be altered significantly during the processing of environmental issues, public hearings, etc. The environmental process could require changes to the plans ranging from site specific special considerations, changing of the alignment or even cancellation of the project.

ROAD DESIGN ACTIVITY

Activity Title: Prepare Detailed Bridge Design

Activity No.: 27

Responsible Unit: ODOT Bridge Division

Activity Description:

After the Bridge Division has completed its preliminary bridge design, secured the necessary approvals, and attended the plan-in-hand field inspection, the Division prepares the detailed bridge design. The basic objective of the detailed design phase of bridge development is to perform the in-depth structural analyses which are necessary to prepare a set of construction plans. The structural analyses, as applicable, may include the:

1. superstructure design (e.g., framing details, deck slab, camber diagram);
2. substructure design (e.g., piers, abutments);
3. foundation design;
4. approach slab design; and
5. bridge rail design.

Note: If any major changes to the preliminary bridge design are made during PIH (Activity 24), updated sounding data must be requested (see Activity 14).

For a combination road and bridge project, the Bridge Division submits its final bridge plans to the Project Engineer for compilation into the final plans (Activity 36).

ROAD DESIGN ACTIVITY

Activity Title: Perform Traffic Engineering Functions

Activity No.: 28

Responsible Unit: ODOT Traffic Engineering Division

Activity Description:

After the plan-in-hand field inspection, the ODOT Traffic Engineering Division performs all needed project work which is within the responsibility of the Division. Depending upon the nature of the project, the Traffic Engineering Division:

1. determines the application of signs and pavement markings;

2. designs and sets the timing for traffic signals;

3. designs the highway lighting system;

4. determines traffic control treatments at railroad/highway grade crossings; and

5. designs the layout of traffic control devices for the maintenance and protection of traffic through the construction zone.

The Traffic Engineering Division is responsible for preparing the necessary plan sheets for direct insertion into the final plans (Activity 36).

ROAD DESIGN ACTIVITY

<u>Activity Title</u>: Update Survey Information

<u>Activity No.</u>: 29

<u>Responsible Unit</u>: ODOT Survey Division

<u>Activity Description</u>:

The plan-in-hand field inspection may reveal the need to obtain additional survey data or to update the existing survey information. For example, the on-site inspection may identify existing utilities which are not located on the preliminary road design plans. The Project Engineer makes the determination on the need for additional survey work and notifies the Survey Division. The Division performs the survey and provides the needed information to the Project Engineer for incorporation into the plans (Activity 30).

ROAD DESIGN ACTIVITY

Activity Title: Revise Preliminary Road Design

Activity No.: 30

Responsible Unit: Project Engineer

Activity Description:

The road design squad, under the direction of the Project Engineer, makes all necessary changes to the preliminary road design plans based on the results of the plan-in-hand field inspection. In particular, the squad ensures that all construction right-of-way impacts from the road design are clearly reflected in the plans. All access control lines, fencing, toe of slopes, etc., are plotted onto the plans for ROW submission. In addition, the squad completes any road design details which may not have been completed in the preliminary road design plans.

ROAD DESIGN ACTIVITY

Activity Title: Assemble Plans for ROW Submittal

Activity No.: 31

Responsible Unit: Project Engineer

Activity Description:

The construction plans for ROW submission are intended to clearly reflect all engineering work which has a ROW impact. This allows the ODOT ROW Division to develop the ROW plans and initiate the ROW acquisition and/or utility accommodation processes. The Project Engineer is responsible for assembling all construction plan sheets, as applicable, from all sources into the plans for ROW submission.

The Project Engineer submits plans which include the Title Sheet, Typical Sections and Plan and Profile Sheets and one set of cross sections to the ROW Division. The actual number of sets submitted depends upon the type of project. The Project Engineer should contact the ROW Division to determine the number of sets needed.

ROAD DESIGN ACTIVITY

Activity Title: Perform ROW/Utilities Functions

Activity No.: 32

Responsible Unit: ODOT Right-of-Way Division/Utilities Branch

Activity Description:

The ODOT Right-of-Way Division determines the ROW requirements from the construction plans submitted in Activity 31. Where necessary, the Division develops ROW plans, completes the ROW acquisition process to accommodate construction, and secures any needed permanent and temporary easements. When the ROW process is completed, the ROW Division notifies the Office Engineer that all ROW is clear (Activity 39).

The Utilities Branch within the ROW Division is responsible for any utility adjustments or relocations. The Branch furnishes each affected utility owner with all pertinent plan sheets. The utility owners are responsible for submitting their relocation plans and schedules to the Utilities Branch for review and approval. The Branch notifies the Office Engineer that all Utility Agreements have been executed (Activity 39). The Branch submits to the Project Engineer any utility plan sheets for direct insertion into the contract plans if they are to be a part of the contract (Activity 36).

ROAD DESIGN ACTIVITY

Activity Title: Calculate Project Quantities

Activity No.: 33

Responsible Unit: Project Engineer

Activity Description:

The road design squad, under the direction of the Project Engineer, calculates all project quantities and prepares the schedule for the Summary Sheets. This includes quantities for roadway, drainage and earthwork items. The squad is responsible for ensuring that it tabulates project quantities consistent with the *ODOT Standard Specifications for Highway Construction* and Supplements (1988, 1991) (4) for pay items, units of measurement, rounding conventions, etc.

Quantities for bridge and/or traffic items are prepared and submitted by the respective Division Project Engineer. Each Division Project Engineer is responsible for entering pay quantity information into the Bid Analysis Management System (BAMS).

ROAD DESIGN ACTIVITY

Activity Title: Secure All Permits

Activity No.: 34

Responsible Unit: ODOT Bridge Division and Planning Division/Transportation Planning Branch

Activity Description:

The preparation of the plans for right-of-way submission (Activity 31) and calculation of project quantities (Activity 33) will provide sufficient detail of the project impacts (acres of wetland impact) to allow the compilation of the data needed to secure the project permits. Depending upon the precise nature of the project, the permits may include any or all of the following:

1. U.S. Army Corps of Engineers, Section 404 Discharge of Fill;
2. U.S. Coast Guard, Section 9 Navigable Waters;
3. U.S. Coast Guard, Section 10 Navigable Waters;
4. Floodplains Development Permit;
5. FAA, airspace clearances (FAA Form 7460-1 (8-85)); and
6. National Pollutant Discharge Elimination System (NPDES), EPA Stormwater Permit.

The Project Engineer will coordinate with the Transportation Planning Branch, Bridge Division or Traffic Engineering Division to furnish all needed information for the State or Federal permit. This information may include a project description, permit justification, environmental impacts, engineering details of the proposed project and site photographs. Chapter Seventeen of the *ODOT Roadway Design Manual* discusses the permit information needs in more detail.

ROAD DESIGN ACTIVITY

Activity Title: Check Squad Review

Activity No.: 35

Responsible Unit: (Other) Road Design Squad

Activity Description:

If there is sufficient time and a squad is available, a complete set of plan sheets is submitted to a road design squad other than the squad which prepared the plans. This squad conducts a spot check of the plans to ensure that they are reasonably accurate, complete, legible and constructable. The purpose of the check is not to evaluate the project from an engineering perspective but, rather, to locate any significant errors and to ensure that the contractor will be able to construct the project as designed.

The check squad notifies the Project Engineer of any needed changes to the plan set.

ROAD DESIGN ACTIVITY

Activity Title: Assemble Final Plans

Activity No.: 36

Responsible Unit: Project Engineer

Activity Description:

The Project Engineer assembles all bridge, traffic, etc., plan sheets into one set of final plans. In addition to all plan sheets developed for the specific project, the final plans include any ODOT Standard Drawings which apply to the project. The Project Engineer includes the Standard Drawing numbers on the Title Sheet, and they are automatically printed and included in the final set of plans by the Reproduction Branch. The road design squad also develops any special details for project construction items which do not conform to the details in the Standard Drawings.

ROAD DESIGN ACTIVITY

Activity Title: Prepare Special Provisions

Activity No.: 37

Responsible Unit: Project Engineer/Specifications Engineer

Activity Description:

Special provisions are included in contracts as required to define work, materials, procedures or requirements not covered in the Standard Specifications or Supplemental Specifications. With few exceptions, each special provision is drafted by the Project Engineer and approved by the applicable Division Engineer. It contains five parts similar to the items in the Standard Specifications: (1) Description of Work, (2) Materials, (3) Construction Methods, (4) Method of Measurement, and (5) Basis of Payment. Special provisions supplementing and/or modifying the Standard Specifications may not require full specifications and may be written to include a reference to the Standard Specifications and provide only the information necessary to detail the supplement and/or modification. Any parts of the Standard Specifications changed by special provisions must be clearly identified. Occasionally, special provisions are used to specify specific types of material or optional material rather than using notes on the plans. Whenever feasible, a simple note on the plans should be used to eliminate the need for special provisions. The use of a note is preferred when it meets the approval of the Project Engineer.

All new special provisions requiring materials which are not covered in the Standard Specifications should have the approval of the Materials Engineer. All new special provisions, regardless of content, should have the prior approval of the Federal Highway Administration. See Chapters Four and Eighteen for additional information on Special Provision development.

After the preparation of the special provisions, the Specifications Engineer submits all project materials to the Office Engineer for final processing and Federal Highway Administration approval.

ROAD DESIGN ACTIVITY

Activity Title: Submit Project

Activity No.: 38

Responsible Unit: Project Engineer

Activity Description:

The Project Engineer submits final plans, the construction estimate, list of special provisions, completed BAMS quantity documentation and set-ups to the Office Engineer. With this step, excepting additions or the discovery of errors, the formal Road Design Process has been completed.

The Project Engineer will be requested to attend and participate in the pre-bid conference or pre-construction conference. He is also expected to consult with the contractor or Field Division personnel relative to change-of-plan decisions.

ROAD DESIGN ACTIVITY

Activity Title: Obtain All Clearances

Activity No.: 39

Responsible Unit: Office Engineer

Activity Description:

Before the project can be further processed, the Office Engineer must ensure that all necessary clearances, approvals, etc., have been obtained. Depending upon the nature of the project, these may include:

1. right-of-way certificate;
2. utility agreements;
3. permit approvals;
4. railroad agreements;
5. local agency agreements;
6. FHWA approvals (design exceptions, experimental features, environmental approvals); and
7. funding.

ROAD DESIGN ACTIVITY

Activity Title: Prepare Engineer's Estimate

Activity No.: 40

Responsible Unit: Office Engineer

Activity Description:

With the project pay items and quantities, the Office Engineer prepares the Engineer's Estimate for the project. The estimate is based on a variety of factors, including:

1. average weighted prices for each pay item;
2. quantity amount for each item (i.e., "large" quantities yield lower unit costs);
3. geographic location;
4. historical or recent trends for each item;
5. project specific factors; and
6. the judgment of the Office Engineer.

ROAD DESIGN ACTIVITY

Activity Title: Submit PS&E Package

Activity No.: 41

Responsible Unit: Office Engineer

Activity Description:

On Federally funded projects, the Office Engineer submits the entire PS&E package to the FHWA Oklahoma Division Office for review and approval. FHWA documents its approval by issuing the PR-1240, which authorizes the expenditure of Federal-aid construction funds for the project.

ROAD DESIGN ACTIVITY

Activity Title: Let Project

Activity No.: 42

Responsible Unit: Office Engineer

Activity Description:

After receiving FHWA approval of the PS&E package, the Office Engineer:

1. prepares the Proposal Book,
2. develops a construction time schedule,
3. mass reproduces the contract package,
4. publishes the formal advertisement to solicit bids from interested contractors,
5. conducts the bid opening,
6. identifies the low bidder,
7. evaluates bids, and
8. recommends award of contract to the State Transportation Commission.

Upon approval of award of bid by the Transportation Commission and concurrence by the FHWA, the PS&E package is submitted to the ODOT Construction Division. The Construction Division then issues a work order and project construction begins.

References

1. *Scoping Procedure* (Adopted by Oklahoma Transportation Commission), written by multi-disciplinary team, October 1990.

2. *ODOT Action Plan*, Oklahoma Department of Transportation, Planning Division (with assistance from FHWA), Revised 1980.

3. *ODOT Drainage Manual*, Oklahoma Department of Transportation, 1992.

4. *ODOT Standard Specifications for Highway Construction* (and Supplements), Oklahoma Department of Transportation, 1988, 1991.

Chapter Three

Policies and Procedures

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Chapter Three

POLICIES AND PROCEDURES

This Chapter discusses items related to the in-house operational practices for Design (the units organized under the Assistant Director — Design).

3.1 COORDINATION OF PLAN DEVELOPMENT

3.1.1 General

For most construction projects, Design is responsible for plan development. Proper plan development demands that all elements in the production of a set of plans be coordinated with all involved ODOT units and all affected units external to ODOT. Each unit must remain informed of developments as project work progresses. Any delays in proper coordination may delay project advertisement and may result in an inefficient use of ODOT resources.

Section 3.1 presents a brief discussion on the coordination activities between Design and other units internal and external to ODOT. In addition, the following should be reviewed for more information on the coordination of plan development:

1. ODOT *Policy Manual*. Section A "General" discusses the basic responsibilities of the functional units within ODOT.

2. Chapter One "ODOT Organization." Chapter One of this *Manual* discusses the ODOT organizational structure and discusses the functional responsibilities of ODOT units, with an emphasis on Design.

3. Chapter Two "Project Development Process." Chapter Two of this *Manual* presents a flowchart for the project development process. The network is based upon the step-by-step procedure required by the road designer in plan development. It also demonstrates the interaction with other units in coordinating plan development.

4. Chapter Four "Plan Development." Chapter Four of this *Manual* discusses ODOT's criteria for the development of the individual sheets within a set of construction plans (e.g., plan/profile sheet, cross section sheet).

3.1.2 Internal ODOT Units

This Section briefly discusses the coordination activities between the Design Divisions and other ODOT units in the development of project plans.

3.1.2.1 Rural/Urban Design Divisions

The Rural and Urban Design Divisions are responsible for project design and plan development. The following summarizes the major duties of the design squads within the Design Divisions:

1. The design squads operate as the focal point for the development of roadway projects administered by ODOT, including the preparation of plans, the preparation of special provisions, the project schedule and coordination with other ODOT units and outside agencies.

2. The design squads perform most of the analyses for road design, prepare reports and gain input from other Divisions.

3. The design squads perform the necessary drafting work for the preparation of in-house project plans.

4. The design squads develop, in cooperation with the Survey Division, the project survey line and limits.

5. The design squads are responsible for the design of all roadway drainage design, including cross-drainage structures which are not classified as bridges.

6. The design squads work with the Traffic Engineering Division to determine the appropriate traffic control devices, pavement markings, lighting and/or traffic signals.

7. The design squads coordinate with utility and railroad companies through the appropriate ODOT Divisions for proper integration into road design plans.

In addition to the design squads, the following support units are also within the Rural Design Division — Automation and Graphics Branch and Quality Control and Design Review Branch. Pavement Design, Roadway Hydraulics, Roadside Development Branch and Standard Drawings Sections are within the Design Support Unit of the Rural Design Division. The Geometric Design Branch is within the Urban Design Division. Chapter One provides information on the responsibilities of these various support units.

3.1.2.2 Bridge Division

The Bridge Division, in general, is responsible for the design of all structures on the State highway system. The following summarizes the coordination between the designer and the Bridge Division:

1. The Bridge Division is responsible for the design of bridges, bridge culverts, retaining walls, sign supports, signal supports and luminaire supports.

2. Where bridges are within the limits of a roadway project, the bridge designer and road designer work together to determine the geometrics at the bridge. This includes bridge widths, vertical clearances, cross slopes, alignment and sidewalks. Where conflicts arise, the Bridge Division typically makes the final determination on geometrics at the bridge.

3. The Bridge Division is responsible for conducting the hydraulic analysis of all bridge classified structures.

4. The Bridge Division is responsible for requesting foundation and boring studies from the Materials Division.

5. For projects on existing highways, the road and bridge designers coordinate on the inspection of existing bridges.

3.1.2.3 Survey Division

The Survey Division is responsible for conducting field and aerial surveys necessary for the design of projects on the State highway system. Upon request from Design personnel, the Survey Division will conduct surveys needed for each project at the level of detail necessary for project design. The Survey Division also compiles the survey data and submits the information to the designer.

3.1.2.4 Traffic Engineering Division

The Traffic Engineering Division is responsible for determining the warrants and placement for all traffic control devices (permanent and temporary), highway lighting, accident data, skid testing, speed studies, posted speed zone studies, fabrication of all highway signing used statewide for sign maintenance, and safety projects including railroad/highway grade crossings. The following summarizes the coordination between the designer and the Traffic Engineering Division:

1. Traffic will determine the selection and placement of the signing, pavement markings, traffic signals and highway lighting with input from the designer. See Chapter Fourteen for additional information.

2. Traffic and the designer will work together to determine the strategy for the maintenance and protection of traffic through the construction zone. See Chapter Fourteen for additional information.

3. Traffic and the designer will work together to determine the access control along the road section (e.g., driveways, median openings).

4. On some safety projects, Traffic will determine the basic safety improvement (e.g., provide an exclusive left-turn lane), and the designer will prepare the detailed design and coordinate the construction plans for the project.

5. Traffic participates in the project scoping process.

3.1.2.5 Planning Division

The Planning Division is responsible for the overall Statewide transportation planning process for all modes of transportation. The Division is also responsible for activities related to environmental impacts and procedures. The designer coordinates with the Planning Division to:

1. determine the functional classification and Federal-aid system of the roadway on which the project is located;

2. acquire general information on the State highway system (e.g., Highway Performance Monitoring System data);

3. acquire design traffic data;

4. acquire data from traffic field studies;

5. coordinate the administrative functions for railroad grade crossings within the project limits (e.g., contacts with the railroad company);

6. ensure that all environmental commitments and procedures are met in project design (see Section 3.8 and 3.9); and

7. coordinate the project scoping process. See Reference 3.

3.1.2.6 Research and Development Division

The Research and Development Division is responsible for all formal research activities conducted by ODOT. The designer will coordinate with the Research and Development Division when experimental items are included within the project design. Chapter Eighteen further discusses experimental items.

3.1.2.7 Materials Division

The Materials Division is responsible for field sampling, testing and certifying all materials used on ODOT projects. This includes soils analyses and materials for pavements and structures. The following summarizes the coordination between the designer and Materials Division:

1. The Materials Division conducts soil borings to determine the structural properties of the subgrade for pavements.

2. Upon request, the Materials Division evaluates soil and rock materials for fills, cuts and structure foundations.

3. Upon request, the Materials Division determines if sufficient borrow material is available in the area of the project.

4. Upon request, the Materials Division tests new materials for potential use on highway projects.

5. Upon request, the Materials Division reviews consultant geotechnical reports and recommendations.

3.1.2.8 Right-of-Way Division

The Right-of-Way Division is responsible for all activities related to the legal right-of-way and access onto the State highway system. This includes appraisals, acquisitions, relocation, property management and agreements with utility companies. The following summarizes the coordination between the designer and Right-of-Way Division (R/W):

1. R/W assists Design during the preliminary design stage by providing right-of-way, utility and relocation estimates.

2. Upon request from Design, R/W provides present right-of-way and ownership verification.

3. The designer provides R/W with the needed design information to determine the right-of-way impacts.

4. R/W prepares, or causes to be prepared by Consultants, a separate set of right-of-way plans for each project where right-of-way adjustments are required.

5. R/W prepares, or causes to be prepared by consultants, right-of-way documents for projects.

6. R/W performs all right-of-way work and procures all takings and easements needed for the project and notifies Design when the right-of-way is clear.

7. The designer and R/W work together to determine the project impacts on utility companies. R/W is the lead unit for contacts with utility companies and negotiates all agreements. The designer coordinates with the Utility Branch to ensure that the utility work is accommodated by the roadway plans.

8. The designer must coordinate with all R/W branches to ensure that all statutory requirements are satisfied.

3.1.2.9 Programs Division

The Programs Division is responsible for assigning job piece and project numbers for all State and Federal construction projects within ODOT and for administering the

Federal budget. This includes preliminary engineering, right-of-way, utility, construction and maintenance projects. The Project Engineer should formally request a project number before any work is performed on a given project. Any major changes to a project including combining projects, separating projects, changing lengths, revising the project description, scope of work or other major factors should be coordinated with the Programs Division to make appropriate changes in the approved construction program. Preliminary cost estimates should also be provided to the Programs Division on all projects when they are initially established. Updated estimates should be submitted to the Programs Division as the project develops.

3.1.2.10 Office Engineer Division

The Office Engineer Division is responsible for all activities related to the advertising and letting of all projects prepared by ODOT. This includes preparing estimates and proposals, conducting bid openings, tabulating bids, recommending project awards and securing contract proposals.

It takes approximately 4 to 4½ months for the Office Engineer Division to process a project. This begins with project submission. Eight to nine weeks are needed to Pre-bid meeting, one week to Letting, one week to Pre-award meeting, one week to Commission meeting, and four to six weeks to complete and execute the contract proposal. When the contract has been executed, the Office Engineer Division transfers the project to the Construction Division.

3.1.2.11 Local Government Coordination Division

The Local Government Coordination (LGC) Division acts as liaison between Federal, State and local governments in the processing of Federal-aid and State-aid projects. The LGC Division processes those local government projects initiated by any of Oklahoma's 77 counties or cities over 5,000 population. The LGC Division distributes plans throughout the Department for assistance in the review process, including those Divisions within Design.

The designer should note that local government projects normally use minimum standards based on the *County Roads Design Guidelines Manual* for county road projects. It should also be noted that FEMA laws are vested in the cities.

3.1.2.12 Field Divisions

There are eight field divisions within the State, and all have the basic responsibility for construction and maintenance of State highways. However, each division has unique design considerations, such as mountainous terrain to flat prairie, various rainfall amounts and soils that vary within the division. The designer should coordinate with the field division representative and closely coordinate all special design features with the applicable field division personnel.

3.1.3 <u>External Units</u>

This Section discusses the specific coordination activities between the Design Divisions and units external to ODOT.

3.1.3.1 Federal Highway Administration

The Federal Highway Administration (FHWA) administers the Federal-Aid highway program. Their basic responsibility is to ensure that ODOT complies with all applicable federal laws in their expenditures

3.1.3.4 Local Governments/Agencies

The designer may need to coordinate project development activities with local governments or agencies. Some of these may include:

1. town or city governments,
2. county governments,
3. Indian tribal councils,
4. local planning commissions,
5. regional planning agencies, and
6. local zoning boards. The responsibility for FEMA compliance is vested in the cities.

3.1.3.5 Oklahoma Turnpike Authority

When an ODOT project crosses over or connects with one of the turnpikes within Oklahoma, the designer needs to coordinate the design with the Oklahoma Turnpike Authority. ODOT is not responsible for conducting any work on any of the highways under the Oklahoma Turnpike Authority jurisdiction.

3.1.3.6 Railroad Companies

The designer's coordination with railroad companies is typically through the Planning Division. Together, they must ensure that the railroad has developed plans and specifications which are consistent with the proposed construction project scope. This may require a fairly extensive level of coordination with the Planning Division and with the railroad. Railroad safety projects are generally coordinated within the Traffic Engineering Division.

3.1.3.7 Utility Companies

The designer's coordination with utility companies is typically through the Right-of-Way (R/W) Division. Together, they must ensure that the utility has developed plans and specifications which are consistent with the proposed construction project. This may require a fairly extensive level of coordination with the R/W Division and with the utility company.

3.2 PROJECT PROGRESS CONTROL REPORTS

3.2.1 ODOT Project Management System

ODOT is presently using a computerized project management system called Automated Records, Tracking and Engineering Management Information System (ARTEMIS). ARTEMIS has the capability to track project schedules, resources and finances for preconstruction as well as the construction phases of a project. ARTEMIS assists management in performing the following functions:

1. Scheduling. ARTEMIS provides an activity flowchart for all activities or tasks for a given project. These activity flowcharts provide information on activity start and finish dates and its duration. This allows management to determine appropriate construction letting dates.

2. Critical Path. ARTEMIS is able to determine the critical path for a project. The critical path is the longest path by duration in a network of activities. This allows management to determine how any stoppage or slowdown of an activity on the critical path will affect the overall completion date of a project.

3. Resource Allocation. ARTEMIS allows management to effectively balance ODOT's work load with the available manpower. This allows management to adjust its use of ODOT personnel, overtime, consultants and/or construction letting dates.

4. Activity Barcharts. ARTEMIS prints out activity barcharts which allow ODOT to quickly determine the status of a project or project activity. This allows management to determine if additional resources may be required to meet the project schedule.

5. Financial Management. ARTEMIS tracks funding balances for a project. This allows management to determine if additional funding is required or if a reduced project scope should be considered.

ARTSTAT is an in-house utility developed by ODOT ARTEMIS personnel. ARTSTAT allows ODOT to effectively interact with the ARTEMIS software. Using ARTSTAT, the ARTEMIS Project Management Coordinator is responsible for initially establishing the project in the system. Once the project is on the system, the Project Engineer, or in some cases a designer from another Division, is responsible for keeping the project information up-to-date in the system. This includes both in-house and consultant-designed projects. Timely updates are critical to the success of the overall project management system.

Figures 3.2A, 3.2B and 3.2C present typical ARTEMIS outputs for selected projects.

3.2.2 Completion Percentages

Table 3.2A presents project completion percentages for various preconstruction activities. These values are only rules of thumb which provide a rough indication of project status.

CPM TYPE: PRE-CONSTRUCTION

JOBPIECE NO: 07954(14) PROJECT FID: MAM-8216 (011)

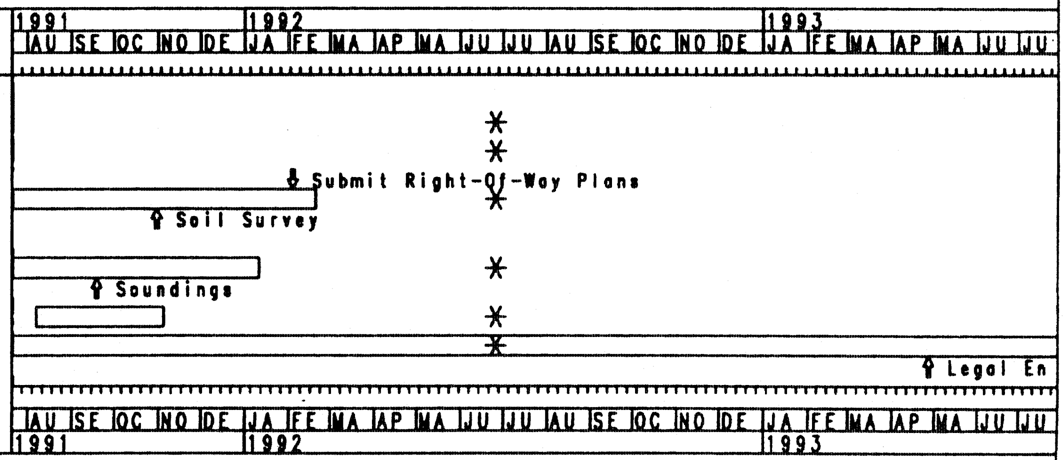
LETTING DATE: 25-JUN-92 FY: PRIORITY: 1 PROJECT TYPE: GRADE, DRAIN, BRIDGE & SURFACE

COMM DIST: VIII COMM ACT: 7-APR-86 COUNTY: TULSA DIV: PROGRAMS HIGH WAY: 51 LENGTH: 0
FUNDING EST: FED: 2475000 STE: 825000 OTH: 0 TOTAL: $3300000

PROJECT DESC: SH 51 (BA EXPY) • BRIDGES OVER S 161ST E. AVE & E 61ST ST. S. + CROSS ST.
- CONT: RDY. (GRP B) (SR 50/55000) (FY 92)

PE BRIDGE: GANJANATHAVAT/FRAKER PE RDWY: DON BRASHEAR PE OTHER:

DESCRIPTION	ESS	EFS	TF	PC
Planning	22-JUL-91	19-JUL-91	0	100
Survey	22-JUL-91	19-JUL-91	0	100
Design	22-JUL-91	19-FEB-92	65	13
Bridge	22-JUL-91	10-JAN-92	105	70
Traffic	7-AUG-91	4-NOV-91	120	0
Land Acq	22-JUL-91	29-JUL-93	-461	53



PROGRESS　　＊ LETTING DATE

ES EF　　—·— TIME NOW (22-JUL-91)　　CHART DATE: 22-JUL-91

Mainframe Barchart drawing system 22-JUL-91 1.52 pm　　Page 1 of 1

ODOT PROJECT SCHEDULING SYSTEM
(Sample Output for a Grading, Drainage, Bridge and Resurfacing Project)

Figure 3.2A

CPM TYPE: ENGINEERING

JOBPIECE NO: 09437(04) PROJECT FID: BRF-0257 (063)

LETTING DATE: 28-MAY-92 FY: PRIORITY: 1 PROJECT TYPE: BRIDGE & APPROCHES

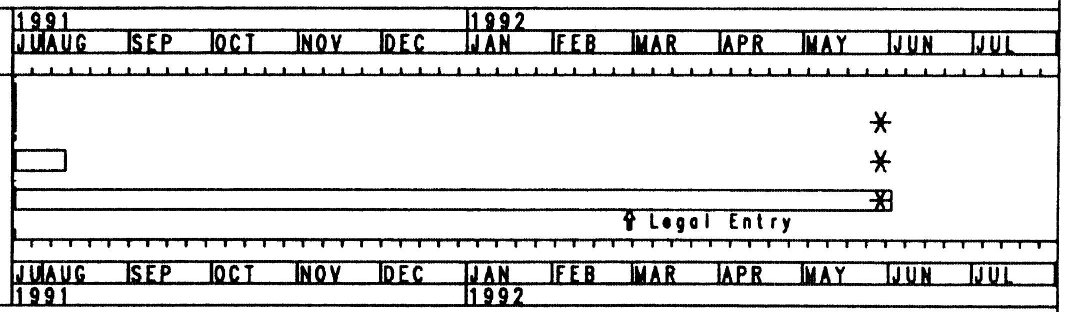
COMM DIST: VI COMM ACT: 5-FEB-90 COUNTY: ELLIS DIV: PROGRAMS HIGH WAY: 15 LENGTH: 0
FUNDING EST: FED: 1120000 STE: 280000 OTH: 0 TOTAL: $1400000 (9106)

PROJECT DESC: SH 15 @ BOGGY CR. & O/FLOW BRIDGES NEAR FARGO. (NBIS2320X1710 & X1719)
- CONT: PE=EC 438/MO/SOLTANI (SR 67/2400) (FY92)

PE BRIDGE: MOLLAIAN/CURB PE RDWY: SOLTANI PE OTHER: COBB

ENGINEERING CONTRACT: NAME: COBB

DESCRIPTION	ESS	EFS	TF	PC
Beginning to P.I.H.		21-JUL-91	0	100
P.I.H. to Final Plans	22-JUL-91	8-AUG-91	-234	96
LAND ACQ HAMMOCK	22-JUL-91	1-JUN-92	-527	38



PROGRESS
ES EF
LETTING DATE
TIME NOW (22-JUL-91)
CHART DATE: 22-JUL-91

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ODOT PROJECT SCHEDULING SYSTEM
(Sample Output for a Bridge Project)

Figure 3.2B

CPM TYPE: ENGINEERING

JOBPIECE NO: 02224(05) PROJECT FID: BHF-0127 (054)

LETTING DATE: 23-JAN-92 FY: PRIORITY: 1 PROJECT TYPE: BRIDGE REDECKING & REHABILITATION

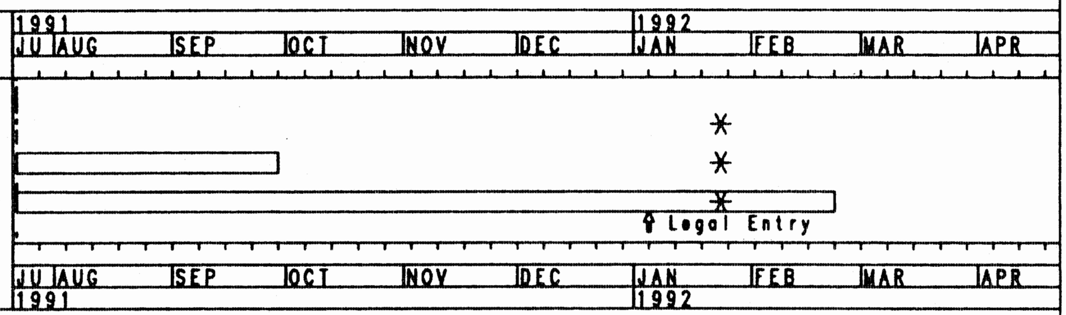
COMM DIST: VIII COMM ACT: 5-FEB-90 COUNTY: CREEK DIV: PROGRAMS HIGH WAY: 51 LENGTH: 0.721
FUNDING EST: FED: 3520000 STE: 1880000 OTH: 0 TOTAL: $5400000 (9106)

PROJECT DESC: SH 51 AT SALT CREEK OVER ARM OF KEYSTONE LAKE,APPROX. 2 MIS W. OF TULSA C/L
- CONT: \REHAB EXIST & CONST NEW PARALLEL BR)(CONSULT/MOE/SOLTANI (SR75/9500)(FY92)

PE BRIDGE: MOLLAIAN/CURB PE RDWY: SOLTANI PE OTHER: GROSSMAN & KEITH

ENGINEERING CONTRACT: NAME: GROSSMAN & KEITH

DESCRIPTION	ESS	EFS	TF	PC
Beginning to P.I.H.		9-MAR-89	0	100
P.I.H. To Final Plans	22-JUL-91	28-SEP-91	-281	88
Land Acq	22-JUL-91	22-FEB-92	-427	79



PROGRESS   * LETTING DATE

ES EF   TIME NOW (22-JUL-91)   CHART DATE: 22-JUL-91

Mainframe Barchart drawing system 22-JUL-91 1.51 pm   Page 1 of 1

ODOT PROJECT SCHEDULING SYSTEM
(Sample Output for a Bridge Redecking and Rehabilitation Project)

Figure 3.2C

Table 3.2A
PROJECT COMPLETION PERCENTAGES

***SURVEY (CONVENTIONAL)**	
Preliminary research and alignment studies completed	5%
Control surveys (horizontal and vertical) completed	10%
Preliminary data	15%
Land and property surveys completed	30%
Final alignment determined	35%
Centerline staked	40%
Topography obtained	55%
Cross sections completed	70%
Drainage survey completed	80%
Additional surveys completed (railroads, section line, etc.)	90%
Office completion of notes, maps and survey submittal	100%
***SURVEY (PHOTOGRAMMETRIC)**	
Reconnaissance aerial photography and large scale mapping obtained	5%
Preliminary research and alignment studies completed	10%
Control surveys (horizontal, vertical and photo) completed	15%
Design photography and photo control obtained	20%
Land and property surveys completed	35%
Compilation and drafting of topographic maps completed	40%
Final alignment determined	45%
Centerline staked and field profile obtained	50%
Topography obtained	60%
Drainage survey completed	70%
Additional surveys completed (railroad, section line, etc.)	75%
Office completion of notes, maps and field submittal	85%
Base P&P sheets and machine cross sections completed	100%
GRADING AND DRAINAGE PLANS	
Project data received and scheduled	15%
Title and P&P sheets prepared	20%
Submitted to Bridge Division	25%
Preliminary grades laid and submitted for pedological survey	30%
Pavement approval based on pedological findings	40%
Grading and drainage completed, ready for plan-in-hand	50%
Plan-in-hand completed	55%
Final R/W submitted	65%
Plans ready for summarization	75%
Submitted for checking	85%
Plans completed and ready for submission to Office Engineer	90%
Submitted to Office Engineer	100%
FUNCTIONAL PLANS	
Project data received and scheduled	15%
Title and P&P sheets prepared	20%
Submitted to Bridge Division	25%
Geometric layout and preliminary grades laid and submitted	30%
Proposed typical section for roadway classification	40%
Grading and drainage proposed, ready for plan-in-hand	50%
Plan-in-hand completed	60%
Plans ready for summarization	70%
Submitted for checking	80%
Plans completed	100%

* Order may vary depending upon the project.

Table 3.2A (Continued)
PROJECT COMPLETION PERCENTAGES

SURFACING PLANS	
Subgrade soils report received	20%
Alternate pavement design completed	25%
Pavement type selected	30%
Typical sections drawn and approved	40%
Plans ready for plan-in-hand	50%
Plan-in-hand completed	55%
Plans completed and summarized	75%
Plans checked	85%
Ready for submission to Office Engineer	90%
Submitted to Office Engineer	100%
EROSION CONTROL PLANS	
Field measurements received	15%
Areas taken from cross sections	35%
Ready for summarization	45%
Quantities summarized	60%
Final sheets prepared	75%
Submitted to Roadside Development Branch	85%
Submitted to Office Engineer	100%
BRIDGE PLANS	
Ready for plan-in-hand	15%
Foundation studies	20%
Plan-in-hand completed	25%
Geometrics and hydraulics completed, plan-in-hand revisions made	35%
Superstructure design completed	55%
Plans detailed and checking completed	75%
Summary of quantities completed	80%
Plans reviewed and drafting finalized	90%
PS&E and final review completed, submitted to Office Engineer	100%
SIGNING PLANS	
Project data received and scheduled	10%
Prepare signing plan sheets and title sheet	40%
Plan-in-hand completed	45%
Prepare sign summary, pay quantities and notes	75%
Submitted for checking to Division, FHWA, etc.	80%
Address comments from checking	90%
Plans completed and ready for submission to Office Engineer	95%
Submitted to Office Engineer	100%
LIGHTING PLANS	
Project data received, evaluated, warranted and scheduled	5%
Prepare title and plan sheets	25%
Prepare and submit preliminary lighting layout and pertinent information for review.	45%
Correct preliminary design and submit revised plans and pertinent information for P-I-H	65%
Plan-in-hand completed and plan-in-hand letter of items discussed	70%
Correct P-I-H revisions and submit revised plans and pertinent information for final review and approval	80%
ODOT final review and approval of plans	90%
Correct and submit final plans and pertinent information for submission	95%
Agreement with City, special provisions and plans submitted to Office Engineer	100%

Table 3.2A (Continued)
PROJECT COMPLETION PERCENTAGES

TRAFFIC SIGNAL PLANS	
Review project site and photos taken	10%
Received data, engineer's estimate and schedule	15%
Field survey of project location completed	20%
Title and plan sheets prepared	45%
Prepare plans for plan-in-hand	55%
Plan-in-hand completed	60%
Plan-in-hand revisions completed	65%
Plans reviewed and checked	75%
Wiring diagram completed	80%
Timing schedule with sequence and other details completed	85%
Plans completed; ready for submission	95%
Plans submitted to Printing Services; set-up submitted to Office Engineer	100%
TRAFFIC OPERATIONS PLANS	
Project data received and attend plan-in-hand	20%
Plan sheets prepared	45%
Planned detour route chosen and sheets prepared	75%
Summary of quantities and traffic control notes completed	95%
Plan sheets submitted for checking	98%
Plans submitted to Project Engineer	100%

3.3 MEETINGS

3.3.1 General

Good communications are a necessity. It is imperative that all meetings be well planned, attended by the proper employees, and the information be disseminated to the affected people in a timely manner. The following will identify some of these meetings and indicate the purpose, the attendees and the reporting of the results.

3.3.2 Administrative Meetings

3.3.2.1 Transportation Commission Meetings

The Transportation Commission Meetings will be attended by the Assistant Director-Design and any division head or other employee who has a primary interest and is requested to attend. The Commission Secretary sends copies of the Commission Minutes to each Division head. It is his responsibility to inform his subordinates of any information affecting his operation or personnel.

Some items which require Commission action are:

1. Commission Policies or Resolutions,
2. Expenditures of Construction Funds,
3. Regulatory Powers Delegated by the Legislature,
4. Changes in the Long-Range Program,
5. Precedent Establishing Items,
6. Highway and Federal-aid Route Changes,
7. Contract Lettings,
8. Contract Awards, and/or
9. Federal-aid Secondary Route Changes.

Items requiring action by the Commission may be prepared using the format shown in Figure 3.3A and routed complete with appropriate exhibits and back-up material through channels to the Assistant Director-Design. Items should be received by the Assistant Director-Design prior to the Department Agenda Meeting (normally held on Wednesday, 1½ weeks prior to the Commission Meeting) to be presented to the Commission Meeting on the first Monday of the following month. Under unusual circumstances, items may be added after the agenda meeting, but these additions are to be minimized. Exhibits are encouraged and can be either hand-outs or mounted graphics legible for a minimum distance of 20 feet. Graphics should be prepared so that color-blind participants will be able to comprehend the message. Notification of the Commission action and the mounted graphics will be returned to the appropriate Division after the meeting.

3.3.2.2 Division Engineers Meetings and Other General Interdepartmental Meetings

These meetings are conducted to improve communications between the Field Division and Central Office and should be attended by the Division Head or his representative. Each division is responsible for ensuring that specific items are on the agenda and handling the presentation.

FOR PRESENTATION AT COMMISSION MEETING ON

AGENDA ITEM NO:

SUBJECT:

COUNTY:

Approval is recommended.

Note: User should check with the Division Engineer for proper form.

COMMISSION REQUESTS
(Sample)

Figure 3.3A

3.3.2.3 Project Control Meetings and Meetings to Review the Lettings for the Following Two Months

Each of these meetings consists of a Design meeting and a Department meeting. The Design meetings are attended by the division heads and/or the project control coordinators. The Department meetings are attended by the Assistant Director-Design and the Chief Engineer. These meetings are conducted to ascertain and to select the projects to be let to contract within the next two months.

3.3.2.4 Pre-Bid Meeting

Pre-bid meetings are conducted by the Office Engineer Division for all construction projects. The Pre-bid meetings are attended by contractors, the Project Engineer, consultants (for consultant-designed projects) and other ODOT personnel who may have had a significant input into the design of the project (e.g., Bridge Division, R/W Division, Traffic Engineering Division, Field Divisions). The Pre-bid meeting is held to provide contractors with the opportunity to ask questions on the plans and to clarify possible confusing items. The Pre-bid meeting should not be held until the contractors have had sufficient time to review the plans, but should be held at least 7 days before the bid opening.

3.3.2.5 Lettings

Lettings (bid openings) are conducted by the Office Engineer Division. They are usually attended only by Department personnel who have a specific duty to perform during the proceedings.

3.3.2.6 Bid Review Meeting

This meeting is usually held preceding the Transportation Commission Meeting to review the bids and recommend projects to be awarded. The Chief Engineer will represent the Office of Design; however, the Assistant Director—Design will be consulted prior to the meeting and will be informed of the pertinent results of the meeting.

3.3.2.7 Staff Meetings

Staff meetings will be held periodically and should be attended by the Division heads. The Division head should designate someone to represent him if he cannot be present. Staff meetings should provide for a two-way flow of communications, and Division heads should feel free to submit items or questions of general interest at the staff meetings.

3.3.2.8 Supervisor's Meetings

Supervisor's meetings will be held periodically and are conducted to improve the communications from squad supervisor level to Section, Branch or Division level. The scheduling of these meetings is not time specific but should be conducted as needed in an attempt to locate problem areas or to keep everyone informed of ODOT operations. All supervisors should attend these meetings and participate in the discussions.

3.3.3 Educational or Technical Meetings at National, Regional or Local Levels

Persons who are interested in attending out-of-state meetings should submit, through proper channels, a request for out-of-state travel and should attach justification for their attendance. For meetings in Oklahoma during working hours, approval should be obtained from the appropriate Division Engineer. Persons authorized to attend the annual AASHTO and Regional AASHTO meetings will be notified by Department memorandum.

3.3.4 Project Meetings

3.3.4.1 Scoping Meeting

During the pre-design stage of a project, a scoping team is formed to determine if the preliminary scope of a project is feasible and reasonable within the funding programmed. The scoping team usually conducts an on-site inspection to discuss all pertinent aspects of the project. Members of the scoping team include the project engineer and ODOT representatives from Planning, Survey, Right-of-Way, Utilities, the Field Division, Traffic Engineering, Bridge, Materials, FHWA and other ODOT personnel as deemed necessary.

3.3.4.2 Plan-In-Hand Meeting

Plan-in-hand meetings are scheduled and coordinated by the Project Engineer. These meetings normally include all involved ODOT personnel, FHWA and local authorities as applicable for the project. The plan-in-hand meeting is to review and resolve project elements prior to the preparation of final plans. See Chapters Two and Four for further information.

3.3.4.3 Pre-Construction Meeting

The Pre-construction meeting is held after the project has been awarded to the contractor and prior to the start of construction. Typically, this meeting is attended by the contractor, the resident engineer and the Equal Employment Officer and may include other ODOT personnel as deemed necessary. This meeting is held to discuss the project, the contract, proposed construction procedures, hiring practices and any other pertinent project details.

3.3.4.4 Final Inspection Meeting

Prior to final acceptance of the construction project by ODOT, a final inspection meeting is held to confirm that the contractor fully completed all construction work according to the contract. This meeting is normally attended by the resident engineer, the contractor, FHWA and other ODOT personnel as deemed necessary.

3.3.4.5 Special Meetings

Special meetings may include those with local communities, legislators, or the general public in response to specific project impacts and/or information. When such a meeting is held, it will be the responsibility of the ODOT representative to prepare a brief report indicating who was in attendance, when, why and where the meeting was conducted and to discuss the results of the meeting. The chain of command will be observed in determining who should receive the report. See Section 3.9.

3.4 CORRESPONDENCE

3.4.1 General

The writer must exercise common sense in correspondence content and in determining who should sign letters. He is also responsible for its distribution to ensure that any affected group or individual receives a copy. Design has established the following policy regarding the signing of outside correspondence:

1. The Director will, as a rule, sign all letters to U.S. Congressmen, Federal offices and agencies outside of Oklahoma, the Governor, legislators and members of the Commission. All letters for signature by the Director will be routed through the Assistant Director-Design for his review and approval prior to going to the Director.

2. All letters establishing design policy should be approved by the Assistant Director-Design, whether its destination is intra- or inter-divisional.

3. Correspondence which is interdivisional or outside of ODOT and does not establish policy will be signed by the Division Engineer or his designee. All intra-divisional correspondence should be routed through the Division Engineer for his approval and information.

4. In general, the Division Engineer will decide who signs project-related correspondence.

3.4.2 Incoming Correspondence

The divisions shall establish their own policies regarding the opening and handling of incoming mail. Each individual receiving mail should ascertain who needs to be notified and/or given copies of such correspondence. Because ODOT's mail facilities are not staffed to handle mail which is personal, the use of the mail room for this purpose is discouraged.

3.4.3 Letter Headings

All correspondence which is sent outside of ODOT will be placed on "State of Oklahoma Department of Transportation" stationery. In order to facilitate filing, headings should reference the appropriate project or subject matter.

Correspondence which is within ODOT will be placed on the standard DH Form 7 stationery. This letter should contain a suitable subject description and/or project number.

3.5 DISTRIBUTION OF PRELIMINARY PLANS AND DRAWINGS

Considerable discretion must be exercised in the issuance of preliminary plans to outside interests prior to right-of-way acquisition to prevent personal advantage by speculators. Preliminary plans are often misinterpreted, which can cause problems later when negotiating for right-of-way from design plans which have changed in the interim. All preliminary plans will be issued along the following guidelines and with the approval of the Division Engineer:

1. Preliminary plans may be issued to official planning commissions, to local political subdivisions whose planning may be affected by the highway location, to officials of applicable Federal and local agencies and to other official bodies having a need for such advance information, provided that the plans are dated and clearly marked or stamped "PRELIMINARY PLANS" and that the receiving group understands that the information is being furnished solely for the purpose of coordinated planning, and that they will provide written assurances the information will remain strictly confidential.

2. Partial plans may be furnished to utility companies and railroads where their facilities may be affected by the highway construction.

3. Persons other than those mentioned above are welcome to review the plans in the various stages of preparation, but they are not to be furnished with prints of any of the plans until the right-of-way is purchased or under condemnation. Prints at this stage may be obtained only through the Right-of-Way Division. All prints of plans issued prior to advertisement for bid or prior to acquisition of right-of-way will be stamped "PRELIMINARY PLANS" and dated.

4. Plans for the general public are only available after the plans have been submitted for letting. Plans at this stage may be obtained through the Reproduction Branch within the Office Services Division.

3.6 COST ACCOUNTING

All ODOT employees are required to record the proper cost accounting code on their A-9 time sheets each month. This allows the comptroller to track the various project costs for ODOT. The activity numbers allow the Project Managers to track how much time is spent on different parts of a project. Table 3.6A summarizes the accounting and activity code tables used by Design. These codes are subject to continuous update and/or change. Therefore, the user should check with his supervisor for the latest codes.

Table 3.6A

ACCOUNTING AND ACTIVITY CODE TABLES

TABLE NUMBER	TITLE
Table 3.6B	Division Account Number for Design
Table 3.6C	Activity Codes for Bridge Division
Table 3.6D	Activity Codes for Urban Design Division
Table 3.6E	Activity Codes for Rural Design Division
Table 3.6F	Activity Codes for Survey Design Division
Table 3.6G	Activity Codes for Traffic Engineering Division

Table 3.6B

DIVISION ACCOUNT NUMBERS FOR DESIGN

UNIT	ACCOUNT NUMBER
Assistant Director - Design	330910
Bridge Division - Overhead	331910
Bridge Division - Structural	331920
Bridge Division - Field Service	331930
Bridge Division - Hydraulics	331940
Bridge Division - Project Engineering	331950
Bridge Division - County Bridges	331960
Urban Design Division - Overhead	332910
Urban Design Division - Project Engineering	332920
Rural Design Division - Overhead	333910
Rural Design Division - Project Engineering	333920
Rural Design Division - Design Review	333930
Rural Design Division - Roadside Management	333940
Rural Design Division - Automation & Graphics	333950
Rural Design Division - Roadway Drainage	333960
Rural Design Division - Engineering Support	333970
Survey Division - Overhead	334910
Survey Division - Aerial Survey	334920
Survey Division - Project Engineering	334930
Traffic Engineer Division - Overhead	335910
Traffic Engineer Division - Planning & Safety	335920
Traffic Engineer Division - Operations	335930
Traffic Engineer Division - Design	335940
Traffic Engineer Division - Services	335950

Table 3.6C

ACTIVITY CODES FOR BRIDGE DIVISION

DESCRIPTION	ACTIVITY NUMBER	X
Administrative		
General	440.001	
Correspondence	440.002	
Special Report	440.003	
Meetings	440.004	
Conferences and Seminars	440.005	
Policy and Procedures	440.006	
Project Program (ARTEMIS)	440.007	
Secretarial	440.008	
Agreements (Utilities, 404, etc.)	440.009	
Construction Coordination		
Construction Coordination	440.x20	0 = State Highway System
Forming Data	440.x21	
Falsework Review	440.x22	
Shop Drawings	440.x23	1 = Local Gov't System
As-builts	440.x24	
Geotechnical Studies	440.x25	
Value Engineering	440.x26	
Special Provisions	440.x27	
Computer Systems Programming & Maintenance	440.x28	
AASHTO Specifications	440.x29	
Hydraulics		
General Hydraulic Study	440.x30	0 = State Highway System
Special Hydraulic Study	440.x31	
Hydraulic Report	440.x32	
Risk/Cost Evaluation	440.x33	1 = Local Gov't System
Channel Control and/or Bank Protection	440.x34	
Hydrology	440.x35	

DESCRIPTION	ACTIVITY NUMBER	X
Bridge Inspection		
Routine Bridge Inspection Evaluation of Fracture Critical Elements Evaluation of Scour Critical Structures Underwater Inspection Inspection of Major Bridges Special Inspection (Damage + others) First Bridge Inspection Review Inspection Reports by Consultants Maintain/Update Bridge Data Base Overload Permits Bridge Repair Plans Load Capacity Rating Analysis Equipment Operation Review Inspection Report by ODOT Personnel	440.x40 440.x41 440.x42 440.x43 440.x44 440.x45 440.x46 440.x47 440.x48 440.x49 440.x50 440.x51 440.x52 440.x53	0 = State Highway System 1 = Local Gov't System
Design Activities		
Preliminary Bridge Plans Preliminary & Comparative Cost Estimates Final Right-of-Way Needs Design of Final Plans Design Check of Final Plans Detailing and Drafting of Final Plans Check Final Plan Details	440.x60 440.x61 440.x62 440.x63 440.x64 440.x65 440.x66	0 = State Highway System 1 = Local Gov't System 2 = CADD
Special Structures		
Bank Protection Plans Retaining Walls Traffic Structures Roadway Design Structures Special Structures (Buildings, etc.) Standard Drawings	440.x80 440.x81 440.x82 440.x83 440.x84 440.x85	0 = State Highway System 1 = Local Gov't System 2 = CADD
Consulting Projects		
Review of Consultant Hydraulic Report Preliminary Plan Review of Consultant Plans Design Review of Final Consultant Plans Final Construction Plan Review of Consultant Plans	440.x90 440.x91 440.x92 440.x93	0 = State Highway System 1 = Local Gov't System 2 = CADD

Table 3.6D

ACTIVITY CODES FOR URBAN DESIGN DIVISION

DESCRIPTION	ACTIVITY NUMBER	X
Project Manager		
Review Consultant Plans	330.201	
Project Meeting/Field Inspection	330.202	
Training/Conferences	330.203	
Project/Contract Estimates	330.204	
Correspondence/Reports	330.205	
Project Management	330.206	
Traffic Analysis	330.207	
Pavement Design	330.208	
Supervision/Administration	330.209	
Design Squad		
Title Sheet	330.x01	0 = Non-CADD
Pay Quantity Sheet	330.x02	
Summary Sheet	330.x03	1 = CADD
Survey/Alignment Sheet	330.x04	
Typical Section Sheet	330.x05	
Special Detail Sheet	330.x06	
Plan and Profile Sheet	330.x07	
Drainage Layout Sheet	330.x08	
Joint Layout Sheet	330.x09	
Cross Section Sheet	330.x10	
Earthwork	330.x11	
Hydraulics	330.x12	
Field Inspection	330.013	
Training/Conferences	330.014	
Plan Review	330.015	
Supervision/Administration	330.016	

Table 3.6E

ACTIVITY CODES FOR RURAL DESIGN DIVISION

DESCRIPTION	ACTIVITY NUMBER	X
Project Manager		
Field Inspection	220.901	
Review Consultant Plans	220.902	
Project Management	220.903	
Supervision & Administration	220.904	
Meetings/Conferences	220.905	
Design Squad		
Title Sheet	220.x01	0 = Non-CADD
Summary Sheet	220.x02	1 = CADD
Special Detail Sheet	220.x03	2 = Checking
Typical Section Sheet	220.x04	8 = Automation & Graphics
Plan & Profile Sheet	220.x05	
Cross Section Sheet	220.x06	
Quantities	220.x07	
Drainage	220.x08	
Joint Field Inspection	220.x09	
Comparative Estimates	220.x10	
Meetings	220.x11	
Correspondence	220.x12	
Consultant Plans	220.x13	
Miscellaneous/Special Projects	220.x14	
Supervision/Training	220.x15	

DESCRIPTION	ACTIVITY NUMBER	X
Automation & Graphics		
Conduct Special Investigation & Write Report	220.800	
Computer File Maintenance	220.805	
Field Inspection	220.810	
Field Liaison/Public Relations	220.815	
ACF (System Security Maintenance)	220.820	
Correspondence/Reports	220.825	
Develop CADD Drawings/Computer Schedules	220.830	
Hardware Maintenance	220.835	
Instruction of ODOT Personnel (Teaching)	220.840	
Job Related Training (Received)	220.845	
Meetings/Conferences	220.850	
Supervision and Administration	220.855	
Write, Revise, Maintain, Computer Programs	220.860	
Write, Revise, & Maintain Manuals	220.865	

Table 3.6F

ACTIVITY CODES FOR SURVEY DIVISION

DESCRIPTION	ACTIVITY NUMBER	X
All Survey Units		
General	110.001	0 = Non-CADD
Correspondence and Reports	110.002	1 = CADD
Conferences, Seminars and Training	110.003	
Mosaics	110.004	
Survey Controls	110.x05	
Alignment and Topography	110.x06	
Cross Sections and Levels	110.x07	
Drainage	110.x08	
Utilities	110.x09	
Land Surveys	110.x10	

Table 3.6G

ACTIVITY CODES FOR TRAFFIC ENGINEERING DIVISION

DESCRIPTION	ACTIVITY NUMBER	X
All Traffic Units		
Accident Coding	550.001	0 = Non-CADD
Accident Studies	550.002	1 = CADD
Conferences and Seminars	550.003	
Correspondence	550.x04	
Counseling and Coordinating	550.005	
Design Studies	550.006	
Division Meetings	550.007	
External Liaison	550.008	
Field Inspection	550.009	
Field Operational Review	550.010	
Intra-Departmental Meetings	550.011	
Internal Liaison	550.012	
Lighting Plan Development	550.x13	
Plan-in-hand	550.014	
Pre-Work Conference	550.015	
Project Management	550.x16	
Review Consultant Plans	550.017	
RR Project Development	550.018	
Safety Imp Plan Development	550.x19	
Schools and Training Sessions	550.020	
Signal Plan Development	550.x21	
Signing & Striping Plan Development	550.x22	
Skid Studies	550.023	
Specifications and Agreements	550.024	
Speed Studies	550.025	
Staff Engineering	550.x26	
Standards	550.x27	
Traffic Control Plan Development	550.x28	
Video Log - Field Filming	550.029	
Video Log - Office	550.030	

3.7 CONSULTING ENGINEER SERVICES

3.7.1 General

The services of consulting engineers will be used by ODOT when such services are determined to be in the best interest of ODOT to accommodate the construction program. More specific information is presented in the Transportation Commission Rules, Regulations and Policies. The Rules should be referenced for detailed information on consulting services.

3.7.2 Qualifications

Each consulting engineering firm which desires to provide services to ODOT is required to prequalify with the Office of Public Affairs. Prequalification will be determined based on a careful examination of each firm's staff training, experience in highway design, staff size, special expertise and past record of performance on highway engineering work. The Office of Public Affairs maintains the prequalification list. The list is updated annually.

Interested consultants should check with the Office of Public Affairs for the current prequalification list and check with the Transportation Commission Rules for detailed information on consultant qualifications.

3.7.3 Selection of Design Projects Requiring Engineering Services of Consultants

3.7.3.1 General

It is the policy of the Oklahoma Transportation Commission that design work will generally be performed by ODOT engineering forces except where specialized services are required which are not available within ODOT or where in-house engineering forces cannot complete the necessary work in time to meet construction letting schedules.

3.7.3.2 Justification of Use of Consulting Engineering Services

Request for proposals (RFP) of projects requiring outside engineering services will be initiated by the appropriate division engineer. A written RFP will be prepared for each project by the division engineer for submission to the Assistant Director — Design and will include a detailed documentation of the following items:

1. a detailed description of engineering services required for the project and a time schedule for the performance of such services;

2. a study of the capability of staff personnel to accomplish the work. This analysis may be in terms of the technical competence of staff forces or in terms of manpower and work load on a squad-by-squad basis. (The scheduling of work by a squad will be considered evidence that the work can be handled with staff forces.); and

3. an estimate of costs of such services using the form shown in Figure 3.7A.

3.7.3.3 Authorization to Contract for Engineering Services

The RFP will be submitted to the Assistant Director — Design, who will review the RFP emphasizing the description and scope of engineering services to be provided, the estimated cost and the time schedule proposed for their accomplishment. The Assistant Director — Design will make a written recommendation to the Director (or Chief Engineer) for his evaluation and

OKLAHOMA STATE HIGHWAY DEPARTMENT

ENGINEERING CONTRACT COST ESTIMATE

E. C. NO. ____________ SWO NO. ____________

COUNTY ____________ CITY ____________ PROJECT NO. ____________

HIGHWAY NO. ____________ DESCRIPTION ____________

NAME OF ORGANIZATION SUBMITTING ESTIMATE ____________

Indicate appropriate type of services at top of column such as: Field Survey, Functional Plans, Bridge Design Roadway Design, Etc.

DIRECT SALARY COSTS	MAN HOURS	COST	MAN HOURS	COST	MAN HOURS	COST
Engineers		$		$		$
Designers						
Draftsmen						
Surveyors						
Supervision						
Others (Specify) ____________						
____________						
Subtotal		$		$		$
PAYROLL ADDITIVE (Vacation, sick leave, retirement, FICA, etc.)	PERCENT					
	$		$		$	
DIRECT NON-PAYROLL COSTS						
Materials & supplies	$		$		$	
Reproduction						
Data Processing						
Photogrammetric mapping						
Travel expenses						
Equipment rental						
Outside engr. consultants						
Other (specify) ____________						
Subtotal - All Direct Costs	$		$		$	
INDIRECT COSTS (Administration, rent, utilities, telephone, etc.)	$		$		$	
Subtotal - Direct & Indirect	$		$		$	
PROFESSIONAL FEE (Profit)	$		$		$	
TOTAL ESTIMATED FEE	$		$		$	

Prepared by ____________ Date ____________
Signature

ENGINEERING CONTRACT COST ESTIMATE
(Sample Form)

Figure 3.7A

approval. Upon written approval by the Director, the selection of a consulting engineer will be made in accordance with the procedure for selection of consulting engineering firms (Section 3.7.4).

3.7.4 Selection of Consulting Engineering Firms for Specific Projects

After the engineering needs for a project have been determined, the following process will be used to select a consultant:

1. The Assistant Director — Design or his designee will contact the Office of Public Affairs for the list of prequalified firms for the specific type of project work.

2. An announcement will be sent to each of the applicable prequalified firms.

3. Interested firms will respond to the announcement with a letter of interest.

4. From the list of interested firms, the Consultant Selection Committee will select a short list of 3 to 5 firms for further evaluation. This selection will be based on the qualifications and capabilities of the prospective firms.

5. Typically, the firms on the short list will be scheduled for an interview.

6. The Consultant Selection Committee will evaluate each firm considering the following factors as a guide:

   a. the firm's general and professional reputation;

   b. experience and professional competence of staff in the particular area of work needed;

   c. specialized knowledge and ability of principals and staff;

   d. past performance on similar projects with special attention to quality of work done, meeting time obligations, cooperativeness and creativity in pursuance of the work;

   e. availability in terms of office location. Consideration will be limited to firms who have or agree to establish a working office at a place acceptable to the Department for ready availability for review and discussion of the work involved;

   f. capability to perform the work within the required time. Existing staff capacity and work load within the required time. Existing staff capacity and work load will be carefully weighed in determining the work capability;

   g. determination of whether the person directly in charge of the work will be a registered professional engineer in the State of Oklahoma;

   h. interviews with the consultant or visits to his offices may be required where the Committee is not familiar with the firm;

   i. as practical, firms located in the area where the work will be performed will be given some preference in selection; and

   j. financial ability to perform the work.

7. The Consultant Selection Committee will rank the firms for the project. The committee will submit the ranking to the Director with a recommendation for selection.

8. After approval by the Director, ODOT will negotiate with the selected firm according to the Department's approved procedures for negotiation. If the negotiations with the selected firm are unsuccessful, negotiations will proceed with the No. 2 firm on the list.

9. After successful negotiations are completed, ODOT will execute the project Agreement.

3.7.5 Completion Report on Engineering Contracts

3.7.5.1 General

Upon completion of the work required under the engineering contract, the project engineer or engineers who have been involved in the work will prepare a written report on the performance of the consultant. To ensure that all terms and conditions of the contract have been met and that all services have been performed, the final payment will not be made to the consultant until the written completion report is submitted.

The following evaluation forms will be completed to rate the performance of the consulting engineer:

1. Consultant Quality Appraisal. Quality Appraisal Form (A) will be completed by three individuals designated by the Assistant Director — Design.

2. Construction Plan Performance. Quality Appraisal Form (B) will be completed by the appropriate division in the ODOT central office after final plan approval.

3. Functional/Preliminary Plans. Quality Appraisal Form (C) will be completed by the appropriate division in the ODOT central office after final plan approval.

A blank copy of each Quality Appraisal Form is presented in the following pages.

As indicated in the evaluation forms, the completion report will include all pertinent information concerning the quality of work, meeting completion deadlines, compliance with contract and design standards, initiative and creativeness, participation of principals in the project, and the general responsiveness and cooperativeness of the consultant and his representatives. Special care will be taken in rating the performance of a firm when there is some question on the advisability of further use of their services for the particular type of work done. The completion report will be confidential and filed with the Consultant Selection Committee.

3.7.5.2 Construction Report on Quality of Plans

When warranted, in addition to the completion report filed at the close of the engineering contract, construction reports will be requested by the Consultant Selection Committee during the construction progress and as the project nears completion. Quality Appraisal Form (D) "Contract Constructability" will be completed by the appropriate Field Division after the project has been completed. The Form will be filed with the Chief Engineer with copies to the Assistant Director — Design, the Consultant Selection Committee and the consultant.

3.7.5.3 Unsatisfactory Completion Report

Where an unsatisfactory report indicates that the firm should not be used for that type of work in the future, a letter will be written by the chairman of the Consultant Selection Committee, with the concurrence of the Director, outlining the deficiencies in the consultant's performance and notifying him

QUALITY APPRAISAL FORM (A)

OKLAHOMA DEPARTMENT OF TRANSPORTATION

CONSULTANT QUALITY APPRAISAL

Name and Address:	Expertise:	Date of Appraisal:
______________________ ______________________ ______________________	____________________ ____________________ ____________________	____________________

Rating categories	Points	Rater A	Rater B	Rater C
Schedule	10			
Attitude	05			
Accuracy of Plans (Rdy, Br, R/W)	20			
Legibility/Clarity of Plans	15			
Traffic Handling Plans	10			
Quality Control Plan	05			
Constructability	10			
Compatibility of Plans Specifications, and Special Provisions	15			
Conformance with ODOT Requirements	10			
TOTAL	100			

Notes and Recommendation:

Overall Rating:

Excellent (90-100) ______________

Good (80-89) ______________

Satisfactory (70-79) ______________

Unsatisfactory (-69) ______________

Prepared By: ______________________________

QUALITY APPRAISAL FORM (B)

CONSTRUCTION PLAN PERFORMANCE

	Excellent	Good	Fair	Poor
1. Surveying/Mapping	____	____	____	____
2. Roadway Design				
a. Plan and Profile	____	____	____	____
b. Geometrics	____	____	____	____
c. Special detail drawings	____	____	____	____
d. Capacity Analysis	____	____	____	____
3. Roadway Drainage	____	____	____	____
4. Traffic Design				
a. Signing, Marking	____	____	____	____
b. Signals	____	____	____	____
c. Traffic Control	____	____	____	____
d. Lighting	____	____	____	____
5. Materials				
a. Geotechnical Investigations	____	____	____	____
b. Pavement Design	____	____	____	____
6. Bridges				
a. Hydraulic	____	____	____	____
b. Structure Design	____	____	____	____
c. Geometric Accuracy	____	____	____	____
d. Foundation Design	____	____	____	____
7. Small Structures (Standards, Ret. Walls, Box Culverts)				
a. Hydraulics	____	____	____	____
b. Accuracy of Plans	____	____	____	____
8. Landscaping Plans/Details	____	____	____	____
9. Utility Relocation Plans	____	____	____	____
10. Special Provisions	____	____	____	____
11. Final Quantities/Summary Sheet	____	____	____	____
12. Cost Estimates	____	____	____	____
13. Environmental Evaluation-Consideration and Permits	____	____	____	____
14. Public Hearing/Meeting Presentations	____	____	____	____
15. Value Engineering Evaluations	____	____	____	____
16. Constructability Review	____	____	____	____

Explain adverse comments: ____

____

____

R/W PLANS PERFORMANCE

	Excellent	Good	Fair	Poor
1. Schedule (Timeless)	____	____	____	____
2. Attitude (Cooperation)	____	____	____	____
3. Accuracy	____	____	____	____
4. Legibility/Clarity of Plans (Quality)	____	____	____	____

Explain adverse comments: ____

____

____

QUALITY APPRAISAL FORM (C)

FUNCTIONAL/PRELIMINARY PLANS

	On Time	Late	Adequate	Inadequate
a. Preliminary permit applications	____	____	____	____
b. Utility relocation plans	____	____	____	____
c. R/W requirements	____	____	____	____
d. Environmental reports	____	____	____	____
e. Public meeting/hearing requirements	____	____	____	____
f. Assigned project staff	____	____	____	____
g. Contract administrative procedures				
(1) Costs and progress reports	____	____	____	____
(2) Project-related correspondence	____	____	____	____
(3) Changes in subconsultants or key staff	____	____	____	____
h. Design notes and computations	____	____	____	____
i. Progress meetings	____	____	____	____
j. Draft Special Provisions	____	____	____	____
k. Preliminary quantity take-off	____	____	____	____
l. Preliminary cost estimate	____	____	____	____
m. Value engineering	____	____	____	____
n. Constructibility	____	____	____	____
o. Preliminary Design Submittal	____	____	____	____

Comments: ____

____

____

QUALITY APPRAISAL FORM (D)

CONTRACT CONSTRUCTABILITY

	Excellent	Good	Fair	Poor
1. Accuracy of Survey Data	____	____	____	____
2. Accuracy of Geometric Data	____	____	____	____
3. Existing and Proposed Plan Details	____	____	____	____
4. Legibility and Clarity of Plans	____	____	____	____
5. Sequence of Construction/ Traffic Control	____	____	____	____
6. Utility Relocation Plans	____	____	____	____
7. Accuracy of R/W Information	____	____	____	____
8. Accuracy of Quantities	____	____	____	____
9. Completeness of Special Provisions	____	____	____	____

Comments: ____

____

____

CONTRACT ADMINISTRATION

	Yes	No
1. Construction Problems resulting from Plans and Specifications	____	____
2. Change order required due to plans or specifications	____	____
3. Contractor claim submitted due to plans or specifications	____	____
4. Liquidated damages assessed against Contractor	____	____

Explain any yes responses: ______________________________

____

____

that future contracts in this area of work will not be considered. A copy of the letter will be sent to the Director and the Assistant Director — Design.

3.7.5.4 Appeals

Consultants removed from the list of qualified engineering firms or removed from consideration in certain areas of work may, if they feel that their disqualification was unwarranted, appeal their case to the Director who will be the final arbiter in each case. Consultants disqualified may, after making appropriate changes in their organization and operating procedures, make a new application to the Consultant Selection Committee for their consideration. An adverse action of the Consultant Selection Committee on an application may be appealed to the Director for arbitration if a dispute arises concerning prequalification.

3.7.6 Engineering Contract Supervision

3.7.6.1 General

The general supervision of engineering contracts is maintained by the Assistant Director — Design with the administrative supervision delegated to the Engineering Contracts Officer and the technical supervision delegated to the Project Manager. When more than one ODOT division is involved, a lead division must be selected.

3.7.6.2 Supervision by the Engineering Contracts Officer

The administrative supervision of the engineering contracts is maintained by the Engineering Contracts Officer or his designee and includes the following:

1. checks the progress of the contract through a monthly review of progress reports, calendar time and correspondence received from the Consulting Engineer;

2. visits with the Consulting Engineer and reviews the progress of his contract; and

3. keeps records of Engineering Contracts concerning due date of contracts, additions to the contracts, money appropriations, money paid out (claim files) and the Consulting Engineers contract file folders.

3.7.6.3 Supervision by the Project Engineer

The technical supervision of the engineering contracts is the responsibility of the Project Engineer(s) and includes the following:

1. provides technical guidance to the Consultant and reviews project plans for adherence to ODOT policy and specifications;

2. coordinates, attends, and documents project meetings;

3. monitors progress on the project and verifies completion percentages on Consultant Invoices as requested by the Engineering Contracts Officer;

4. provides the Engineering Contracts Officer with copies of all pertinent project-related correspondence; and

5. prepares plans for submission.

3.8 ENVIRONMENTAL PROCEDURES

3.8.1 General

The Transportation Planning Branch within the Planning Division has overall responsibility for meeting the State and Federal requirements for environmental procedures. The Branch has prepared the *ODOT Action Plan*. The publication presents the step-by-step procedures used by ODOT for all projects to meet the environmental requirements.

3.8.2 Environmental Classifications

One of the basic operational features of ODOT's environmental procedures is the classification of all projects into one of four classes. This classification will determine the level of detail necessary for the environmental impact analyses. As presented in the *ODOT Action Plan*, the four classes are defined in the following.

3.8.2.1 Class I

Actions of superior, large and considerable importance involving substantial planning, time, resources or expenditures. Any action likely to precipitate significant impacts to land use; planned growth; development patterns; traffic volumes; travel patterns; transportation services, including public transportation; and natural and manmade resources, would be considered a major action and thereby requiring an Environmental Impact Statement (EIS).

3.8.2.2 Class II

Actions which require the acquisition of significant amounts of rights-of-way which may or may not have a significant impact on the quality of the human environment. They could involve substantial planning, time, resources or expenditures to determine if they would induce significant impacts to land use, planned growth, development patterns, traffic volumes, travel patterns or natural or cultural resources. If significant impacts are found, then an EIS is required, and the action needs to be reclassified as a Class I action.

3.8.2.3 Class III

Actions which do not require the acquisition of significant amounts of rights-of-way and may or may not have a significant impact on the quality of the human environment. They could involve substantial planning, time, resources or expenditures to determine if they would induce significant impacts to land use, planned growth, development patterns, traffic volumes, travel patterns or natural or cultural resources. If significant impacts are found, then an EIS is required, and the action needs to be reclassified as a Class I action.

3.8.2.4 Class IV (Categorical Exclusions)

Actions normally classified as categorical exclusions are those which do not involve substantial planning, time, resources or expenditures. These actions will not induce significant impacts to land use, planned growth, development patterns, traffic volumes, travel patterns or natural or cultural resources.

3.8.3 Design Division Role

3.8.3.1 Project Development Stage

To conduct the environment analysis during project development, a Scoping Team is established for the project. Membership on the Scoping Team will be determined on a project-by-project basis as dictated by the specific nature of the proposed project. Each Team member is responsible for evaluating the proposed alternatives for impacts in his area of expertise.

A representative from Design will be included on the Scoping Team. The Design representative will evaluate the alignment, cross section elements, intersections, interchanges, drainage, traffic, etc., for each alternative. Of these elements, the likely right-of-way needs is often the single most important road design factor in the environmental analysis. He will report his analyses to the Project Coordinator from the Transportation Planning Branch for consideration in the overall project evaluation.

3.8.3.2 Project Design Stage

Once the preferred project alternative has been selected, the project is transferred to Design for detailed construction plan development. The Project Engineer is responsible for ensuring that all environmental commitments made during the project development stage are fully implemented in the project design. These may include:

1. implementation of any environmental mitigation plans,

2. developing proper monitoring and enforcement provisions during construction for inclusion in the contract Special Provisions, and

3. properly addressing the noise, water quality, historical, archeological and erosion impacts of the project.

3.9 PUBLIC HEARINGS/MEETINGS

3.9.1 General

The Transportation Planning Branch within the Planning Division has overall responsibility for the advertisement and conduct of public hearings and meetings. The Branch has prepared the *ODOT Action Plan*, which discusses the Department's criteria for public involvement. The *Action Plan* meets the requirements of the FHWA for public hearings.

3.9.2 Types

ODOT conducts two types of public hearings/ meetings:

1. Informal Public Meetings. These are typically held relatively early in project development (e.g., before the study of alternative locations begins) to generally inform the public of project status and likely future developments.

2. Formal Public Hearing. These are held (or an opportunity is advertised) for all Class I and Class II projects. The hearing will include a detailed discussion on the proposed alternatives and the impacts each one will have. As appropriate, the public hearing may include a discussion on relocation assistance programs, right-of-way acquisition and tentative construction schedules.

The *Action Plan* discusses the advertisement for and conduct of the public hearing/meeting in more detail.

3.9.3 Design Division Role

Design typically has a significant role in the presentation of the public hearing/meeting for roadway projects. The typical attendees from Design will be the Division Engineer (or his designee), the Project Engineer and, where applicable, the consultant. Any of these may discuss the design features of the proposed alternatives such as roadway width, right-of-way impacts; access control, drainage requirements, traffic impacts, erosion control measures, etc. The design personnel will assist the Planning Division in the preparation of displays as requested. Planning Division personnel will be responsible for conducting the public hearing and providing handouts and pertinent materials to the audience.

3.10 REFERENCES

1. *ODOT Policy Manual*, Books 1 and 2, compilation of current Oklahoma Department of Transportation Policy Directives.

2. *Policy and Procedures*, Volumes 1 and 2, Office of Design, Oklahoma Department of Highways, 1969.

3. *Scoping Procedure*, ODOT Improved Quality of Service Group on Scoping, adopted by the Oklahoma Transportation Commission, October 1, 1990.

4. *ODOT Action Plan*, Oklahoma Department of Transportation, Planning Division (with assistance from FHWA), revised 1980.

5. *State of Oklahoma County Roads Design Guidelines Manual*, Oklahoma Department of Transportation and the Association of County Commissioners of Oklahoma, June 1991.

6. *Oklahoma Transportation Commission Rules, Regulations, Policies, and Procedures*, adopted December 6, 1976, revised August 28, 1987.

7. *ARTEMIS• Training, Introduction to ARTEMIS, Mainframe V.2.1*, Metier Management Systems.

8. *ARTEMIS• 9000 Manual*, Metier Management Systems.

9. *Network and Scheduling/9000 Training Manual*, Metier Management Systems.

of federal funds and to ensure that ODOT meets the applicable engineering requirements for their proposed highway projects. FHWA maintains a Division Office within each State, and this is the basic point of contact for ODOT.

The designer must coordinate with the FHWA during key points in project development. The FHWA involvement in a project may include:

1. approving the justification for new or modified interstate interchanges,

2. participating in the project scoping process,

3. providing design acceptance,

4. participating in design reviews,

5. approving the plans, specifications and estimate (PS&E),

6. participating in Value Engineering (VE) programs,

7. conducting routine construction inspections, and

8. conducting final project inspections.

The designer also must inform the FHWA of any significant changes in the proposed design. At the discretion of the Division Engineer, all contacts with the FHWA should be at the Project Engineer level or higher.

3.1.3.2 Other Federal Agencies

ODOT may need to coordinate project development activities with other Federal agencies, depending upon the nature of the project. In many cases, this will be for environmental reasons. Some of these agencies include:

1. Fish and Wildlife,
2. Environmental Protection Agency,
3. U.S. Forest Service,
4. Federal Emergency Management Agency,
5. Bureau of Land Management,
6. Army Corps of Engineers,
7. U.S. Coast Guard,
8. Federal Aviation Administration,
9. Bureau of Indian Affairs,
10. Interstate Commerce Commission, and
11. National Park Service.

Typically, when coordination is needed, the designer will not be responsible for direct contacts with the Federal agency. For example, the Bridge Division will be responsible for securing the Army Corps Section 404 permit. However, the designer will be responsible for supplying much of the information needed to work with and coordinate with the Federal agency.

3.1.3.3 Other State Agencies

Depending upon the nature of the project, the designer may need to coordinate project development activities with other State agencies. Some of these agencies may include:

1. Department of Health,
2. Department of Wildlife,
3. Parks Department,
4. State Aeronautics Commission,
5. Farmrail Cooperation,
6. Corporation Commission, and
7. Water Resources Board.

Chapter Four

PLAN DEVELOPMENT

This chapter will be prepared and submitted in the future.

Chapter Five

Basic Design Controls

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(Continued)

Chapter Five

BASIC DESIGN CONTROLS

5.1 HIGHWAY SYSTEMS

5.1.1 Project Prefixes

Table 5.1A presents prefixes for both Federal-aid and State-funded projects. These are used to identify projects and to indicate the source of funds for each project.

5.1.2 Functional Classification System

The functional classification concept is one of the most important determining factors in highway design. In this concept, highways are grouped by the character of service they provide. Functional classification recognizes that the public highway network in Oklahoma serves two basic and often conflicting functions — access to property and travel mobility. Each highway or street will provide varying levels of access and mobility, depending upon its intended service. 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. If this objective is achieved, the benefits to the traveling public will be maximized.

The functional classification system provides the framework for determining the geometric design of individual highways and streets. Once the function of the highway facility is defined, the highway designer can select an appropriate design speed, roadway width, roadside safety elements, amenities and other design values. The entire *ODOT Roadway Design Manual* is based upon this systematic concept to determining highway design.

The ODOT Planning Division has functionally classified all public highways and streets within Oklahoma. The project engineer should contact the Planning Division to determine the applicable highway functional classification.

5.1.2.1 Arterials

Arterial highways are characterized by a capacity to quickly move relatively large volumes of traffic but an often restricted capacity to serve abutting properties. The arterial system typically provides for high travel speeds and the longest trip movements. Rural arterials provide connections between the major urban areas and provide a level of service suitable for statewide or interstate travel. The rural arterial system provides integrated, continuous movements without the need for stub connections.

In urban areas, the arterial system provides these functions: (1) It serves the major centers of activity within the urban area; (2) it carries the highest traffic volumes and longest trip movements; and (3) it serves both major intra-city and through trips. The rural and urban arterial systems are connected to provide continuous through movements at approximately the same level of service.

Table 5.1A

PROJECT PREFIXES

TO BE PREPARED

In Oklahoma, arterials include the following:

1. Principal Arterials. In both rural and urban areas, the principal arterials provide the highest traffic volumes and the greatest trip lengths. Principal arterials can be further subdivided into the following classifications:

   a. Freeways. The freeway, which includes Interstate highways, 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. For these reasons, freeways are considered a special type of highway within the functional classification system, and separate design criteria have been developed for these facilities.

   b. (Other) Principal Arterials. These facilities may be 2 lane or multilanes, with or without a median. Partial control of access is desirable along these facilities, but control of access by regulation is often provided. The access is limited by the governing regulations (e.g., ODOT *Policy on Driveway Regulations for Oklahoma Highways* (5)) or by local jurisdictions.

2. Other Arterials. In rural areas, other arterials will provide a mix of interstate and interregional travel service. In urban areas, other arterials may carry local bus routes and provide intra-community connections, but they will not, for example, penetrate neighborhoods. When compared to the principal arterial system, other arterials may provide lower travel speeds, accommodate shorter trip lengths and lower traffic volumes, but they provide more access to property.

5.1.2.2 Collectors

Collector routes are characterized by a roughly even distribution of their access and mobility functions. Traffic volumes and speeds may be somewhat lower than those of arterials. In rural areas, collectors serve intra-regional needs and provide connections to the arterial system. All cities and towns within a region will be connected. In urban areas, collectors act as intermediate links between the arterial system and points of origin and destination. Urban collectors typically penetrate residential neighborhoods and commercial/industrial areas. Local bus routes will often include collector streets.

5.1.2.3 Local Roads and Streets

All public roads and streets not classified as arterials or collectors are classified as local roads and streets. 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 distances short. Through traffic is often deliberately discouraged.

5.1.3 Urban Design Classification (by Type of Area)

The functional classification system is divided into urban and rural categories. However, in many cases an urban/rural designation is not sufficiently specific to determine the appropriate project design. Therefore, ODOT has adopted urban design classifications of "suburban" and "urban" to reflect the extent of roadside development. These designations are reflected in the tables of geometric design criteria for new construction/reconstruction (Chapter Twelve) and for existing highways (Chapter Thirteen). The designer should consider the following descriptions when selecting the applicable design classification for urban areas:

1. Suburban. These areas are usually located at the fringes of urbanized and small urban areas. The predominant character of the surrounding environment may be residential, but it will also typically include a considerable number of commercial establishments and especially strip commercial development. There may also be a few industrial parks in suburban areas. On suburban roads and streets, drivers usually have a significant degree of freedom but, nonetheless, they must also devote some of their attention to entering and exiting vehicles. Roadside development is characterized by low to moderate density. Pedestrian activity may or may not be a significant design factor. Right-of-way is often available for roadway improvements.

   A typical suburban arterial may have strip commercial development and perhaps a few residential properties. Posted speed limits usually range between 35 and 50 mph, and there may be several signalized intersections along the arterial.

   Local and collector streets in suburban areas may be located in residential areas, but may also serve commercial areas. Posted speed limits typically range between 25 and 50 mph. The majority of intersections will have stop or yield control, but there may be an occasional traffic signal.

2. Urban. These areas normally refer to the densely developed commercial areas. The roadside development is most often commercial on arterials. A substantial number of collector/local roads and streets in urban areas pass through a high-density, residential environment (e.g., apartment complexes, row houses). Access to property is the primary function of the road network in urban areas; the average driver rarely passes through an urban area for mobility purposes. Pedestrian considerations may be important, especially at intersections. Right-of-way for roadway improvements is usually not available.

   Because of the high density of development in urban areas, the distinction between the functional classes (local, collector or arterial) becomes less important when considering geometric design elements such as design speed. The primary distinction among the three functional classes is often the relative traffic volumes and, therefore, the number of lanes. As many as half the intersections may be signalized; posted speed limits typically range between 25 and 45 mph.

5.1.4 Federal-Aid System

The Federal-aid system consists of those routes within Oklahoma which are eligible for the categorical Federal highway funds. ODOT, working with the local governments and in cooperation with FHWA, has designated those routes within the State which are eligible for Federal funds. The criteria are based on the relative importance of the highway route and the anticipated functional classification 5-10 years in the future. 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 implemented a major realignment of the Federal-aid system. Traditionally, the system had previously been divided into Interstate, primary, secondary and urban Federal-aid systems. 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 the 1991 ISTEA.

5.1.4.1 National Highway System

The National Highway System (NHS) is the system of those highways determined to have the greatest national importance to transportation, commerce and defense in the United States. Ultimately, the NHS will represent a total highway mileage not to exceed 155,000 miles nationwide. Each State, in cooperation with local officials, must designate its proposed NHS by December 1993, with ultimate Congressional approval by September 1995. As an interim measure, the NHS consists of all Interstate highways and all other principal arterials.

To properly manage the NHS, the FHWA has 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 monitoring, congestion and safety.

5.1.4.2 Surface Transportation Program

The Surface Transportation Program (STP) provides Federal funds for Federal-aid roads. These are defined as public roads not on the NHS and not functionally classified as a minor rural collector or a local road or street. 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. The basic objective of the STP is to provide Federal funds for improvements to facilities not considered to have significant national importance. The STP requires a minimum of Federal requirements for funding eligibility.

The STP established minimum set asides for specific activities. These are a 10% requirement for safety improvements (e.g., hazard elimination projects) and 10% for "transportation enhancement activities" (e.g., pedestrian and bicycle facilities, landscaping, historic preservation).

5.1.4.3 Off-System

Off-system roads and streets are any public facilities not on any Federal-aid system and not a Federal-aid road. FHWA has established programs which, in certain cases, provide funds for these routes (e.g., the Highway Bridge Replacement and Rehabilitation Program.)

5.1.5 <u>Jurisdictional System</u>

The State of Oklahoma contains approximately 111,180 miles of public roads as of 1988. The network has been divided into several jurisdictional systems based on the organization or agency responsible for highway and street improvements, for maintenance and/or for traffic enforcement.

5.1.5.1 State Highway System

The State highway system, under the jurisdiction of the Oklahoma Department of Transportation, consists of those intercounty and interstate highways, including their extensions through incorporated areas, as designated by the Oklahoma Transportation Commission. The State highway system contains 12,300 miles (1988), which is approximately 11% of all public roads in Oklahoma. In rural areas, the State system includes all principal and minor arterials (primary State highways) and all major collector routes. In urban areas, the State system includes all principal and minor arterials and many collector routes. The State highway system is subdivided into primary and secondary routes.

The Oklahoma Transportation Commission, at its discretion, may designate as a part of the State highway system any roads connecting State Parks, National Parks and Stated-owned institutions with the State highway system, county highway system or city street where the right-of-way for such connection has been obtained and title thereto is in the State of Oklahoma or any agency thereof. See Reference 6.

5.1.5.2 County Road System

The county governments are responsible for all rural roads within their boundaries which are not on the State highway system. There are 86,800 miles (1988) of county-maintained roads in Oklahoma. The Department's Local Government Division is responsible for administering both State and Federal funds which are available for highway improvements on eligible county routes. See Reference 6.

5.1.5.3 Municipal Street System

The municipal street system consists of all local city streets within the corporate limits not on the State highway system. The extension of these routes outside the corporate limits, but still within the urbanized or small urban area, are also the responsibility of the city. The county may, in some cases, have jurisdiction of section line roads within the urban boundary (see the ODOT Planning Division). For some facilities outside the corporate city limits but within the urban boundary, the city and county have established agreements for funding improvements, maintenance and traffic enforcement.

5.1.5.4 Special Purpose Highways

ODOT provides funds for several special purpose highways which are not part of the State, county or municipal systems. These highways include:

1. Lake and Recreation Access Highways. Those highways which are intended to provide direct access to public user facilities located within the immediate vicinity of lakes and other recreation areas operated by a public agency of the State of Oklahoma, one of its political subdivisions or the Federal government.

2. Industrial Access Highways. Those highways which encourage and assist local efforts toward industrial development by providing direct access facilities to specific industrial operations or to officially designated industrial areas wherein industrial operations are under way or have been committed by a specific time schedule. The industrial operations must be guaranteed by the public jurisdiction or by the commitment of either public or private funds to receive State funds. The State and public jurisdiction must also agree on maintenance and traffic enforcement arrangements.

3. Airport Access Highways. Those highways which provide access between airports owned or operated by local governments and the State or local highway system. To receive State funds, the State and local government must reach agreement on funding, maintenance and traffic enforcement arrangements.

Eduardo Elder

OKLAHOMA DEPT. OF TRANSPORTATION

Roadway Design (405)521-2695

Date: April 8, 1998

To: **All Roadway Design Personnel**

From: Roadway Design Engineer

Subject: Design Speed and Highway Geometrics

Attached you will find an excerpt from Title 47 of Oklahoma Statutes, the Vehicle Code. As a result of this 1996 change to the code, the legal speed limit on many of Oklahoma highways was increased.

While it is not always possible to obtain higher design speeds and at the same time stay within project scope, the following should serve as preferred guidelines:

Typical		Design Speeds English	Metric
Interstate	Rural Urban	75 mph 65 mph	120 km/hr 110 km/hr
4 lane divided		70 mph	110 km/hr
2 on 4		70 mph	110 km/hr
Super 2 (w/passing lanes)		70 mph	110 km/hr
2 lane w/shoulders		65 mph	110 km/h
4/5 lane undivided w/shldrs.		55/65 mph*	90/110 km/hr*
curb and gutter section		45 mph**	70 km/hr**

* See section 9.4.4 of ODOT Design Manual
** or as posted speed limits warrants

Although site conditions sometimes prohibit, a prudent designer will try to obtain mid-range or high values for horizontal and vertical curves when designing a new alignment rather than obtaining minimum values.

Christine M. Senkowski

Christine M. Senkowski, P. E.
Roadway Design Engineer

CMS:BS:mg

cc: Managers
Squad Supervisors
Asst Dir. - Preconst.

§47-11-801.

A. Any person driving a vehicle on a highway shall drive the same at a careful and prudent speed not greater than nor less than is reasonable and proper, having due regard to the traffic, surface and width of the highway and any other conditions then existing, and no person shall drive any vehicle upon a highway at a speed greater than will permit the driver to bring it to a stop within the assured clear distance ahead.

B. Except when a special hazard exists that requires lower speed for compliance with subsection A of this section, the limits specified in this act or established as hereinafter authorized shall be maximum lawful speeds, and no person shall drive a vehicle on a highway at a speed in excess of such maximum limits:

1. Seventy-five (75) miles per hour in locations comprising:
   a. the turnpike system, and
   b. rural segments of the interstate highway system, as may be designated by the Transportation Commission. Provided, however, the Commission shall determine prior to the designation of such segments that the public safety will not be jeopardized;
2. Seventy (70) miles per hour in locations which are:
   a. four-lane divided highways including, but not limited to, the interstate highway system, and
   b. super two-lane highways. As used in this section, a super two-lane highway shall mean any two-lane highway with designated passing lanes, and consisting of paved shoulders not less than eight (8) feet in width.
3. Sixty-five (65) miles per hour in other locations;
4. No person shall drive a school bus at a speed greater than a maximum of fifty (50) miles per hour except on turnpikes and interstate highways where the maximum shall be sixty-five (65) miles per hour;
5. On any highway outside of a municipality, the speed limit in a properly marked school zone shall be a maximum of twenty-five (25) miles per hour, provided the zone is marked with appropriate warning signs placed in accordance with the latest edition of the Manual on Uniform Traffic Control Devices. The Department of Transportation shall mark such school zones, or entrances and exits onto highways by buses or students, so that the maximum speed provided by this section shall be established therein. Exits and entrances to controlled-access highways which are within such school zones shall be marked in the same manner as other highways. The county commissioners shall mark such school zones along the county roads so that the maximum speed provided by this section shall be established therein. Said signs may be either permanent or temporary. The Department of Transportation shall give priority over all other signing projects to the foregoing duty to mark school zones. The

5.2 SPEED

5.2.1 Definitions

1. Design Speed. Design speed is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern.

2. 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 average running speed is the distance summation for all vehicles divided by the running time summation for all vehicles.

3. Average Travel Speed. Average travel speed is the distance summation for all vehicles divided by the total time summation for all vehicles. (Note: Average running speed only includes the time the vehicle is in motion. Therefore, on uninterrupted flow facilities which are not congested, average running speed and average travel speed are equal.)

4. Operating Speed. The term "operating speed" is commonly used to characterize prevailing vehicular speeds on a highway segment, either through field measurements of speed or through informal field observations. Although no precise percentile is used to define operating speed, it may be assumed to be between the 80th and 90th percentile of actual travel speeds.

5. 85th-Percentile Speed. The 85th-percentile speed is the speed below which 85 percent of vehicles travel on a given highway. The most common application of the value is its use as one of the factors, and usually the most important factor, for determining the posted, legal speed limit of a highway section. Field measurements for the 85th-percentile speed will be conducted during off-peak hours when drivers are free to select their desired speed.

6. Posted Speed Limit. The initial posted speed limit on State highways is based on an engineering and traffic investigation. The Traffic Engineering Division conducts the study on the State highway system. The selection of a posted speed limit is based on several factors:

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

   b. the design speed used during project design;

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

   d. functional classification and type of area;

   e. type and density of roadside development, cultural and roadside friction;

   f. the accident experience during the previous 12 months;

   g. parking practices and pedestrian activity; and

   h. safe speed for curves or hazardous locations within the zone (11).

5.2.2 Design Speed Selection

A design speed is selected for each project which will establish criteria for several design elements including horizontal and vertical curvature, superelevation and sight distance. Chapter Twelve presents the design speed criteria for new construction and reconstruction projects. Chapter Thirteen provides the design speed criteria for projects on existing highways.

In addition to the above, the selected design speed should equal or exceed the anticipated posted/regulatory speed limit of the facility after construction. This applies to all projects. The posted speed limit will be determined based on actual operating speeds of the completed facility and on several factors not directly related to the project design speed. Therefore, to avoid a potential conflict, the project engineer should, early in project development, coordinate the design speed selection with the Traffic Engineering Division to assist in predicting the posted speed limit of the completed facility. If the proposed design speed will be less than the predicted posted speed limit, the project engineer must choose one of the following approaches:

1. increase the project design speed to equal the predicted posted speed limit;

2. post the project with a legal speed limit equal to the design speed; or

3. on Federal-aid projects, seek a design exception from FHWA.

Of these options, posting the project at the design speed may be the least desirable. This approach will likely ensure that actual operating speeds will exceed the posted speed limit.

In selecting a design speed, the project engineer should avoid artificially selecting a design speed low enough to eliminate any design exceptions. For example, if the ODOT criteria yield a design speed of 60 mph and one or more geometric features are adequate only for 45 mph, the project design speed should be 60 mph and <u>not</u> 45 mph. The project engineer will then be required to seek design exceptions for the 45-mph geometric features.

5.3 TRAFFIC VOLUME ANALYSES

5.3.1 Definitions

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

2. Level of Service (LOS). A qualitative concept which has been developed to characterize acceptable degrees of congestion as perceived by motorists. In the *Highway Capacity Manual*, the qualitative descriptions of each level of service (A to F) have been converted into quantitative measures for the capacity analysis for each highway element, including:

   a. freeway mainline;
   b. freeway mainline/ramp junctions;
   c. freeway weaving areas;
   d. interchange ramps;
   e. two-lane, two-way rural highways;
   f. multilane rural highways;
   g. signalized intersections;
   h. unsignalized intersections; and
   i. urban and suburban arterials.

   Chapters Twelve and Thirteen present guidelines for selecting the level of service for highway design.

3. Average Annual Daily Traffic (AADT). The total yearly volume in both directions of travel divided by the number of days in a 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. The use of an ADT may produce a bias because seasonal peaks could be included or excluded during the count period.

5. Rate of Flow. The 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 (typically, 15 minutes).

6. Peak-Hour Factor (PHF). A ratio of the volume occurring during the peak hour to the peak rate of flow during a given time period within the peak hour (typically, 15 minutes). PHF is calculated as follows:

$$PHF = \frac{Hourly\ Volume}{Peak\ Rate\ of\ Flow\ (within\ the\ hour)}$$

   If 15-minute periods are used, then:

$$PHF = \frac{V}{4\ x\ V_{15}}$$

   Where: V = hourly volume, vph

   V_{15} = volume during the peak 15-minute period of the peak hour, vph

   The *Highway Capacity Manual* discusses the peak-hour factor in more detail.

7. Design Hourly Volume (DHV). The 1-hour volume in both directions of travel in the design year selected for determining the highway design. Note that, for capacity analyses, the DHV is typically converted to an hourly flow rate based on the maximum 15-minute flow rate during the DHV.

8. Directional Distribution (D). The division, by percent, of the traffic in each direction of travel during the DHV, ADT and/or AADT.

9. Directional Design Hourly Volume (DDHV). The 1-hour volume in one direction of travel during the DHV. The DDHV is calculated as follows:

$$DDHV = AADT \times K \times D$$

where each term is defined in this section of the *ODOT Roadway Design Manual.*

10. (Design) Service Flow Rate. The maximum hourly vehicular volume which can pass through a highway element at the selected level of service.

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

12. Delay. The primary 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. Truck Factor (T). A factor which reflects the percentage of heavy vehicles (trucks, buses and recreational vehicles) in the traffic stream during the DHV, ADT and/or AADT.

14. K. The proportion of AADT occurring in the design hour. K will vary based on the hour selected for design and the characteristics of the specific highway facility.

15. AM/PM Peak - Off-Peak Volumes. The 1-hour volumes for each movement at an intersection or interchange. AM/PM peak, off-peak volumes should be used for intersection and interchange analyses in suburban and urban areas, or where traffic volumes are high.

5.3.2 Design Year Selection

5.3.2.1 Roadway Design

A highway should be designed to accommodate the traffic volumes expected to occur within the life of the facility under reasonable maintenance. This involves projecting the traffic conditions for a selected future year. Recommended design years are presented in Table 5.3A. The design year is measured from the expected construction completion date. Future traffic volumes on State highways are provided by the Planning Division.

5.3.2.2 Other Highway Elements

The following presents other design year recommendations that should be considered:

1. Bridges/Underpasses. The structural life of a bridge or underpass may be 50 years or more. For new bridges (including bridge replacements), the initial clear roadway width of the bridge or underpass will be based on the 20-year traffic volume projections beyond the construction completion date. In addition, the bridge designer should, on selected projects, evaluate if the bridge and underpass design will reasonably accommodate structural expansion to meet the clear roadway width needs based on a 50-year projection.

   Bridge rehabilitation projects are those for which a significant amount of the existing substructure or superstructure will remain in place. For bridge rehabilitation

Table 5.3A

RECOMMENDED DESIGN YEAR SELECTION
(Traffic Volumes for Road Design)

PROJECT SCOPE OF WORK	TYPICAL	MINIMUM
New Construction/Reconstruction (All Projects)	20 Years	20 Years
3R Freeway Projects	20 Years	10 Years
3R Non-Freeway Projects	10 Years	Current
Spot Improvements	10 Years	Current

NOTE: Some major projects require the development of functional plans. This stage of project development is assumed to occur approximately 10 years before construction completion. Therefore, on these projects, traffic volume projections will be based on 30 years beyond the year the functional plans are prepared.

projects which include non-safety improvements to all or part of the substructure or superstructure (including the bridge deck), the clear roadway width will be based on the 20-year traffic volume projection beyond the construction completion date. See Chapters Twelve and Thirteen.

2. Right-of-Way. The designer should consider potential right-of-way needs for the anticipated long-term corridor growth for a year considerably beyond that used for roadway design. No specific design year is recommended; however, when selecting an initial median width on a divided highway, for example, the designer should evaluate the potential need for future expansion of the facility to add through travel lanes. As another example, flatter side slopes in the initial design provide more future options.

3. Drainage Design. Drainage appurtenances are designed to accommodate a flow rate based on a specific design year (or frequency of exceedance). The selected design year or frequency will be based on the functional class of the facility and the specific drainage appurtenance, such as culverts. The *ODOT Drainage Manual* and Chapter Fifteen of the *ODOT Roadway Design Manual* present ODOT's criteria for selecting a design year for drainage.

4. Pavement Design. The pavement structure is designed to withstand the

vehicular loads it will sustain during the design analysis period without falling below a selected terminal pavement serviceability. The pavement design analysis period will vary according to the urban/rural location and traffic volume of the facility and functional class. Chapter Sixteen presents ODOT's criteria for selecting a design year for pavement design.

5. Environmental Analyses. Some environmental analyses require the selection of a future year for design (e.g., noise analyses). The Planning Division determines the specific criteria for environmental analyses.

5.3.3 Capacity Analyses

5.3.3.1 Objective

The highway mainline, intersection or interchange should be designed to accommodate the selected design hourly volume (adjusted for the peak-hour factor) at the selected level of service. For multilane facilities, the directional design hourly volume is used to conduct the capacity analysis. In addition, the designer should analyze intersections and interchanges using AM/PM peak, off-peak volumes in urban and suburban areas or where traffic volumes are high. This often impacts the geometric design of the highway.

The capacity analysis involves adjusting the various highway factors which affect capacity until the proposed design meets the level of service (LOS) criteria specified in Chapters Twelve and Thirteen, adjusted for the peaking characteristics. The capacity analysis of the proposed facility will include the following factors:

1. the selected design year (Section 5.3.2);
2. the selected level of service (Chapters Twelve and Thirteen);
3. the projected traffic volumes (from the Planning Division);
4. the measure of effectiveness for the specific highway element (Table 5.3B);
5. the proposed geometrics of the highway element;
6. the analytical methods in the *Highway Capacity Manual*; and
7. the analytical methods in *Procedure for Analysis and Design of Weaving Sections, A Users Guide* (7), when ramp and weaving conditions exceed the *Highway Capacity Manual* limitations.

If the proposed highway design does not meet the level of service (LOS) criteria specified in Chapters Twelve and Thirteen, the highway design should be modified so that the LOS criteria is satisfied.

5.3.3.2 Responsibility

The project engineer is responsible for coordinating all capacity analyses required by the project. The Geometric Design Branch of the Urban Design Division is available as a resource to the project engineer to assist in all capacity analyses. The Traffic Engineering Division is also available as a resource to assist in capacity analyses for signalized intersections. The capacity analyses should then become part of the permanent project file.

Table 5.3B

MEASURES OF EFFECTIVENESS FOR CAPACITY ANALYSES

TYPE OF FACILITY	MEASURE OF EFFECTIVENESS
Freeways	
Basic freeway segments	Density (pc/mi/ln)
Weaving areas	Average travel speed (mph)
Ramp junctions	Flow rates (pcph)
Multilane Highways	Density (pc/mi/ln)
Two-Lane Highways	Percent time delay (%) Average travel speed (mph)
Signalized Intersections	Average individual stopped delay (sec/veh) Demand flow rate to capacity ratio
Unsignalized Intersections	Reserve capacity (pcph)
Arterials	Average travel speed (mph)

Source: (2)

Definitions:

1. pc/mi/ln = passenger cars/miles/lane
2. pcph = passenger cars/hour
3. sec/veh = seconds/vehicle

5.4 ACCESS CONTROL

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 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).

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

1. Full Control. Full control of access means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected frontage roads or local roads only and by prohibiting crossings at grade or direct driveway connections. The freeway is the common term used for this type of highway. Full control of access maximizes the capacity, safety and vehicular speeds on the freeway.

2. Partial Control. Partial control of access means that the authority to control access is exercised to give preference to through traffic to a degree that, in addition to access connections with selected frontage or local roads, there may be some crossings at grade and some private driveway connections. The proper selection and spacing of at-grade intersections and service connections will provide a balance between the mobility and access service of the highway.

3. Control by Regulation. All highways warrant some degree of access control. If access points to other public roads and driveways are properly spaced and designed, the adverse effects on highway capacity and safety will be minimized. These access 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. Control by regulation is exercised by ODOT on the State highway system and by local jurisdictions on other facilities to determine where private interests may have access to and from the public road system.

The designer should reference the following to determine the allowable access for the State highway system:

1. *ODOT Policy on Driveway Regulations for Oklahoma Highways* for 3R non-freeway projects) (5);

2. Oklahoma Transportation Commission *Rules, Regulations, Policies and Procedure* (6);

3. *ODOT Right-of-Way Specifications Manual* (8); and

4. *ODOT Roadway Design Manual*:

   a. Section 9.7 "Warrants for Median Openings,"

   b. Section 9.8 "Driveway Design" (for new construction/reconstruction projects),

   c. Section 10.5 "Access Control at Interchanges," and

   d. Section 10.1 "New Access Points for Interstate Highway System."

5.5 NON-HIGHWAY DESIGN CONTROLS

The characteristics of drivers and vehicles significantly influence the selected design criteria. When the driver and vehicle are properly accommodated, the safety and serviceability of the highway system are enhanced. When they are not accommodated, accidents and inefficient operation may result.

5.5.1 Driver

The appropriate considerations for drivers are already built into the applicable geometric design values (stopping sight distance, horizontal curvature, superelevation, roadway widths, etc.). However, a brief discussion of "typical" drivers has value.

Drivers vary widely in their operating skills, experience, intelligence and physical condition. The highway should be as forgiving as practical to minimize the adverse effects of driver errors. The following discusses certain principles and driver traits which should be incorporated into the highway design:

1. Information Processing. Drivers are limited in how quickly they can gather information, make a decision and take action. They must process information related to lane placement, speed, traffic control devices, highway alignment, roadside conflicts and weather. If the amount, complexity or clarity of the information is inappropriate or excessive, driver error leading to an accident can result.

2. Primacy. Certain driving functions are more important than others. In order of importance they are:

    a. Control — activities related to the physical control of the vehicle via the steering wheel, brake or accelerator.

    b. Guidance — activities related to selecting a safe speed and vehicular path on the highway.

    c. Navigation — activities related to planning and executing a trip from point of origin to destination.

    The highway designer must be aware of the relative importance of these activities and ensure that the more important highway information is properly conveyed to the driver. This could result in removing or relocating lower priority information, if it is likely to interfere with the higher priority information.

3. Expectancy. Drivers are conditioned through experience and training to expect and anticipate what lies ahead on the highway. If this driver expectancy is violated, it will increase the time needed by the driver to assess the situation and make the correct decision. These violations should be avoided. Where they are unavoidable, the designer should allow for increased warning time.

4. Speed. Speed must be considered when accommodating the driver. Higher speeds reduce the visual field and restrict peripheral vision.

A User's Guide to Positive Guidance (FHWA) (9) contains more detailed information related to driver characteristics and highway design accommodation for the driver.

5.5.2 Vehicle

The physical and operational characteristics of vehicles using the highway are important controls in roadway design. Design criteria may vary according to the type of vehicle, and the volume of each type of vehicle in the traffic stream.

Vehicular characteristics which impact design include:

1. Offtracking. The design of intersection turning radii, travelway widening for horizontal curves, and pavement widths for interchange ramps are usually controlled by the largest design vehicle likely to use the facility with some frequency.

2. Storage Requirements. Turn bay storage lengths, bus turnouts and parking layouts are determined by the number and types of vehicles to be accommodated.

3. Acceleration and Deceleration. Eye height and braking distances vary for passenger cars and trucks, which can impact sight distance considerations. Acceleration and deceleration rates often govern the dimensioning of such design features as speed-change lanes at intersections or interchange ramps, climbing lanes or passing lanes.

4. Vehicular Stability. Certain vehicles with high centers of gravity may be prone to skidding or overturning, affecting design speed selection and superelevation design elements.

Tables 5.5A and 5.5B present vehicular dimensions and characteristics for the AASHTO design vehicles.

Table 5.5A

DESIGN VEHICLE DIMENSIONS

Design Vehicle Type	Symbol	Dimensions (feet)										
		Overall			Overhang		Wheelbases					
		Height	Width	Length	Front	Rear	WB_1	WB_2	S	T	WB_3	WB_4
Passenger car	P	4.25	7	19	3	5	11					
Single unit truck	SU	13.5	8.5	30	4	6	20					
Single unit bus	BUS	13.5	8.5	40	7	8	25					
Articulated bus	A-BUS	10.5	8.5	60	8.5	9.5	18		4^a	20^a		
Combination trucks												
Intermediate semitrailer	WB-40	13.5	8.5	50	4	6	13	27				
Large semitrailer	WB-50	13.5	8.5	55	3	2	20	30				
"Double Bottom" semi-trailer — full-trailer	WB-60	13.5	8.5	65	2	3	9.7	20	4^b	5.4^b	20.9	
Interstate Semitrailer	WB-62*	13.5	8.5	69	3	3	20	40-42				
Interstate Semitrailer	WB-67**	13.5	8.5	74	3	3	20	45-47				
Triple Semitrailer	WB-96	13.5	8.5	102	2.5	3.3	13.5	20.7	3.3^d	6^d	21.7	21.7
Turnpike Double Semitrailer	WB-114	13.5	8.5	118	2	2	22	40	2^c	6^c	44	
Recreation vehicles												
Motor home	MH		8	30	4	6	20					
Car and camper trailer	P/T		8	49	3	10	11	18	5			
Car and boat trailer	P/B		8	42	3	8	11	15	5			
Motor home and boat trailer	MH/B		8	53	4	8	20	21	6			

* = Design vehicle with 48′ trailer as adopted in 1982 STAA (Surface Transportation Assistance Act).

** = Design vehicle with 53′ trailer as grandfathered in 1982 STAA (Surface Transportation Assistance Act).

Source: (1)

a = Combined dimension 24, split is estimated.
b = Combined dimension 9.4, split is estimated.
c = Combined dimension 8, split is estimated.
d = Combined dimension 9.3, split is estimated.

WB_1, WB_2, WB_3, WB_4 are effective vehicle wheelbases.
S is the distance from the rear effective axle to the hitch point.
T is the distance from the hitch point to the lead effective axle of the following unit.

Table 5.5B

MINIMUM TURNING RADII OF DESIGN VEHICLES

Design Vehicle Type	Passenger Car	Single Unit Truck	Single Unit Bus	Articu-lated Bus	Semi-Trailer Inter-mediate	Semi-Trailer Combi-nation Large	Semi-Trailer Full Trailer Combi-nation	Inter-State Semi-Trailer	Inter-State Semi-Trailer	Triple Semi-Trailer	Turnpike Double Semi-Trailer	Motor Home	Passenger Car with Travel Trailer	Passenger Car with Boat and Trailer	Motor Home and Boat Trailer
Symbol	P	SU	BUS	A-BUS	WB-40	WB-50	WB-60	WB-62*	WB-67**	WB-96	WB-114	MH	P/T	P/B	MH/B
Minimum design turning radius (ft)	24	42	42	38	40	45	45	45	45	50	60	40	24	24	50
Minimium inside radius (ft)	13.8	27.8	24.4	14.0	18.9	19.2	22.2	9.1	00	20.7	17	26.0	2.0	6.5	35

Source: (1)

* Design vehicle with 48′ trailer as adopted in 1982 STAA (Surface Transportation Assistance Act).

** Design vehicle with 53′ trailer as grandfathered in 1982 STAA (Surface Transportation Assistance Act).

5.6 PROJECT SCOPE OF WORK

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

5.6.1 New Construction

New construction is defined as horizontal and vertical alignment on new location. In addition, any intersection or interchange which falls within the project limits of a new highway mainline or is relocated to a new point of intersection is considered new construction. Chapters Five through Twelve present ODOT's criteria for new construction.

5.6.2 Reconstruction

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 the highway will remain 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 facility cannot accommodate its current or future traffic demands or because the existing alignment is significantly deficient. The extent of pavement improvement will also be a major factor in defining a reconstruction project. In addition, any intersection which falls within the limits of a reconstruction project will be reconstructed as needed.

Because of the significant level of work for reconstruction, the design of the project will be determined by the criteria for new construction. Therefore, Chapters Five through Twelve will apply to reconstruction projects.

5.6.3 3R Projects (Non-Freeways)

3R projects (rehabilitation, restoration, resurfacing) on non-freeways 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 typically work within the existing alignment. Right-of-way acquisition is usually not involved, although small takings (e.g., easements for culvert extensions) are sometimes justified. 3R projects may include any number of the following project-level improvements:

1. pavement resurfacing, rehabilitation and/or reconstruction;
2. lane and shoulder widening;
3. converting an existing median to a two-way, left-turn (TWLT) lane;
4. adding a TWLT lane;
5. adding a truck-climbing lane;
6. converting an uncurbed urban street into a curbed street;
7. geometric and/or roadside safety improvements;
8. drainage improvements; and/or
9. intersection improvements (e.g., adding turn lanes, flattening turning radii, channelization, corner sight distance improvements, etc.).

In addition to the basic project-level improvements, it may be warranted to include other incidental improvements as part of the 3R non-freeway project. These may include any number of the following:

1. flattening an occasional horizontal or vertical curve;
2. adjustments to the roadside clear zone;
3. flattening side slopes;
4. revising the location, spacing or design of existing driveways along the mainline;
5. adding or removing parking lanes;
6. adding sidewalks;
7. relocating utility poles; and/or
8. upgrading guardrail and other safety appurtenances to meet current criteria.

Chapter Thirteen presents the ODOT criteria for the design of 3R non-freeway projects.

5.6.4 3R Projects (Freeways)

3R projects (resurfacing, restoration, and/or rehabilitation) on existing freeways are primarily intended to extend the service life of the existing facility and to enhance highway safety. In addition, these projects should make cost-effective improvements to the existing geometrics, where practical. 3R freeway projects may include any number of the following project-level improvements:

1. pavement resurfacing, rehabilitation and/or reconstruction;
2. geometric and/or roadside safety improvements;
3. drainage improvements;
4. addition of auxiliary lanes (e.g., a truck-climbing lane);
5. realigning or widening an existing ramp or modifying an existing interchange; and/or
6. adding a new interchange.

In addition to the basic project-level improvement, it may be warranted to include other incidental improvements as part of the 3R freeway project. These may include any number of the following:

1. adjusting the roadside clear zone,
2. flattening side slopes,
3. improvements to interchange gore areas,
4. upgrading guardrail and other safety appurtenances to meet new criteria,
5. flattening a horizontal or vertical curve, and/or
6. lengthening existing acceleration or deceleration lanes.

Chapter Thirteen presents the ODOT criteria for the design of 3R freeway projects.

5.6.5 Spot Improvements

Spot improvements are intended to correct an identified deficiency at an isolated location. The deficiency may be related to structural, geometric, safety or drainage 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 examples of spot improvements are:

1. safety improvements funded by the Surface Transportation Program, and

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

Chapter Thirteen presents the ODOT criteria for the design of spot improvement projects.

5.7 SIGHT DISTANCE

5.7.1 Stopping Sight Distance

Stopping sight distance (SSD) is a basic design control which has a critical effect on the safety and serviceability of the highway facility. The following sections present various SSD criteria.

5.7.1.1 Passenger Cars (Level Grade)

Table 5.7A presents SSD criteria for passenger cars on level grades for various assumptions within the SSD model. Except for the last column, these criteria are for information only and should not normally be used directly for design. The last column in Table 5.7A presents the AASHTO SSD criteria (desirable and minimum). These values will normally be used in design.

5.7.1.2 Trucks (Level Grade)

Table 5.7B presents SSD criteria for trucks on level grades. The designer should consider using these criteria at the following sites:

1. facilities with high truck volumes,
2. facilities with a high incidence of truck accidents,
3. railroad/highway grade crossings, and
4. special use facilities (e.g., truck weigh stations).

5.7.1.3 Passenger Cars (Grade Adjusted)

Table 5.7C presents the AASHTO SSD criteria adjusted for downgrades. If the downgrade is 3% or steeper, the designer should consider using these SSD values.

5.7.1.4 Trucks (Grade Adjusted)

Table 5.7D presents the truck SSD values from Table 5.7B adjusted for grades. The designer should consider using these criteria at the sites listed in Section 5.7.1.2 and where the downgrade is 3% or steeper.

5.7.1.5 SSD Application

The application of the SSD to a specific geometric element (e.g., crest vertical curve) is discussed in the applicable section of the *ODOT Roadway Design Manual.*

5.7.2 Decision Sight Distance

Drivers may be 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 volumes and traffic control devices may all compete for the driver's attention. This may increase the required driver perception/reaction time beyond that provided by the AASHTO SSD values (2.5 seconds). Examples of these locations include:

1. freeway exits,
2. freeway lane drops,
3. left-side entrances or exits,
4. at-grade intersections near a horizontal curve,
5. railroad/highway grade crossings,
6. detours,
7. along high-speed, high-volume urban arterials with considerable roadside friction, or

Table 5.7A

STOPPING SIGHT DISTANCE[e]
(Passenger Cars – Level Grade)

Initial Vehicular Speed (V_i)[a] (mph)	Perception/ Reaction Time[b] (seconds)	Distance Traveled (feet)	Braking Action[c]	Braking Distances (feet)	Total Calculated SSD (feet)	AASHTO Rounded for Design[d] (feet)
20	1.0	29	Locked-Wheel	33	62	--
			Comfort	33	62	--
	2.5	73	Locked-Wheel	33	106	Des: 125 Min: 125
			Comfort	33	106	--
25	1.0	37	Locked-Wheel	55	92	--
			Comfort	55	92	--
	2.5	92	Locked-Wheel	55	147	Des: 150 Min: 150
			Comfort	55	147	--
30	1.0	44	Locked-Wheel	77	121	--
			Comfort	86	130	--
	2.5	110	Locked-Wheel	77	187	Des: 200 Min: 200
			Comfort	86	196	--
35	1.0	51	Locked-Wheel	120	171	--
			Comfort	147	198	--
	2.5	128	Locked-Wheel	120	248	Des: 250 Min: 225
			Comfort	147	275	--
40	1.0	59	Locked-Wheel	167	226	--
			Comfort	216	275	--
	2.5	147	Locked-Wheel	167	314	Des: 325 Min: 275
			Comfort	216	363	--
45	1.0	66	Locked-Wheel	218	284	--
			Comfort	298	364	--
	2.5	165	Locked-Wheel	218	383	Des: 400 Min: 325
			Comfort	298	463	--
50	1.0	73	Locked-Wheel	278	351	--
			Comfort	380	453	--
	2.5	183	Locked-Wheel	278	461	Des: 475 Min: 400
			Comfort	380	563	-

Table 5.7A

STOPPING SIGHT DISTANCE[e]
(Passenger Cars – Level Grade)
(Continued)

Initial Vehicular Speed (V_i)[a] (mph)	Perception/ Reaction Time[b] (seconds)	Distance Traveled (feet)	Braking Action[c]	Braking Distances (feet)	Total Calculated SSD (feet)	AASHTO Rounded for Design[d] (feet)
55	1.0	81	Locked-Wheel	336	417	--
			Comfort	500	581	--
	2.5	202	Locked-Wheel	336	538	Des.: 550 Min: 450
			Comfort	500	702	--
60	1.0	88	Locked-Wheel	414	502	--
			Comfort	619	707	--
	2.5	220	Locked-Wheel	414	634	Des: 650 Min: 525
			Comfort	619	839	--
65	1.0	95	Locked-Wheel	486	581	--
			Comfort	781	876	--
	2.5	238	Locked-Wheel	486	724	Des: 725 Min: 550
			Comfort	781	1019	--
70	1.0	103	Locked-Wheel	583	686	--
			Comfort	943	1046	--
	2.5	257	Locked-Wheel	583	840	Des: 850 Min: 625
			Comfort	943	1200	--

a. V_i is the speed of the vehicle when the object in the road is first perceptible to the driver.

b. Perception/reaction time is the time needed by the driver, from the moment the object is perceptible, to comprehend the nature of the object and to apply the brakes. The 1.0-second time is considered adequate for most drivers in a panic situation; i.e., if the hazard is serious and obvious, most drivers react quickly. The 2.5-second time is the perception/reaction time adopted by the AASHTO *A Policy on Geometric Design of Highways and Streets*. It is considered adequate for 90% of drivers in simple to moderately complex environments.

c. AASHTO assumes a locked-wheel, emergency braking maneuver on a wet pavement in *A Policy on Geometric Design of Highways and Streets*. "Comfort" braking action assumes that the brakes never lock and assumes that the average driver is unable to fully use the vehicle's braking power. The numerical values are from NCHRP 270 *Parameters Affecting Stopping Sight Distance*.

d. These are the SSD values presented for use in design by AASHTO in *A Policy on Geometric Design of Highways and Streets*, where V_i = design speed on a project. Desirable SSD values are based on the design speed; minimum SSD values are based on an assumed initial speed equal to the low-volume average running speed.

e. Use 3.5-ft height of eye and 0.5-ft height of object. See Chapters Six and Seven.

Table 5.7B

STOPPING SIGHT DISTANCE[d]
(Trucks -- Level Grade)

Initial Vehicular Speed (V_i)[a] (mph)	Perception/ Reaction Time[b] (seconds)	Distance Traveled (feet)	Braking Action[c]	Braking Distances[c] (feet)	Total Calculated SSD (feet)
20	1.0	29	Comfort	77	106
	2.5	73	Comfort	77	150
25	1.0	37	Comfort	132	169
	2.5	92	Comfort	132	224
30	1.0	44	Comfort	186	230
	2.5	110	Comfort	186	296
35	1.0	51	Comfort	265	316
	2.5	128	Comfort	265	393
40	1.0	59	Comfort	344	403
	2.5	147	Comfort	344	491
45	1.0	66	Comfort	441	507
	2.5	165	Comfort	441	606
50	1.0	73	Comfort	538	611
	2.5	183	Comfort	538	721
55	1.0	81	Comfort	641	722
	2.5	202	Comfort	641	843

Table 5.7B

STOPPING SIGHT DISTANCE[d]
(Trucks -- Level Grade)
(Continued)

Initial Vehicular Speed (V_i)[a] (mph)	Perception/ Reaction Time[b] (seconds)	Distance Traveled (feet)	Braking Action[c]	Braking Distances[c] (feet)	Total Calculated SSD (feet)
60	1.0	88	Comfort	744	832
	2.5	220	Comfort	744	964
65	1.0	95	Comfort	879	974
	2.5	238	Comfort	879	1117
70	1.0	103	Comfort	1013	1116
	2.5	257	Comfort	1013	1270

a. V_i is the speed of the vehicle when the object in the road is first perceptible to the driver.

b. Perception/reaction time is the time needed by the driver, from the moment the object is perceptible, to comprehend the nature of the object and to apply the brakes. The 1.0-second time is considered adequate for most drivers in a panic situation; i.e., if the hazard is serious and obvious, most drivers react quickly. The 2.5-second time is the perception/reaction time adopted by the AASHTO *A Policy on Geometric Design of Highways and Streets*. It is considered adequate for 90% of drivers in simple to moderately complex environments.

c. For trucks, only a "comfort" braking action is assumed; a "locked-wheel" stop is considered inappropriate for trucks. The numerical values are from TRR 1208 in a paper entitled "Stopping Sight Distance Design for Large Trucks" (Table 5). The values are based on a driver control efficiency of 0.62, considered a worst-performing driver.

d. Use 8.0-ft height of eye and 0.5-ft height of object. See Chapters Six and Seven.

Table 5.7C

STOPPING SIGHT DISTANCE[4]
(Passenger Cars – Grade Adjusted)

Design Speed (mph)	3% Downgrade		6% Downgrade		9% Downgrade	
	Desirable (feet)	Minimum (feet)	Desirable (feet)	Minimum (feet)	Desirable (feet)	Minimum (feet)
20	125	125	130	130	135	135
25	155	155	160	160	170	170
30	210	210	220	220	230	230
35	265	240	280	255	300	275
40	345	295	365	315	395	345
45	425	350	455	380	490	415
50	505	430	545	470	595	520
55	590	490	640	540	---	---
60	700	575	760	635	---	---
65	785	610	855	680	---	---
70	920	695	1010	785	---	---

Source: (1) Revised

Notes:

1. The grade-adjusted SSD's are calculated from the AASHTO formula for vehicular breaking distances on grades ($d = V^2/30 (f \pm G)$). The perception/reaction time is 2.5 seconds. See *A Policy on Geometric Design of Highways and Streets*.

2. The grade-adjusted values are calculated by assuming V equals the design speed for the SSD grade increase for both the desirable and minimum SSD values.

3. For downgrades intermediate between 3%, 6% and 9%, use a straight-line interpolation to calculate SSD.

4. Use 3.5-ft height of eye and 0.5-ft height of object. See Chapters Six and Seven.

Table 5.7D

STOPPING SIGHT DISTANCE[3]
(Trucks -- Grade Adjusted)

Design Speed (mph)	3% Downgrade (feet)	6% Downgrade (feet)	9% Downgrade (feet)
20	166	191	234
25	255	305	398
30	339	407	533
35	457	562	766
40	574	708	968
45	714	891	1236
50	850	1060	1465
55	996	1242	1707
60	1136	1408	1910
65	1321	1646	2250
70	1504	1874	2557

Notes:

1. The grade-adjusted SSD's for trucks are calculated by first determining the average coefficient of friction (f) from the truck braking values on level grade in Table 5.7B ($f = V^2/30d$). The value of f is then used to calculate the truck braking distance on grade using the AASHTO formula ($d = V^2/30\ (f \pm G)$). This braking value is then added to the distance traveled in 2.5 seconds of perception/reaction time to produce the grade-adjusted truck SSD's.

2. For downgrades intermediate between 3%, 6% and 9%, use a straight-line interpolation to calculate SSD.

3. Use 8.0-ft height of eye and 0.5-ft height of object. See Chapters Six and Seven.

8. traffic signals on high-speed rural highways.

Table 5.7E presents the decision sight distance criteria. The application of the criteria will depend upon the rural/urban location and on the type of avoidance maneuver.

5.7.3 Passing Sight Distance

Passing sight distance considerations are limited to two-lane, two-way highways. On these facilities, vehicles may overtake slower-moving vehicles, and the passing maneuver must be accomplished on a lane used by opposing traffic.

Passing sight distance values provided in Table 5.7F are based on the distance needed to safely complete a normal passing maneuver. Table 5.7F also presents the MUTCD (11) values used by the Traffic Engineering Division for marking no-passing zones. The MUTCD values are based on a different set of assumptions than the passing sight distance criteria. Also note that the MUTCD pavement marking criteria are used for capacity adjustments on two-lane, two-way highways (percent no-passing zones).

The designer should note that, on existing highways, it will rarely be warranted to improve the existing passing sight distance on the highway. On new construction/ reconstruction projects, the designer should provide passing sight distance over as high a proportion of the highway as practical. However, it will not likely be warranted to make significant improvements to the horizontal and vertical alignment solely to increase the available passing sight distance.

Table 5.7E

DECISION SIGHT DISTANCE[a]

Vehicular 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

Source: (1)

Application:

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

[a] Use 3.5-ft height of eye and 0.5-ft height of object. See Chapters Six and Seven.

Table 5.7F

PASSING SIGHT DISTANCE/MUTCD CRITERIA

AASHTO[b]		MUTCD[c]	
Design Speed (mph)	Passing Sight Distance (feet)	85th Percentile Speed (mph)	Passing Sight Distance (feet)
30	1100	30	500
40	1500	40	600
50	1800	50	800
60	2100	60	1000
70	2500	70	1200

Sources: (1) and (11)

[b] Use 3.5-ft height of eye and 4.25-ft height of object (AASHTO).
[c] Use 3.5-ft height of eye and 3.5-ft height of object (MUTCD).

5.8 ADHERENCE TO DESIGN CRITERIA

The *ODOT Roadway Design Manual* presents literally thousands of pieces of information on geometric design for application on individual projects. In general, the project engineer is responsible for making every reasonable effort to meet these criteria in the project design. However, it will not always be practical to meet the ODOT criteria. Therefore, this Section presents ODOT's procedures for the necessary action when ODOT's design criteria are not met.

5.8.1 Design Criteria Checklist

The end of this Section presents ODOT's "Design Criteria Checklist." This form must be completed by the project engineer for every project under his responsibility. It will become part of the permanent project file. The Checklist establishes the following hierarchy of importance for ODOT's design criteria.

5.8.1.1 Level One (Design Exceptions)

this is more involved than Level II

Level One includes the controlling design criteria established by the FHWA and the handicapped accessibility criteria. These criteria are judged to be those design elements which are the most critical indicators of a highway's safety and its overall serviceability. On Federal-aid projects, it will be necessary to obtain a written design exception from FHWA when the proposed project design will not meet any one of these criteria. See Section 5.8.2 and the Design Criteria Checklist.

5.8.1.2 Level Two (Design Documentation)

this is for ODOT inhouse

Level Two includes design criteria which are judged to be important indicators of a highway's safety and serviceability but not as critical as the Level One criteria. When Level Two criteria are not met, no formal approval from FHWA is required; however, the project engineer should discuss the Level Two criteria at functional and/or construction plan-in-hand stage or earlier. See the Design Criteria Checklist.

5.8.1.3 Level Three (Design Decision)

Level Three includes all design criteria not on the Design Criteria Checklist. No action is required when Level Three criteria are not met; however, the project engineer may determine that his supervisor should be informally notified of the situation.

5.8.2 Federal Aid Projects

On Federal aid projects, FHWA must approve any exceptions to the Level One design criteria listed on the Design Criteria Checklist. See FHPM 6-2-1-1 for more information on the FHWA process. The following procedure will apply to the request for design exceptions on Federal aid projects:

1. The project engineer will complete the "Design Exceptions" form for the Level One design criteria. (See next page.)

2. The project engineer will prepare a report on the requested design exception(s). The report will present supporting documentation, which may include discussions on:

   a. the ODOT criteria for the element,

   b. the proposed value for the element,

   c. the accident experience in the vicinity of the element,

DESIGN EXCEPTION

To: Federal Highway Administration

From: ____________________ Division

Subject: Design Exception for Project Number ______________________

J/P #: _______________ System: ___________ Route: _________________

County: _____________ Field Division: ______________________________

Project Location: ____

____

Functional classification: (Check one from each column)

Rural: _____ Arterial: _____
Urban: _____ Collector: _____
Local Road: _____
Freeway: _____

Major exceptions to ODOT design:

_____ Design Speed
_____ Shoulder Width
_____ Structural Capacity
_____ Vertical Alignment
_____ Stopping Sight Distance
_____ Superelevation
_____ Lane Width
_____ Bridge Width
_____ Horizontal Alignment
_____ Grades
_____ Cross Slopes
_____ Vertical Clearance

Cost to build to full standard: ____________________

Cost to build with design exception: ____________________

Mitigating measures: ____

____

____ Attached:_____ In Report:_____

Return to ODOT by:______________________________

ODOT Concurrence: ________________________ ________________________
Engineer Manager Design Division Engineer

______________________________ _________________
Asst. Director - Design Date

______________________________ _________________
FHWA District Engineer Date

Comments: ____

____

____

Attachments: Bridge data _______ Accident history _______
Location map _______ Traffic data _______
Special conditions (traffic generators) _______
Other ____

____

d. the environmental impacts of meeting the ODOT criteria,

e. the construction costs of meeting the ODOT criteria,

f. the serviceability impacts of not meeting the ODOT criteria (e.g., level of service), and

g. the mitigating actions taken for each element which does not meet ODOT criteria.

3. The project engineer will submit the package to the ODOT Design Division Engineer, whichever applies, requesting his review.

4. The Design Division Engineer will review the request and return the package to the project engineer with any comments noted.

5. After the package has been revised by the project engineer, the Design Division Engineer and Assistant Director — Design will sign the "Design Exception" form on the indicated lines.

6. The project engineer will prepare the package for submission to the FHWA Division Office with a cover letter requesting concurrence from FHWA. The Design Division Engineer will sign the cover letter to FHWA.

7. The FHWA Division Administrator will concur with the request, if deemed appropriate. NOTE: Any requested exceptions to the accessibility criteria for handicapped individuals must be approved by the FHWA Washington Office.

Oklahoma Department of Transportation

DESIGN CRITERIA CHECKLIST

Application

The project engineer must complete this form in its entirety for every project under his jurisdiction. This Checklist will document the adherence of the proposed project design to the design criteria adopted by ODOT and documented in the *ODOT Roadway Design Manual*. The form will then become part of the permanent project file.

Project Identification

State Job/Piece No.: ______________________________
Project No.: ______________________________
Route No.: ______________________________
Functional Classification: ______________________________
Project Location: ______________________________
____
____
County/City: ______________________________
Project Length: ______________________________

Project Scope of Work

Check the appropriate box. See Section 5.6 of the *ODOT Roadway Design Manual* for definitions.

- ☐ New construction
- ☐ Reconstruction
- ☐ 3R (non-freeway)
- ☐ 3R (freeway)
- ☐ Spot improvement

Provide a brief project description:

____
____
____
____
____

Level One Design Criteria (Design Exceptions) FHWA clearance - more involved

Check the appropriate boxes on Page 3 of the "Design Criteria Checklist." Note that the determination of whether or not the proposed project design meets ODOT's design criteria is dependent upon the project scope of work. If, for example, a 3R non-freeway project is under design, Chapter Thirteen of the *ODOT Roadway Design Manual* will apply. For Federal aid projects, see Section 5.8.2 of the *Manual*. For any proposed design element which does not meet ODOT's criteria on non Federal-aid projects, the project engineer should prepare an attachment to this form which:

1. identifies the design element,
2. identifies ODOT's criteria,
3. identifies the proposed design criteria, and
4. provides justification for the variation.

Level Two Design Criteria - Inhouse ODOT

Check the appropriate boxes on Page 4 of the "Design Criteria Checklist." Note that the determination of whether or not the proposed project design meets ODOT's design criteria is dependent upon the project scope of work. If, for example, a 3R non-freeway project is under design, Chapter Thirteen of the *ODOT Roadway Design Manual* will apply. For any proposed design element which does not meet ODOT's criteria, the project engineer should prepare an attachment to this form which:

1. identifies the design element,
2. identifies ODOT's criteria,
3. identifies the proposed design criteria, and
4. provides a brief justification.

Level One Design Criteria

Design Criteria for Mainline (Provide numerical value for project, where indicated.)	Does the proposed design meet ODOT's criteria?		
	Yes	No*	N/A
1. Design Speed _______ mph			
2. Lane Width _______ feet			
3. Shoulder Width _______ feet			
4. Bridge Widths			
5. Through Travel Lane Cross Slope: __________%			
6. Structural Capacity			
7. Horizontal Curvature (Maximum Degree of Curve)			
8. Stopping Sight Distance at Horizontal Curves (Level SSD)			
9. Superelevation Rate			
10. Stopping Sight Distance at Vertical Curves (Level SSD)			
11. Comfort Criteria at Sag Vertical Curves (3R non-freeway projects only)			
12. Maximum Grades (new construction/ reconstruction projects only)			
13. Vertical Clearances			
14. Accessibility Criteria for Handicapped Individuals			

* Justification for design exception must be prepared and approved. See Section 5.8 of *ODOT Roadway Design Manual.*

Note: Numbers 1, 2, 3, and 5 apply throughout the project. The remaining criteria (e.g., superelevation rates) apply to specific sites within the project limits.

Level Two Design Criteria

Design Criteria		Does the proposed design meet ODOT's criteria?		
		Yes	No*	N/A
1. Basic Design Controls				
a. Level of service (mainline)				
b. SSD application at horizontal curves (downgrade adjusted SSD used)	Horz.			
SSD application for vertical curves (downgrade adjusted SSD used)	Vert.			
c. Truck SSD (level) (at specific sites)				
2. Horizontal Alignment (Mainline)				
a. Travelway widening				
b. Design criteria for compound curves				
c. Superelevation transition length				
d. Superelevation distribution between tangent and curve				
e. Rollover of outside shoulder on super-elevated curves				
f. Superelevation development on reverse curves				
3. Vertical Alignment (Mainline)				
a. Minimum grade				

* See Section 5.8 of *ODOT Roadway Design Manual.*

Design Criteria	Does the proposed design meet ODOT's criteria?		
	Yes	No*	N/A
b. Critical length of grade			
c. Warrants for truck-climbing lanes			
d. Design criteria for truck-climbing lanes (e.g., lane width)			
e. Minimum length of vertical curve			
f. Maximum drainage length of vertical curve (curbed facilities)			
g. Maximum grade (3R projects)			
4. Cross Section Elements (Mainline)			
a. Shoulder cross slopes			
b. Design of parking lanes			
c. Use of barrier curb (V≤45 mph)			
d. Design of sidewalks			
e. Fill slopes			
f. Roadside ditch slopes			
g. Cross section transitions at bridges/underpasses			
h. Design of frontage roads			

* See Section 5.8 of *ODOT Roadway Design Manual*.

Design Criteria	Does the proposed design meet ODOT's criteria?		
	Yes	No*	N/A
i. Median widths			
j. TWLT lane width			
k. Cross section transition details (e.g., 4-lane to 2-lane)			
5. Roadside Safety			
a. Horizontal clearances (clear zones)			
b. Barrier warrants			
c. Barrier length of need			
d. Deceleration criteria for impact attenuators			

* See Section 5.8 of *ODOT Roadway Design Manual*.

Design Criteria		Does the proposed design meet ODOT's criteria?		
		Yes	No*	N/A
6. At-Grade Intersections				
a. Level of service				
b. Skew angle				
c. Profile				
d. Design for right turns				
e. Turning roadway widths				
f. Turn-lane warrants				
g. Turn-lane tapers	Approach Taper			
	Departure Taper			
	Bay Taper			
h. Turn-lane lengths	Deceleration			
	Storage			
i. Intersection sight distance				
j. Median opening length				
k. Minimum island size				
l. Driveway widths				
m. Traffic control				
n. Maximum grade				

* See Section 5.8 of *ODOT Roadway Design Manual.*

Design Criteria		Does the proposed design meet ODOT's criteria?		
		Yes	No*	N/A
7. Interchanges				
a. Exit design	Deceleration			
	Taper			
	Gore Treatment			
b. Entrance design	Acceleration			
	Taper			
c. Ramp design speed				
d. Ramp maximum grade				
e. Ramp width				
f. Ramp horizontal curvature				
g. Superelevation development	Superelevation Rate			
	Transition Length			
	Distribution Between Tangent & Curve			
h. Ramp vertical curvature				
i. Access control				
j. Decision sight distance (in advance of exit gore)				
k. Level of service (weaving and ramps)				
l. Freeway lane drops	Location			
	Taper			

Prepared By ______________________________

Project Engineer (ODOT) or Consultant

Date ______________________________

* See Section 5.8 of *ODOT Roadway Design Manual.*

5.9 REFERENCES

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

2. *Highway Capacity Manual*, Special Report 209, Transportation Research Board, 1985.

3. NCHRP 270 *Parameters Affecting Stopping Sight Distance*, Transportation Research Board, June 1984.

4. "Stopping Sight Distance for Large Trucks," TRR 1208, Transportation Research Board, 1989.

5. *Policy on Driveway Regulations for Oklahoma Highways*, ODOT, Revised January 1987.

6. *Oklahoma Transportation Commission Rules, Regulations, Policies and Procedures*; Oklahoma Transportation Commission, Revised August 1989.

7. *Procedure for Analysis and Design of Weaving Sections — A Users Guide*, Jack Leisch, October 1985.

8. *Right of Way Specifications Manual*, Oklahoma Department of Transportation, November, 1987.

9. FHWA Report No. FHWA-TO-81-1, *A User's Guide to Positive Guidance*, Federal Highway Administration, U.S. Department of Transportation, 1981.

10. Federal Highway Policy Manual, Section 6-2-1-1, Federal Highway Adminsitration, U.S. Department of Transportation.

11. *Manual on Uniform Traffic Control Devices*, Federal Highway Administration, U.S. Department of Transportation, 1988.

Chapter Six

Horizontal Alignment

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Chapter Six

HORIZONTAL ALIGNMENT

Chapter Six presents ODOT criteria for the design of the elements of horizontal alignment. This includes horizontal curvature, superelevation, travelway widening and sight distance around horizontal curves.

6.1 HORIZONTAL CURVES

6.1.1 Types of Horizontal Curves

6.1.1.1 General

This Section discusses the several types of horizontal curves which may be used to achieve the necessary highway deflection. For each type, the discussion briefly describes the curve and presents the ODOT application or usage of the curve type. Section 6.6 presents detailed figures for the basic curve types (simple, compound and spiral), and Section 6.6 presents the necessary details and mathematical equations for the typical applications of horizontal curves to highway alignment.

6.1.1.2 Simple Curves

Simple curves are continuous arcs of constant radius which achieve the necessary highway deflection without an entering or exiting taper. The degree of curvature (D) defines the circular arc that a simple curve will transcribe. ODOT uses the arc definition to define a simple curve on mainline. This is defined as the central angle subtended by a 100-ft distance measured on the circle's circumference (the "arc"). All angles and distances of simple curves are computed in a horizontal plane.

Because of their simplicity and ease of design, survey and construction, ODOT typically uses the simple curve on highway mainline.

6.1.1.3 Compound Curves

Compound curves are a series of two or more simple curves. ODOT uses compound curves on highway mainline only to meet field conditions where a simple curvature is not applicable (e.g., to avoid obstructions which cannot be relocated). When used on mainline, the degree of curve for any one circular arc should be greater than or equal to 2/3 the degree of curve of the succeeding arc ($1.5\ D_1 \geq D_2$).

Chapter Nine discusses the use of compound curves at intersections at-grade (e.g., for curb radii, for turning roadways). Chapter Ten discusses the use of compound curves on interchange ramps and at intersections between ramps and crossing roads.

6.1.1.4 Spiral Curves

Spiral curves provide an entering transition into a simple curve with a variable rate of curvature along its layout. In the most common application on highway mainline, the rate of curvature begins at D=0° (tangent) and gradually increases up to D for the curvature of the simple curve.

ODOT uses spiral curves on highway mainline only to meet field conditions where a simple curve is not applicable (e.g., to avoid obstructions which cannot be relocated). Existing spiral curves are typically retained where improvements are made to existing highways.

6.1.1.5 Reverse Curves

Reverse curves are two simple curves with deflections in opposite directions which are joined by a relatively short tangent distance. Superelevation development for reverse curves requires special attention. This is discussed in Section 6.2.

6.1.2 General Theory

Horizontal curves are, in effect, transitions between two tangents. These deflectional changes are necessary in virtually all highway alignments to avoid impacts on a variety of field conditions (e.g., right-of-way, natural features, man-made features).

6.1.2.1 Ball-Bank Indicator

When a vehicle moves in a circular path, it is forced radially outward by centrifugal force. Figure 6.1A illustrates the dynamics of a vehicle negotiating a horizontal curve, and it presents the geometry of the ball-bank indicator. This is a device which can be mounted on a vehicle in motion. The ball-bank reading indicates the combined effect of the body roll angle (ρ), centrifugal force angle (θ) and superelevation angle (ϕ). The centrifugal force is counterbalanced by the vehicle weight component related to the roadway superelevation or the side friction developed between tires and surface or by a combination of the two.

6.1.2.2 Basic Curve Equations

The point-mass formula is used to define vehicular operation around a curve. Where the curve is expressed using the 100-ft arc definition, the basic equation for a simple curve is:

$$D = \frac{85{,}660\ (e + f)}{V^2}$$

where:

D = degree of curvature
e = superelevation rate
f = side-friction factor
V = vehicular speed, mph

Where the curve is expressed using its radius, the basic equation for a simple curve is:

$$R = \frac{V^2}{15\ (e + f)}$$

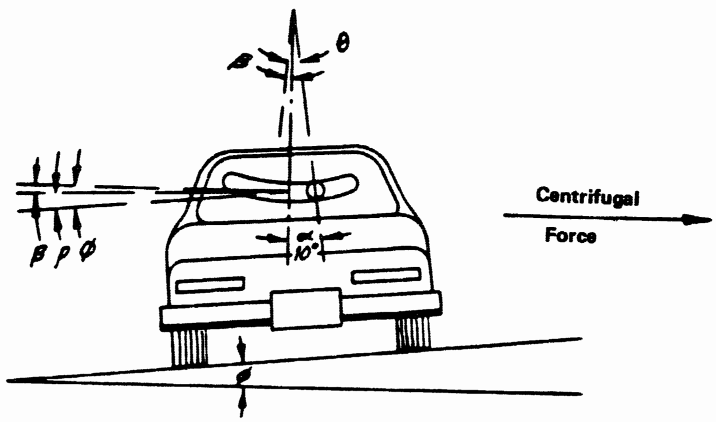
where:

R = radius of curve, ft
e = superelevation rate
f = side-friction factor
V = vehicular speed, mph

Curve radii and degree of curvature are related as follows (see Section 6.6 for the derivation of the equation):

$$R = 5729.577951/D$$

In general, ODOT practice is to use "D" for simple curves which have a radius greater than 500 ft and to use "R" for curve radii less than or equal to 500 ft.



α = Ball-bank indicator angle
ρ = Body roll angle
θ = Centrifugal force angle
ϕ = Superelevation angle
β = ϕ - ρ

Source: (1)

GEOMETRY FOR BALL-BANK INDICATOR

Figure 6.1A

6.1.2.3 Theoretical Approaches

Establishing horizontal curvature criteria requires a determination of the theoretical basis for the various factors in the basic curvature equations. These include the side-friction factor (f) and the distribution method between side friction and superelevation. The theoretical basis will be one of the following:

1. Open-Roadway Conditions. The theoretical basis for horizontal curvature assuming open-roadway conditions includes:

   a. relatively low side-friction factors (i.e., a relatively small level of driver discomfort)(see Section 6.1.2.5); and

   b. the use of AASHTO Method 5 to distribute side friction and superelevation (see Section 6.2).

   Open-roadway conditions apply to all rural facilities and all urban facilities where the design speed (V) >45 mph.

2. Low-Speed Urban Streets. The theoretical basis for horizontal curvature assuming low-speed urban street conditions includes:

   a. relatively high side-friction factors to reflect a high level of driver acceptance of discomfort (see Section 6.1.2.5); and

   b. the use of AASHTO Method 2 to distribute side friction and superelevation (see Section 6.2).

   Low-speed urban streets are defined as streets within an urban or urbanized area where the design speed (V) ≤ 45 mph.

3. Turning Roadway Conditions. The theoretical basis for horizontal curvature assuming turning roadway conditions includes:

   a. higher side-friction factors (than open-roadway conditions) to reflect a higher level of driver acceptance of discomfort (see Section 6.1.2.5); and

   b. a range of acceptable superelevation rates for combinations of curve radius and design speeds to reflect the need for flexibility to meet field conditions for turning roadway and ramp design (see Chapters Nine and Ten).

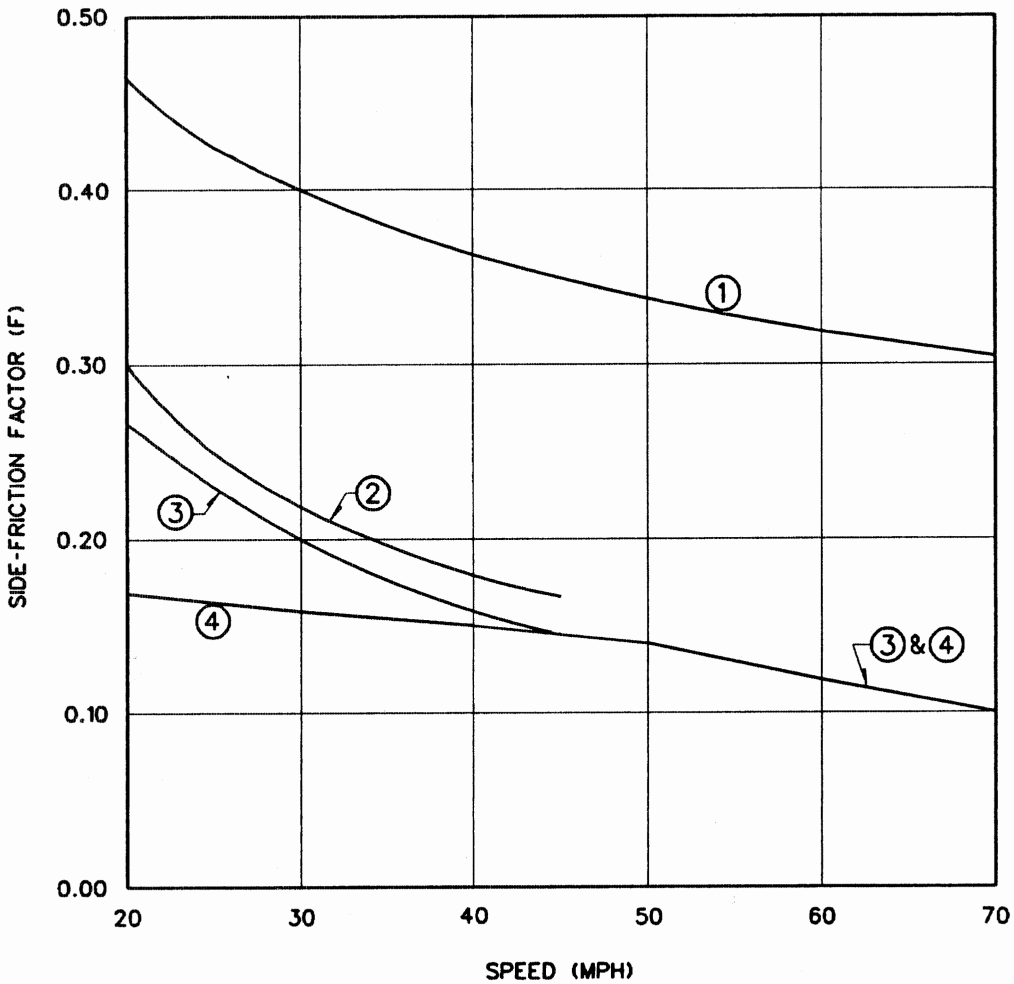
   Turning roadway conditions typically apply to turning roadways at intersections at-grade and to interchange ramps where the curve radius (R) ≤ 500 ft.

6.1.2.4 Superelevation

Superelevation allows a driver to negotiate a curve at a higher speed than would otherwise be comfortable. Superelevation and side friction work together to offset the outward pull of the vehicle as it traverses the horizontal curve. In highway design, it is necessary to establish limiting values of superelevation (e_{max}) based on the operational characteristics of the facility. e_{max} values used by ODOT are discussed in Section 6.2.

6.1.2.5 Side Friction

AASHTO has established limiting side-friction factors (f) for various design speeds and various highway operating conditions (see Figure 6.1B). It is important to note that the f values used in design represent a threshold of driver discomfort not the point of impending skid. As indicated in Figure 6.1B, different sets of f values have been established for different operating conditions



1. ESTIMATED POINT OF IMPENDING SKID ASSUMING SMOOTH TIRES AND WET PCC PAVEMENT.

2. SIDE-FRICTION FACTORS FOR LOW-SPEED URBAN STREETS (V $\leq$ 45 MPH).

3. SIDE-FRICTION FACTORS FOR TURNING ROADWAYS AT INTERSECTIONS AT-GRADE. THEY MAY ALSO BE USED FOR INTERCHANGE RAMPS WHERE THE CURVE RADIUS $\leq$ 500'.

4. SIDE-FRICTION FACTORS FOR OPEN-ROADWAY CONDITIONS.

Source: (1) & (2)

SIDE-FRICTION FACTORS (F)

Figure 6.1B

(see Section 6.1.2.3). The basis for the distinction is that drivers, through conditioning, will accept different levels of discomfort on different facilities.

6.1.3 Maximum Degree of Curvature

The following tables present the maximum degree of curvature (D_{max}) for open-roadway facilities and low-speed urban streets; criteria for turning roadways are presented in Chapter Nine. To define D_{max}, a maximum superelevation rate (e_{max}) must be selected. These are indicated in the following sections. See Section 6.2 for more discussion on the selection of e_{max}.

The tables for D_{max} are as follows:

1. Table 6.1A is applicable for facilities where e_{max} = 0.08 and open-roadway conditions apply.

2. Table 6.1B is applicable to facilities where e_{max} = 0.06 and open-roadway conditions apply.

3. Table 6.1C is applicable to low-speed urban streets where e_{max} = 0.06.

4. Table 6.1D is applicable to low-speed urban streets where e_{max} = 0.04.

6.1.4 Maximum Deflection Without Curve

It may be appropriate to design a facility without a horizontal curve where small deflection angles are present. As a guide, the designer may retain deflection angles of about 1° or less for mainline through traffic conditions. The absence of a horizontal curve will not likely affect driver response or aesthetics. See Chapter Nine for deflections at intersections at-grade.

6.1.5 Minimum Length of Curve

The following will apply when the calculated length of curve (100 Δ/D) is less than the following recommended minimums:

1. $\Delta \leq 5°$. The minimum length of curve should be as follows for open roadways:

   a. $1° \leq \Delta < 3°$: 300 ft
   b. $3° \leq \Delta < 5°$: 500 ft

   The minimum length of curves on low-speed urban streets will be determined on a case-by-case basis.

2. All Major Highways. The minimum length of curve should be 15V, where V equals the design speed in mph.

3. Freeways. For aesthetics, it is desirable that the minimum length of curve be 30V.

Table 6.1A

MAXIMUM DEGREE OF CURVATURE (D_{MAX})
(e_{max} = 0.08, Open-Road Conditions)

Design Speed, V (mph)	e_{max}	f_{max}	Maximum Degree of Curvature, D_{max}*
20	0.08	.17	53° 30′
25	0.08	.165	33° 30′
30	0.08	.16	22° 45′
35	0.08	.155	16° 30′
40	0.08	.15	12° 15′
45	0.08	.145	9° 30′
50	0.08	.14	7° 30′
55	0.08	.13	6° 00′
60	0.08	.12	4° 45′
65	0.08	.11	3° 45′
70	0.08	.10	3° 00′

Table 6.1B

MAXIMUM DEGREE OF CURVATURE (D_{MAX})
(e_{max} = 0.06, Open-Road Conditions)

Design Speed, V (mph)	e_{max}	f_{max}	Maximum Degree of Curvature, D_{max}*
20	0.06	.17	49° 15′
25	0.06	.165	30° 45′
30	0.06	.16	21° 00′
35	0.06	.155	15° 00′
40	0.06	.15	11° 15′
45	0.06	.145	8° 45′
50	0.06	.14	6° 45′
55	0.06	.13	5° 15′
60	0.06	.12	4° 15′
65	0.06	.11	3° 30′
70	0.06	.10	2° 45′

* $D = \frac{85{,}660\,(e + f)}{V^2}$

Table 6.1C

MAXIMUM DEGREE OF CURVATURE (D_{MAX})
(e_{max} = 0.06, Low-Speed Urban Streets (V≤45))

Design Speed, V (mph)	e_{max}	f_{max}	Minimum Radius (ft)*	Maximum Degree of Curvature, D_{max}**
20	0.06	.300	75	77° 00′
25	0.06	.252	135	50° 45′
30	0.06	.221	215	26° 45′
35	0.06	.197	320	17° 45′
40	0.06	.178	450	12° 45′
45	0.06	.163	610	9° 15′

Table 6.1D

MAXIMUM DEGREE OF CURVATURE (D_{MAX})
(e_{max} = 0.04, Low-Speed Urban Streets (V ≤ 45))

Design Speed, V (mph)	e_{max}	f_{max}	Minimum Radius (ft)*	Maximum Degree of Curvature, D_{max}**
20	0.04	.300	80	72° 45′
25	0.04	.252	145	47° 30′
30	0.04	.221	230	24° 45′
35	0.04	.197	345	16° 45′
40	0.04	.178	490	11° 30′
45	0.04	.163	665	8° 30′

* $R = \dfrac{V^2}{15\,(e + f)}$ **$D = \dfrac{85{,}660\,(e + f)}{V^2}$

Note: Both minimum radius and maximum degree of curvature have been independently rounded for design purposes from their basic equations; there is not an exact conversion between the two values in the tables.

6.2 SUPERELEVATION

6.2.1 Definitions

1. Superelevation (S). Superelevation is the amount of cross slope or "bank" provided on a horizontal curve to help counterbalance the outward pull of a vehicle traversing the curve.

2. Maximum Superelevation (e_{max}). The maximum rate of superelevation (e_{max}) is an overall superelevation control used on a specific facility. Its selection depends on several factors including climatic conditions, terrain conditions, type of area (rural or urban) and highway functional class.

3. Superelevation Transition Length. The superelevation transition length is the distance required to transition the roadway from a normal crown section to the full superelevation (S) needed. Superelevation transition length is the sum of the tangent runout and superelevation runoff:

    a. Tangent Runout (TR). Tangent runout is the change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed (i.e., the outside lane(s) is level).

    b. Superelevation Runoff (L). Superelevation runoff is the change in cross slope from the end of the tangent runout (adverse crown removed) to a section that is sloped at the design superelevation (S).

4. 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.

5. Superelevation Breakover. Superelevation breakover is the algebraic difference (A) between the superelevated travel lane slope and shoulder slope on the outside of a horizontal curve.

6. Normal Crown (NC). The typical cross section on a tangent section (i.e., no superelevation).

7. Reverse Crown (RC). A superelevated roadway section which is sloped across the entire traveled way in the same direction and at a rate equal to the cross slope on a tangent section, typically 2%.

6.2.2 Distribution of Superelevation and Side Friction

As discussed in Section 6.1, the maximum degree of curvature is based on the e_{max} and f_{max} which apply to the facility. For curvature flatter than the maximum, a methodology must be applied to distribute superelevation and side friction for a given degree of curvature and design speed. The following will apply:

1. Open-Roadway Conditions. Superelevation and side friction are distributed by AASHTO Method 5, which allows S and f to gradually increase in a curvilinear manner up to e_{max} and f_{max}. See Reference (1) for more information.

2. Low-Speed Urban Streets. Superelevation and side friction are distributed by AASHTO Method 2, which allows f to increase up to f_{max} before any superelevation is introduced. The practical effect of AASHTO Method 2 is that superelevation is rarely warranted on low-speed urban streets ($V \leq 45$ mph). See

Reference (1) for more information. See the discussion in Section 6.2.4 for ODOT practices for where low-speed urban street conditions apply.

The distribution methodology for superelevation and side friction determines the development of superelevation criteria presented in Sections 6.2.3 and 6.2.4.

6.2.3 Superelevation Rates (Open-Roadway Conditions)

6.2.3.1 General

Open-roadway conditions are typically used on all rural highways and all urban facilities where V>45 mph. These types of facilities generally exhibit relatively uniform traffic operations. Therefore, for superelevation development, the flexibility normally exists to design horizontal curves with the more conservative AASHTO Method 5 (for distribution of superelevation and side friction). The following sections present the specific design criteria for superelevation rates assuming open-roadway conditions.

6.2.3.2 Maximum Superelevation Rate

The selection of a maximum rate of superelevation (e_{max}) depends upon several factors. These include urban/rural location, prevalent climatic conditions within Oklahoma, highway component (road or bridge), pavement type (paved or unpaved) and, for freeways, whether the facility is elevated. For open-roadway conditions, ODOT has adopted the following for the selection of e_{max}:

1. Rural, Paved Roadways. An e_{max} = 0.08 is typically used on all rural roadways which are paved. However, bridges on rural roadways typically have an e_{max} = 0.06 (see Comment #4). Exceptions to using e_{max} = 0.08 on the roadway should be evaluated on a case-by-case basis. Exceptions may include areas of frequent traffic congestion, where at-grade intersections occur on a horizontal curve, or where slow-moving vehicles frequently negotiate the curve (e.g., a curve near the entrance of an industrial facility). In these cases, an e_{max} = 0.06 may be more appropriate.

2. Urban Freeways. An e_{max} = 0.08 is typically used on urban freeways which are not elevated. Exceptions should be evaluated on a case-by-case basis.

3. Urban Non-Freeways (V>45 mph). An e_{max} = 0.06 is typically used on all urban non-freeways where the design speed is greater than 45 mph. Exceptions should be evaluated on a case-by-case basis. For example, an e_{max} = 0.08 may be appropriate where drainage considerations indicate.

4. Urban Non-Freeways (V≤45 mph). Desirably, these facilities will be designed assuming open-roadway conditions and an e_{max} = 0.06. However, it may be acceptable to assume low-speed urban street conditions and/or an e_{max} = 0.04. See Section 6.2.4 for more discussion.

5. Bridges. An e_{max} = 0.06 is typically used on all bridges where open-roadway conditions apply regardless of geographic location. This application means, for example, that a rural arterial will typically be designed with e_{max} = 0.08, but any curves on bridges on the arterial will typically be designed with e_{max} = 0.06.

6. Elevated Roadways. An e_{max} = 0.06 is typically used on all elevated roadways, either continuous or intermittent.

7. Gravel County Roads. Some counties within Oklahoma use an e_{max} = 0.10 on gravel roads. This is an acceptable design.

6.2.3.3 Superelevation Tables

Based on the selection of e_{max} and the use of AASHTO Method 5 to distribute S and f, the following tables allow the designer to select the superelevation rate (S) for any combination of degree of curvature (D) and design speed (V):

1. Table 6.2A applies to e_{max} = 0.08 for V = 30 mph to 70 mph.

2. Table 6.2B applies to e_{max} = 0.06 for V = 30 mph to 70 mph.

3. Table 6.2C applies to V = 20 and 25 mph for e_{max} = 0.06 and 0.08.

Where an e_{max} = 0.10 or e_{max} = 0.04 is used, the designer should see Reference (1) for superelevation rates.

6.2.3.4 Maximum Curvature Without Superelevation

A horizontal curve with a very small degree of curvature does not require superelevation. For a given design speed, the normal crown section (NC) used on tangent sections can be maintained throughout a very flat curve. On sharper curves for the same design speed, a point is reached where a slope of .020 across the total pavement width is desirable (RC). Table 6.3D provides the threshold (or maximum) degree of curvature for a normal crown section and a reverse crown section at various design speeds. This table applies to all highways where open-roadway conditions are used, and it applies to all e_{max} values.

6.2.4 Superelevation Rates (Low-Speed Urban Streets)

6.2.4.1 General

Low-speed urban street conditions may be used for superelevating streets in urban and urbanized areas where $V \leq 45$ mph. On these facilities, providing superelevation at horizontal curves is frequently impractical because of roadside conditions and, in some cases, may result in undesirable operational conditions. The following lists some of the characteristics of low-speed urban streets which may complicate superelevation development:

1. surface drainage considerations,

2. built-up roadside development,

3. frequent intersections and driveways,

4. non-uniform travel speeds,

5. limited right-of-way,

6. wide pavement areas, and

7. prevailing construction practices in Oklahoma.

On urban non-freeways where $V \leq 45$ mph, the designer should desirably apply open-roadway conditions; if so, the superelevation rates are determined by the criteria presented in Section 6.2.3. It is acceptable to design the horizontal curvature on these facilities assuming low-speed urban street conditions, based on the truck volume (T) as follows:

1. $T \leq 2\%$. Typically, it will be acceptable to assume low-speed urban street conditions.

2. $2\% < T \leq 4\%$. The acceptability of using low-speed urban street criteria will be determined on a case-by-case basis.

Table 6.2A

SUPERELEVATION RATE(S) AND LENGTH OF RUNOFF (L_2)
(Open-Roadway Conditions)

D	V = 30 mph		V = 35 mph		V = 40 mph		V = 45 mph		V = 50 mph		V = 55 mph		V = 60 mph		V = 65 mph		V = 70 mph	
	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)
0° 15′	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
0° 30′	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	59*	RC	60*
0° 45′	NC	0	NC	0	NC	0	NC	0	RC	48*	RC	52*	.022	59*	.025	74*	.028	84*
1° 00′	NC	0	NC	0	NC	0	RC	46*	.021	51*	.025	64*	.029	78*	.032	94*	.036	108*
1° 15′	NC	0	NC	0	RC	42*	.021	48*	.026	63*	.030	77*	.035	94*	.039	115*	.044	132*
1° 30′	NC	0	RC	40*	.021	45*	.025	57*	.030	72*	.035	90*	.041	110*	.046	135*	.051	153*
1° 45′	NC	0	RC	40*	.024	51*	.029	66*	.034	82*	.040	103*	.046	123*	.052	153*	.058	174*
2° 00′	RC	36*	.022	44*	.027	57*	.033	75*	.038	92*	.045	116*	.051	136*	.058	170*	.065	195*
2° 15′	RC	36*	.024	47*	.030	63*	.036	82*	.042	101*	.049	126*	.056	150*	.063	185*	.071	213
2° 30′	.021	38*	.027	53*	.033	70*	.039	88*	.046	111*	.053	136*	.061	163*	.068	200	.075	225
2° 45′	.023	42*	.029	57*	.035	74*	.042	95*	.049	118*	.057	146*	.065	174*	.072	211	.078	234
3° 00′	.025	45*	.031	61*	.038	80*	.045	102*	.053	128*	.060	154*	.068	182	.075	220		
3° 15′	.026	47*	.033	65*	.041	87*	.048	109*	.056	135*	.064	164*	.071	190	.078	229		
3° 30′	.028	51*	.035	69*	.043	91*	.051	116*	.058	140*	.066	169*	.074	198	.079	232		
3° 45′	.030	54*	.037	73*	.045	95*	.053	120*	.061	147*	.069	177	.076	203	.080	235		
4° 00′	.031	56*	.039	77*	.047	99*	.056	127*	.063	152	.071	182	.078	208				
4° 30′	.035	63*	.043	85*	.052	110*	.060	136	.068	164	.075	192	.080	214				
5° 00′	.038	69*	.047	92*	.055	116*	.064	145	.071	171	.078	200						
5° 30′	.041	74*	.050	98*	.059	124*	.067	152	.074	178	.080	205						
6° 00′	.043	78*	.053	104*	.062	131	.070	158	.077	185								
6° 30′	.046	83*	.055	108*	.064	135	.073	165	.079	190								
7° 00′	.048	87*	.058	114	.067	141	.075	170	.080	192								
7° 30′	.050	90*	.060	118	.069	145	.077	174	.080	192								
8° 00′	.053	96*	.062	122	.071	150	.078	176										
8° 30′	.054	98*	.064	126	.073	154	.079	179										
9° 00′	.056	101	.066	130	.075	158	.080	181										
9° 30′	.058	105	.068	134	.076	160	.080	181										
10° 00′	.060	108	.070	137	.078	164												
10° 30′	.061	110	.071	139	.078	164												
11° 00′	.063	114	.073	143	.079	166												
11° 30′	.064	116	.074	145	.080	168												
12° 00′	.065	117	.075	147	.080	168												
13° 00′	.068	123	.077	151														
14° 00′	.070	126	.079	155														
15° 00′	.072	130	.079	155														
16° 00′	.074	134	.080	157														
17° 00′	.076	137																
18° 00′	.077	139																
19° 00′	.078	141																
20° 00′	.079	143																
21° 00′	.080	144																
22° 00′	.080	144																
	D_{max} = 22° 45′		D_{max} = 16° 30′		D_{max} = 12° 15′		D_{max} = 9° 30′		D_{max} = 7° 30′		D_{max} = 6° 00′		D_{max} = 4° 45′		D_{max} = 3° 45′		D_{max} = 3° 00′	

e_{max} = 0.08

Key:
- D = Degree of curvature
- V = Design speed
- S = Superelevation Rate
- L_2 = Length of superelevation runoff (from adverse crown removed to full superelevation) for 2-lane roadway
- NC = Normal crown
- RC = Reverse crown; superelevate at normal crown slope (.020 typical)

* These are calculated lengths. See Table 6.2G for minimum superelevation runoff lengths.

Table 6.2B

SUPERELEVATION RATE(S) AND LENGTH OF RUNOFF (L_2) (Open-Roadway Conditions)

D	V = 30 mph		V = 35 mph		V = 40 mph		V = 45 mph		V = 50 mph		V = 55 mph		V = 60 mph		V = 65 mph		V = 70 mph	
	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)
0° 15′	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
0° 30′	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	59*	RC	60*
0° 45′	NC	0	NC	0	NC	0	NC	0	NC	0	RC	52*	.021	56*	.024	71*	.026	78*
1° 00′	NC	0	NC	0	NC	0	RC	46*	RC	48*	.023	59*	.027	72*	.030	88*	.033	99*
1° 15′	NC	0	NC	0	RC	42*	RC	46*	.024	58*	.028	72*	.032	86*	.036	106*	.040	120*
1° 30′	NC	0	RC	40*	RC	42*	.024	55*	.028	68*	.032	82*	.037	99*	.041	121*	.046	138*
1° 45′	NC	0	RC	40*	.023	49*	.027	61*	.032	77*	.036	93*	.041	110*	.046	135*	.051	153*
2° 00′	RC	36*	.021	42*	.025	53*	.030	68*	.035	84*	.040	103*	.045	120*	.050	147*	.055	165*
2° 15′	RC	36*	.023	45*	.028	59*	.033	75*	.038	92*	.043	110*	.048	128*	.053	156*	.057	171*
2° 30′	RC	36*	.025	49*	.030	63*	.035	79*	.040	96*	.045	116*	.051	136*	.056	164*	.059	177*
2° 45′	.022	40*	.027	53*	.032	68*	.038	86*	.042	101*	.048	123*	.053	142*	.058	170*	.060	180*
3° 00′	.023	42*	.029	57*	.034	72*	.040	91*	.045	108*	.050	128*	.055	147*	.059	173*	D_{max} = 2° 45′	
3° 15′	.025	45*	.030	59*	.036	76*	.041	93*	.046	111*	.052	133*	.057	152*	.060	176*		
3° 30′	.026	47*	.032	63*	.038	80*	.043	98*	.048	116*	.054	139*	.058	155*	D_{max} = 3° 30′			
3° 45′	.028	51*	.034	67*	.039	82*	.045	102*	.050	120*	.055	141*	.059	158*				
4° 00′	.029	53*	.035	69*	.041	87*	.046	104*	.052	125*	.057	146*	.060	160*				
4° 30′	.031	56*	.038	75*	.043	91*	.049	111*	.054	130*	.059	151*	D_{max} = 4° 15′					
5° 00′	.034	62*	.040	79*	.046	97*	.051	116*	.056	135*	.060	154*						
5° 30′	.036	65*	.042	83*	.048	101*	.054	122*	.058	140*	D_{max} = 5° 15′							
6° 00′	.038	69*	.044	87*	.050	105*	.055	125*	.059	142*								
6° 30′	.039	71*	.045	89*	.052	110*	.057	129*	.060	144*								
7° 00′	.041	74*	.047	92*	.053	112*	.058	131*	D_{max} = 6° 45′									
7° 30′	.042	76*	.049	96*	.055	116*	.059	134*										
8° 00′	.043	78*	.050	98*	.056	118*	.060	136*										
8° 30′	.045	81*	.051	100*	.057	120*	.060	136*										
9° 00′	.046	83*	.053	104*	.058	122*	D_{max} = 8° 45′											
9° 30′	.047	85*	.054	106*	.059	124*												
10° 00′	.048	87*	.055	108*	.059	124*												
10° 30′	.049	89*	.056	110*	.060	126												
11° 00′	.050	90*	.057	112*	.060	126												
11° 30′	.051	92*	.058	114*														
12° 00′	.052	94*	.058	114*	D_{max} = 11° 15′													
13° 00′	.054	98*	.059	116*														
14° 00′	.055	99*	.060	118*														
15° 00′	.056	101	.060	118*														
16° 00′	.058	105	D_{max} = 15° 00′															
17° 00′	.058	105																
18° 00′	.059	107																
19° 00′	.060	108																
20° 00′	.060	108																
	D_{max} = 21° 00′																	

e_{max} = 0.06

Key:
- D = Degree of curvature
- V = Design speed
- S = Superelevation Rate
- L_2 = Length of superelevation runoff (from adverse crown removed to full superelevation) for 2-lane roadway
- NC = Normal crown
- RC = Reverse crown; superelevate at normal crown slope (.020 typical)

* These are calculated lengths. See Table 6.2G for minimum superelevation runoff lengths.

Table 6.2C

SUPERELEVATION RATE(S) AND LENGTH OF RUNOFF (L_2)
(V=20 mph and 25 mph, Open-Roadway Conditions)

D	e_{max} = 0.08, V = 20 mph		e_{max} = 0.06, V = 20 mph		e_{max} = 0.08, V = 25 mph		e_{max} = 0.06, V = 25 mph	
	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)	S	L_2(ft)
0° 15′	NC	0	NC	0	NC	0	NC	0
0° 30′	NC	0	NC	0	NC	0	NC	0
0° 45′	NC	0	NC	0	NC	0	NC	0
1° 00′	NC	0	NC	0	NC	0	NC	0
1° 15′	NC	0	NC	0	NC	0	NC	0
1° 30′	NC	0	NC	0	NC	0	NC	0
1° 45′	NC	0	NC	0	NC	0	NC	0
2° 00′	NC	0	NC	0	NC	0	NC	0
2° 15′	NC	0	NC	0	NC	0	NC	0
2° 30′	NC	0	NC	0	RC	35*	NC	0
2° 45′	NC	0	NC	0	RC	35*	RC	35*
3° 00′	NC	0	NC	0	RC	35*	RC	35*
3° 15′	NC	0	NC	0	RC	35*	RC	35*
3° 30′	NC	0	NC	0	.021	36*	RC	35*
3° 45′	RC	32*	RC	32*	.023	40*	.021	36*
4° 00′	RC	32*	RC	32*	.024	41*	.023	40*
4° 30′	RC	32*	RC	32*	.027	47*	.025	43*
5° 00′	.021	34*	RC	32*	.029	50*	.027	47*
5° 30′	.023	37*	.022	36*	.031	53*	.029	50*
6° 00′	.025	40*	.023	37*	.034	58*	.031	53*
6° 30′	.026	42*	.025	40*	.036	62*	.032	55*
7° 00′	.028	45*	.026	42*	.038	65*	.034	58*
7° 30′	.030	48*	.027	44*	.040	69*	.035	60*
8° 00′	.031	50	.029	47*	.042	72*	.037	64*
8° 30′	.033	53	.030	48*	.044	75	.038	65*
9° 00′	.035	56	.031	50	.046	79	.039	67*
9° 30′	.036	58	.032	52	.047	81	.040	69*
10° 00′	.037	60	.033	53	.049	84	.041	70*
10° 30′	.039	63	.034	55	.050	86	.042	72*
11° 00′	.040	64	.035	56	.052	89	.043	74*
11° 30′	.041	66	.036	58	.053	91	.043	74*
12° 00′	.043	69	.037	60	.054	93	.044	75
13° 00′	.045	72	.038	61	.057	98	.046	79
14° 00′	.047	76	.039	63	.059	101	.047	81
15° 00′	.049	79	.040	64	.061	104	.049	84
16° 00′	.051	82	.041	66	.063	108	.050	86
17° 00′	.053	85	.043	69	.065	111	.051	87
18° 00′	.054	87	.044	71	.067	115	.053	91
19° 00′	.056	90	.045	72	.068	116	.054	93
20° 00′	.057	91	.046	74	.070	120	.055	94
21° 00′	.058	93	.047	76	.071	121	.056	96
22° 00′	.060	96	.048	77	.073	125	.057	98
23° 00′	.061	98	.048	77	.074	127	.057	98
24° 00′	.062	99	.049	79	.075	128	.058	99
25° 00′	.063	101	.050	80	.076	130	.058	99
26° 00′	.064	103	.051	82	.077	132	.059	101
27° 00′	.066	106	.052	83	.078	133	.059	101
28° 00′	.067	107	.052	83	.078	133	.060	103
29° 00′	.068	109	.053	85	.079	135	.060	103
30° 00′	.069	111	.054	87	.079	135	.060	103
31° 00′	.070	112	.054	87	.080	137		
32° 00′	.070	112	.055	88	.080	137		
33° 00′	.071	114	.056	90	.080	137		
34° 00′	.072	115	.056	90				
35° 00′	.073	117	.057	91				
36° 00′	.074	119	.057	91				
37° 00′	.074	119	.057	91				
38° 00′	.075	120	.058	93				
39° 00′	.076	122	.058	93				
40° 00′	.076	122	.059	95				
41° 00′	.077	123	.059	95				
42° 00′	.077	123	.059	95				
43° 00′	.078	125	.059	95				
44° 00′	.078	125	.060	96				
45° 00′	.079	127	.060	96				
46° 00′	.079	127	.060	96				
47° 00′	.079	127	.060	96				
48° 00′	.079	127	.060	96				
49° 00′	.080	128	.060	96				
50° 00′	.080	128						
51° 00′	.080	128						
52° 00′	.080	128						
53° 00′	.080	128						
	D_{max} = 53° 30′		D_{max} = 49° 15′		D_{max} = 33° 30′		D_{max} = 30° 45′	

Key:

D = Degree of curvature
V = Design speed
S = Superelevation Rate
L_2 = Length of superelevation runoff (from adverse crown removed to full superelevation) for 2-lane roadway
NC = Normal crown
RC = Reverse crown; superelevate at normal crown slope (.020 typical)

* These are calculated lengths. See Table 6.2G for minimum superelevation runoff lengths.

Table 6.2D

DEGREE OF CURVATURE FOR NORMAL CROWN SECTION AND REVERSE CROWN SECTION (Open-Roadway Conditions)

Design Speed (mph)	Degree of Curvature		
	Normal Crown	Reverse Crown	See Tables 6.2A, 6.2B, or 6.2C
20	D < 3° 25′	3° 25′ ≤ D ≤ 4° 29′	D > 4° 29′
25	D < 2° 30′	2° 30′ ≤ D ≤ 3° 23′	D > 3° 23′
30	D < 1° 45′	1° 45′ ≤ D ≤ 2° 18′	D > 2° 18′
35	D < 1° 25′	1° 25′ ≤ D ≤ 1° 54′	D > 1° 54′
40	D < 1° 04′	1° 04′ ≤ D ≤ 1° 22′	D > 1° 22′
45	D < 0° 53′	0° 53′ ≤ D ≤ 1° 08′	D > 1° 08′
50	D < 0° 43′	0° 43′ ≤ D ≤ 0° 55′	D > 0° 55′
55	D < 0° 37′	0° 37′ ≤ D ≤ 0° 48′	D > 0° 48′
60	D < 0° 31′	0° 31′ ≤ D ≤ 0° 41′	D > 0° 41′
65	D < 0° 28′	0° 28′ ≤ D ≤ 0° 34′	D > 0° 34′
70	D < 0° 25′	0° 25′ ≤ D ≤ 0° 31′	D > 0° 31′

Notes:

1. Table is based on e_{max} = 0.08, but it may be used for both e_{max} = 0.06 and e_{max} = 0.08.

2. The limit for NC is based on a theoretical superelevation rate of +.015. The upper limit for RC is based on a theoretical superelevation rate of +.020.

3. T > 4%. It will rarely be acceptable to assume low-speed urban street conditions.

Because of the criticality of trucks negotiating horizontal curves with a high demand for side friction, T includes all trucks considered "light" trucks and heavier.

Where used, AASHTO Method 2 is used to distribute superelevation and side friction in determining superelevation rates for the design of horizontal curves on low-speed urban streets. In addition, as discussed in Section 6.1, relatively high side-friction factors are used. The practical impact is that superelevation is rarely warranted on these facilities. The following sections present the design criteria for superelevation rates assuming low-speed urban street conditions:

6.2.4.2 Maximum Superelevation Rates

The following will apply to e_{max}, whether open-roadway or low-speed urban street conditions are used:

1. Desirable. An e_{max} = 0.06 will desirably be used.

2. Acceptable. It is acceptable to use an e_{max} = 0.04.

6.2.4.3 Superelevation Figure

Figure 6.2A is used to determine the superelevation rate for horizontal curves of known radii on a low-speed urban street of known design speed. The figure is divided into three areas. The following examples illustrate how to use Figure 6.2A for site conditions within each area.

* * * * * * * * * *

Example 6-1

Given: Design speed = 25 mph
Radius = 200′
Cross slope (on tangent) = 2%

Problem: Determine the superelevation rate.

Solution: From Figure 6.2A, the required superelevation rate = -.043 . Therefore, a normal crown section may be maintained throughout the curve (i.e., S = -.020).

Example 6-2

Given: Design speed = 35 mph
Radius = 400′

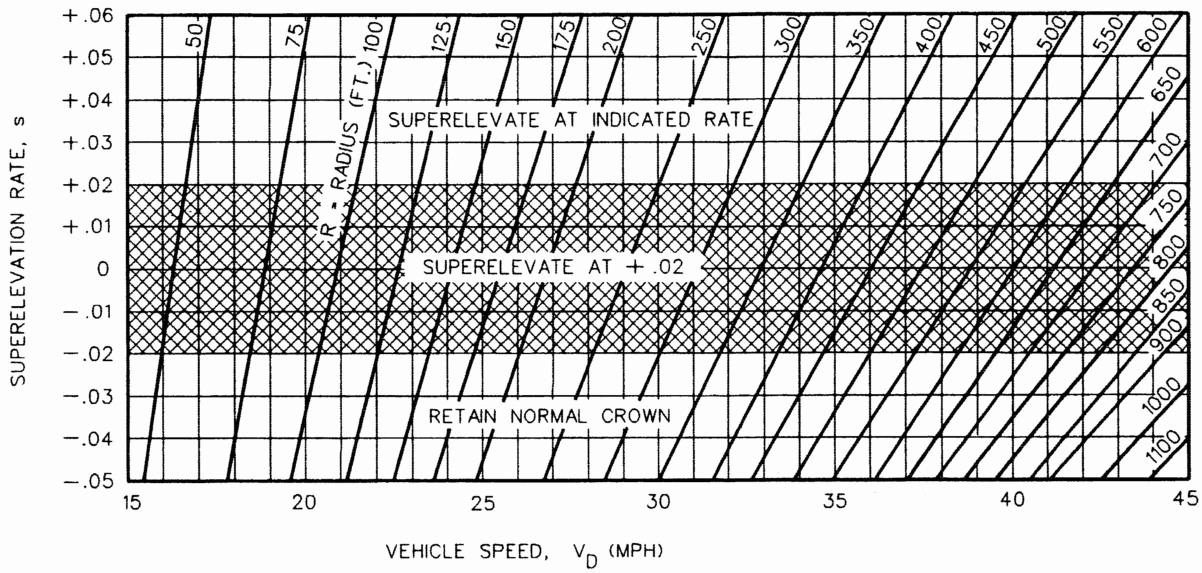
Problem: Determine the superelevation rate.

Solution: From Figure 6.2A, the required superelevation rate = +.007. This falls in the area where the roadway should be uniformly superelevated at the cross slope of the roadway on tangent (typically .020). This is the desirable treatment. However, it is acceptable to superelevate the roadway at the theoretical superelevation rate (+.007), if this is consistent with field conditions (e.g., surface drainage will work properly).

Example 6-3

Given: Design speed = 40 mph
Radius = 500′

Problem: Determine the superelevation rate.



Notes:

1. Figure denotes three areas for the determination of superelevation rates. See Section 6.2.4 for examples on how to use the figure.

2. Desirably, e_{max} = 0.06; it is acceptable to use e_{max} = 0.04.

3. The basic equation for the figure is:

$$R = \frac{V^2}{15\ (e + f)}$$

where:

R = curve radius, ft
V = design speed, mph
e = superelevation rate
f = side-friction factor

For design purposes, rounded minimum radii are used as follows:

	Design Speed, mph					
	20	25	30	35	40	45
R_{min} (ft), e = .040	80	145	230	345	490	665
R_{min} (ft), e = .060	75	135	215	320	450	610

Source: (1)

SUPERELEVATION RATES
(Low-Speed Urban Streets)

Figure 6.2A

Solution: Figure 6.2A yields a required superelevation rate = +.035. Therefore, the entire pavement should be transitioned to this rate.

* * * * * * * * * * *

6.2.4.4 Minimum Radii Without Superelevation

As with highways designed using open-roadway conditions, horizontal curves with sufficiently large radii do not require superelevation; i.e., the normal crown section can be maintained around a curve. The threshold exists where the theoretical superelevation equals -.020. The lower boundary of the shaded area in Figure 6.2A illustrates this threshold. For convenience, Table 6.2E presents corresponding radii for normal crown (NC) and reverse crown (RC).

6.2.5 Transition Length (Open-Roadway Conditions)

As defined in Section 6.2.1, the superelevation transition length is the distance required to transition the roadway from a normal crown section to the full design superelevation (S) (as determined from the tables based on the selected e_{max}). The superelevation transition length is the sum of the tangent runout distance (TR) and superelevation runoff length (L).

6.2.5.1 Two-Lane Roadways

Superelevation Runoff

Tables 6.2A, 6.2B and 6.2C present the calculated superelevation runoff lengths (L_2) for two-lane roadways for various combinations of degree of curvature and design speed. The lengths are calculated as follows:

$$L_2 = S \times W \times RS \geq L_{min}$$

where:

L_2 = Superelevation runoff length for a two-lane roadway.

W = Width of rotation (assumed to be 12′).

RS = Relative longitudinal slope between the profile grade and far edge of two-lane roadway (see Table 6.2F)

S = Superelevation rate

L_{min} = Minimum superelevation runoff length regardless of calculated L_2 (see Table 6.2G)

Table 6.2F provides the maximum RS values for each design speed. The calculated L_2 values are subject to minimum lengths (L_{min}), which are based on approximately two seconds of travel time. These are presented in Table 6.2G.

The superelevation runoff lengths for two-lane roadways apply to the following:

1. a two-lane, two-way roadway rotated about its centerline;
2. a two-lane, two-way roadway rotated about either edge of travelway; or
3. either directional roadway of a four-lane divided facility, where the median width exceeds 40 ft, rotated either about its centerline or edge of travelway.

Table 6.2E

RADII FOR NORMAL CROWN SECTION AND REVERSE CROWN SECTION
(Low-Speed Urban Streets)

Design Speed (mph)	Curve Radii		
	Normal Crown	Reverse Crown*	See Figure 6.2A
20	R > 95.24′	95.24′ ≤ R ≤ 83.33′	R > 83.33′
25	R > 179.60′	179.60′ ≤ R ≤ 153.19′	R > 153.19′
30	R > 298.51′	298.51′ ≤ R ≤ 248.96′	R > 248.96′
35	R > 461.39′	461.39′ ≤ R ≤ 376.34′	R > 376.34′
40	R > 675.11′	675.11′ ≤ R ≤ 538.72′	R > 538.72′
45	R > 944.06′	944.06′ ≤ R ≤ 737.70′	R > 737.70′

* The shaded area in Figure 6.2A reflects these radii ranges. In this range, it is desirable to reverse the crown and superelevate the roadway at a uniform slope of +.020. However, it is acceptable to superelevate at the theoretical rate from Figure 6.2A, if consistent with field conditions.

Note: The limit for NC is based on a theoretical superelevation rate of -.020. The upper limit for RC is based on a theoretical superelevation rate of +.020. Radii are calculated from:

$$R = \frac{V^2}{15\,(e + f)}$$

Table 6.2F

RELATIVE LONGITUDINAL SLOPES (RS)

Design Speed (mph)	RS (minimum)	Grade Difference G(%) (max.)*
20	133	0.75
25	142	0.70
30	150	0.67
35	163	0.61
40	175	0.58
45	188	0.53
50	200	0.50
55	213	0.47
60	222	0.45
65	244	0.41
70	250	0.40

* G(%) = 1/RS x 100

Table 6.2G

MINIMUM SUPERELEVATION RUNOFF LENGTHS (L_{min})

Design Speed (mph)	Minimum Superelevation Runoff Lengths (ft)
20	50
25	75
30	100
35	110
40	125
45	135
50	150
55	160
60	175
65	190
70	200

Tangent Runout

The tangent runout distance is calculated as follows:

$$TR = S_{normal}/K$$

where:

TR = Tangent runout distance for a two-lane roadway

S_{normal} = Travel lane cross slope on tangent (typically .02)

K = Transition rate = S/L_2

S = Design superelevation rate (i.e., full superelevation for horizontal curve)

L_2 = Superelevation runoff length for a two-lane roadway

The use of the transition rate, K, will ensure that the relative longitudinal gradient of the tangent runout equals that of the superelevation runoff.

6.2.5.2 Multilane Highways

Superelevation Runoff

The superelevation runoff distance for multilane highways is calculated by:

$$L = L_2 \times C \geq L_{min}$$

where:

L = Superelevation runoff length

L_2 = Superelevation runoff length for a two-lane roadway

C = Ratio of runoff length for a multilane highway to runoff length for a two-lane roadway (see Table 6.2H)

L_{min} = Minimum superelevation runoff length regardless of calculated L (see Table 6.2G)

Table 6.2H

C VALUES
(Superelevation Runoff Lengths)

Number of Lanes (Both Directions)	C	
	Desirable	Minimum
2	1.0	1.0
3	1.5	1.2
4	2.0	1.5
5	2.5	1.8
6	3.0	2.0
7	3.5	2.3
8	4.0	2.5

Note: Where the median width is greater than 40 ft, each roadway is evaluated separately. For example, if a 6-lane divided facility has a median width greater than 40 ft, the number of lanes in one direction (three) is used in the table to read C = 1.5 desirable and C = 1.2 minimum.

Tangent Runout

Where all travel lanes have the same cross slope on tangent, the tangent runout distance is calculated as follows:

$$TR = S_{normal}/K$$

where:

TR = Tangent runout distance for the multilane highway

S_{normal} = Travel lane cross slope on tangent (typically .02)

K = Transition rate = S/L

S = Design superelevation rate (i.e., full superelevation for horizontal curve)

L = Superelevation runoff length for the multilane highway

The use of the transition rate, K, will ensure that the relative longitudinal gradient of the tangent runout equals that of the superelevation runoff.

Where the travel lanes on tangent have variable cross slopes, the tangent runout distance is calculated in two steps. First, it is necessary to flatten the steeper cross slope to match the flatter cross slope as follows:

$$TR_1 = \Delta\ Slope_1/K$$

where:

TR_1 = Distance needed to rotate the steeper cross slope to match the flatter cross slope.

$\Delta Slope_1$ = Difference between the two cross slopes, ft/ft

K = Transition rate = S/L

After TR_1 has been implemented, a uniform pavement slope is present on the outside travel lanes. From this point, the second segment of the tangent runout is calculated as follows:

$$TR_2 = \Delta\ Slope_2/K$$

where:

TR_2 = Distance needed to rotate the cross slope of the outside travel lanes to a level slope

$\Delta Slope_2$ = Difference between the cross slope of the outside travel lanes and level (i.e., zero slope)

K = Transition rate = S/L

6.2.5.3 Application of Transition Length

Once the superelevation runoff and tangent runouts have been calculated, the designer must determine how to fit the length in the horizontal and vertical planes. The following will apply:

1. Simple Curves. Typically, 75% of the superelevation runoff length will be placed on the tangent and 25% on the curve. Exceptions to this practice may be necessary to meet field conditions. The generally acceptable range is 60% - 80% on tangent and 20% - 40% on curve. In extreme cases, the superelevation runoff may be distributed 50% - 100% on the tangent and 0% - 50% on the curve (e.g., to avoid placing any superelevation development on a bridge).

2. Spiral Curves. The superelevation runoff length is typically fitted to the spiral curve length (TS to SC and CS to ST). All of the tangent runout is placed on the tangent before the TS and after the ST.

3. Compound/Reverse Curves. See Sections 6.2.9 and 6.2.10 for a discussion on superelevation development for compound and reverse curves.

4. Field Application (Vertical Profile). At the beginning and ending of the superelevation transition, angular breaks would occur in the profile if not smoothed. These abrupt angular breaks are smoothed by field personnel during construction, as discussed in the *ODOT Standard Specifications for Highway Construction*, by the insertion of short vertical curves at the two angle points. Designers should graphically or numerically work the transition areas to check potential drainage runoff pocketing. As a guide, the transitions should have a length in feet equal to the design speed in mph.

5. Ultimate Development. If the facility is planned for ultimate development to an expanded facility, the designer should, where practical, reflect this in the initial superelevation transition application. For example, a four-lane divided facility may be planned for an ultimate six-lane divided facility. Therefore, the superelevation runoff length for the initial four-lane facility should be consistent with the future requirements of the six-lane facility.

6. Typical Figures/Examples. Sections 6.2.12 and 6.2.13 present typical figures for superelevation development and examples for the calculation of tangent runout and superelevation runoff.

6.2.6 Transition Length (Low-Speed Urban Streets)

Low-speed urban streets are those urban facilities where $V \leq 45$ mph. If open-roadway conditions are used to determine the superelevation rate (Section 6.2.3), then the superelevation transition length should be determined by the criteria for open-roadway conditions (Section 6.2.5). If the superelevation rate is determined by low-speed urban street conditions (Section 6.2.4), then the superelevation transition lengths may be determined by the criteria that follow.

6.2.6.1 Two-Lane Roadways

<u>Superelevation Runoff</u>

Table 6.2I presents the minimum superelevation runoff (L_2) lengths for two-lane roadways. With the use of a straight-line interpolation for intermediate superelevation rates, the superelevation runoff may be calculated for any design speed and superelevation rate.

<u>Tangent Runout</u>

The tangent runout distance is calculated as follows:

width / slope

$$TR = S_{normal}/K = \frac{S_{normal}\,L_2}{S} = \frac{wP}{SER\text{-rate}}$$

where:

TR	=	Tangent runout distance for a two-lane roadway
S_{normal}	=	Travel lane cross slope on tangent (typically .02)
K	=	Transition rate = S/L_2
S	=	Design superelevation rate (i.e., full superelevation for horizontal curve)
L_2	=	Superelevation runoff length for a two-lane roadway

The use of the transition rate, K, will ensure that the relative longitudinal gradient of the tangent runout equals that of the superelevation runoff.

6.2.6.2 Multilane Highways

Section 6.2.5 presents criteria for superelevation transition lengths for multilane highways assuming open-roadway conditions. The procedures and formulas also apply to multilane highways assuming low-speed urban street conditions, with the following changes:

1. L_2, the superelevation runoff length for two-lane roadways, will be based on Table 6.2I.

2. L_{min} will be based on Table 6.2I.

6.2.6.3 Application of Transition Length

The criteria presented in Section 6.2.5.3 for open-roadway conditions also apply to low-speed urban streets.

6.2.7 <u>Axis of Rotation</u>

The following discusses the axis of rotation for two-lane, two-way highways and multilane highways. Section 6.2.12 presents typical figures illustrating the application of the axis of rotation in superelevation development.

6.2.7.1 Two-Lane, Two-Way Highways

The axis of rotation will typically be about the centerline of the roadway on two-lane, two-way highways. This method will yield the least amount of elevation differential between the pavement edges and their normal profiles. It is acceptable to rotate about the inside or outside edge of the travelway. This may be

Table 6.2I

SUPERELEVATION RUNOFF LENGTHS
(Low-Speed Urban Streets, Two-Lane Roadways)

Design Speed (mph)	Superelevation Rate (S)	Minimum Radius (ft)	Superelevation Runoff (L_2) (ft)
20	.02 .04 .06	85 80 75	65 70 75
25	.02 .04 .06	155 145 135	70 75 80
30	.02 .04 .06	250 230 215	75 85 90
35	.02 .04 .06	380 345 320	85 95 100
40	.02 .04 .06	540 490 450	95 105 115
45	.02 .04 .06	740 665 610	110 120 130

Note: For superelevation rates intermediate between those in table, use a straightline interpolation to calculate the superelevation runoff length.

necessary to meet field conditions (e.g., drainage on a curbed facility, roadside development).

On a two-lane highway with an auxiliary lane (e.g., climbing lane), the axis of rotation will typically be about the centerline of the two through lanes.

6.2.7.2 Multilane Divided Highways

The axis of rotation will typically be about the profile grade or base line(s) for the multilane facility. The two median edges are typically the two base lines on divided facilities. When these are used as the axes of rotation, the median will remain in the same horizontal plane throughout the curve.

Depending upon field conditions, it may be acceptable to use any travel lane edge as the axis of rotation. For example, on a four-lane divided facility, the axes of rotation may be about the centerlines of the two roadways. Unless the two roadways are on independent alignment, this method results in different elevations at the median edges and, therefore, a compensating slope is necessary across the median.

Several highway features may significantly influence superelevation development for multilane divided highways. These include guardrail, median barriers, drainage and other field conditions. The designer should carefully consider the intended function of these features and ensure that the superelevated section and selected axis of rotation does not compromise their operation. In addition, the designer should consider the likely ultimate development of the facility and select an axis of rotation that will lend itself to future expansion.

6.2.8 Shoulder Superelevation

Figure 6.2B illustrates shoulder treatment on superelevated sections. The examples in Section 6.2.13 illustrate the details of shoulder superelevation. The following discusses specific criteria.

6.2.8.1 High Side (Outside Shoulder)

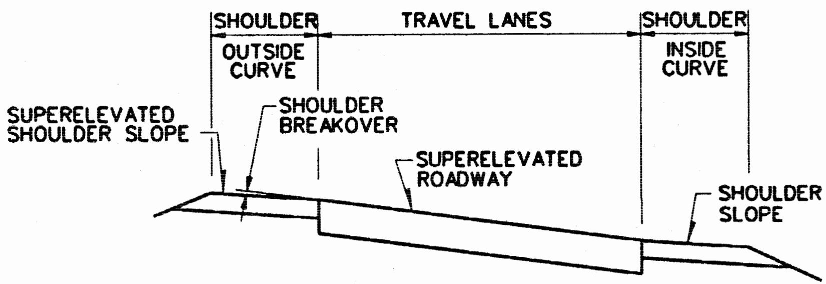
On the high side of superelevated sections, there will be a break in the cross slopes of the travel lane and shoulder. The following criteria will apply to this shoulder breakover:

1. Algebraic Difference. If practical, the breakover should not exceed 7%. It is acceptable for the breakover to be 8%.

2. Minimum Shoulder Slope. The minimum cross slope is 1% on the high-side shoulder in a superelevated section.

3. Direction of Slope. If practical, the shoulder should slope away from the travel lane. However, in some cases it will not be possible to meet the criteria for #'s 1 and 2, except by sloping the shoulder in the same direction as the travel lane. For example, if S = .075, a 1% shoulder slope away from the travel lane would yield an 8.5% breakover. This exceeds the recommended 8% limit.

4. Shoulder as Deceleration Lane. At some intersections, drivers may use a paved shoulder as a right-turn lane on a superelevated horizontal curve. Chapter Nine presents cross slope breakover criteria between a turning roadway and a through travel lane at an intersection at-grade. Where the shoulder is used by turning vehicles, the designer may want to use the turning roadway breakover criteria (4% to 5%) rather than the 8% maximum breakover.



Note: See Section 6.2.8 for criteria on treatment of shoulders through superelevated curves.

SHOULDER TREATMENT THROUGH SUPERELEVATED CURVE

Figure 6.2B

6.2.8.2 Low Side (Inside Shoulder)

On the low side of a superelevated section, the following will apply:

1. Bituminous Shoulder. ODOT typical practice is to retain the normal shoulder slope throughout the entire superelevated section. See Chapter Twelve for ODOT criteria. However, where the shoulder is currently used as a travel lane or will likely be converted to a travel lane in the future, the shoulder superelevation should be as described in #2.

2. Concrete Shoulder. For PC concrete shoulders, ODOT typical practice is to retain the normal shoulder slope until the adjacent superelevated travel lane reaches that slope. The shoulder is then superelevated concurrently with the travel lane until the design superelevation is reached (i.e., the inside shoulder and travel lane will remain in a plane section).

6.2.9 Compound Curves

Superelevation development for compound curves requires the consideration of several factors. For two-lane roadways, these are discussed in the following sections for two Cases:

1. Case I: The distance between the PC and PCC is 300 ft or less.

2. Case II: The distance between the PC and PCC is greater than 300 ft.

6.2.9.1 Case I

For Case I, superelevation development for compound curvature on two-lane roadways should meet the following objectives:

1. Relative Longitudinal Gradient (RS). A uniform RS should be provided throughout the superelevation transition (from normal section to full superelevation at the PCC).

2. Superelevation at PCC. The criteria in Section 6.2.3 (open-roadway conditions) or 6.2.4 (low-speed urban street conditions) will yield the design superelevation (S_2) for the second curve. S_2 should be reached at the PCC.

3. Superelevation at PC. Section 6.2.3 or 6.2.4 will yield the design superelevation (S_1) for the first curve. At the PC, ¾ S_1 should be reached.

4. Superelevation Runoff Length. Section 6.2.3 or 6.2.4 will yield the superelevation runoff (L) for the first curve. The superelevation should be developed such that ¾ L is reached at the PC.

5. Tangent Runout Length. TR will be determined as described in Section 6.2.3 or 6.2.4.

To meet all or most of these objectives, the designer may need to try several combinations of curve lengths, degree of curvature and longitudinal gradients to find the most practical design. Section 6.2.12 presents a typical figure for Case I superelevation development for a compound curve. Section 6.2.13 presents an example problem for Case I.

6.2.9.2 Case II

For Case II, the distance between the PC and PCC (>300 ft) is normally large enough to allow the two curves to be evaluated individually. Therefore, the superelevation development on two-lane roadways should meet the following objectives for Case II:

1. First Curve. Superelevation should be developed assuming the curve is an independent simple curve. Therefore, the criteria in Section 6.2 for superelevation rate, transition length and distribution between tangent and curve apply.

2. Intermediate Treatment. Superelevation for the first curve (S_1) is reached a distance of ¼ the superelevation runoff length beyond the PC. S_1 is maintained until it is necessary to develop the needed superelevation rate (S_2) for the second curve.

3. Second Curve. Assuming the second curve has a higher degree of curvature than the first curve, a higher rate of superelevation will be required ($S_2 > S_1$). S_2 should be reached at the PCC. The distance needed for the additional superelevation development is not specified, except that the maximum RS for the highway design speed must not be exceeded. One logical treatment would be to apply the same RS used for the superelevation transition of the first curve. This would provide a uniform change in gradient for the driver negotiating the compound curve.

Section 6.2.12 presents a typical figure for Case II superelevation development for a compound curve.

6.2.9.3 Multilane Highways

Superelevation development for compound curvature on multilane highways should, as practical, be designed to:

1. meet the principles of superelevation development for simple curves on multilane highways (see applicable criteria in Section 6.2); and

2. meet the objectives for Case I or Case II as described for two-lane roadways.

The treatment for multilane highways will be determined on a case-by-case basis, reflecting individual site conditions.

6.2.10 Reverse Curves

Reverse curves are two closely spaced simple curves with deflections in opposite directions. For this situation, it may not be practical to achieve a normal crown section between the curves. A plane section continuously rotating about its axis (e.g., the centerline) can be maintained between the two curves, if they are close enough together. The designer should adhere to the applicable superelevation development criteria for each curve. The following will apply to reverse curves:

1. Normal Section. The designer should not attempt to achieve a normal tangent section between reverse curves unless the normal section can be maintained for a minimum of two seconds of travel time, and the superelevation transition requirements can be met for both curves. These criteria yield the following minimum tangent distance (between PT of first curve and PC of second curve):

   $$L_{tan} \geq .75L_1 + TR_1 + 2(1.467V) + TR_2 + .75L_2$$

   where:

   L_{tan} = Tangent distance between PT and PC, ft

   L_1 = Superelevation runoff length for first curve, ft

   TR_1 = Tangent runout length for first curve, ft

   V = Design speed, mph

   TR_2 = Tangent runout length for second curve, ft

   L_2 = Superelevation runoff length for second curve, ft

   As a modification to the equation for L_{tan}, developing a normal section is acceptable if between 60% - 80% of L_1 and L_2 can be provided on the intervening tangent.

2. Continuously Rotating Plane. If a normal section is not provided, the pavement will be continuously rotated in a plane about its axis. In this case, the minimum distance between the PT and PC will be that needed to meet the superelevation transition requirements:

   $$L_{tan} = .75L_1 + TR_1 + TR_2 + .75L_2$$

   where terms are as defined in Comment #1. As a modification to the equation for L_{tan}, it is acceptable to provide between 60% - 80% of L_1 and L_2 on the intervening tangent.

6.2.11 Bridges

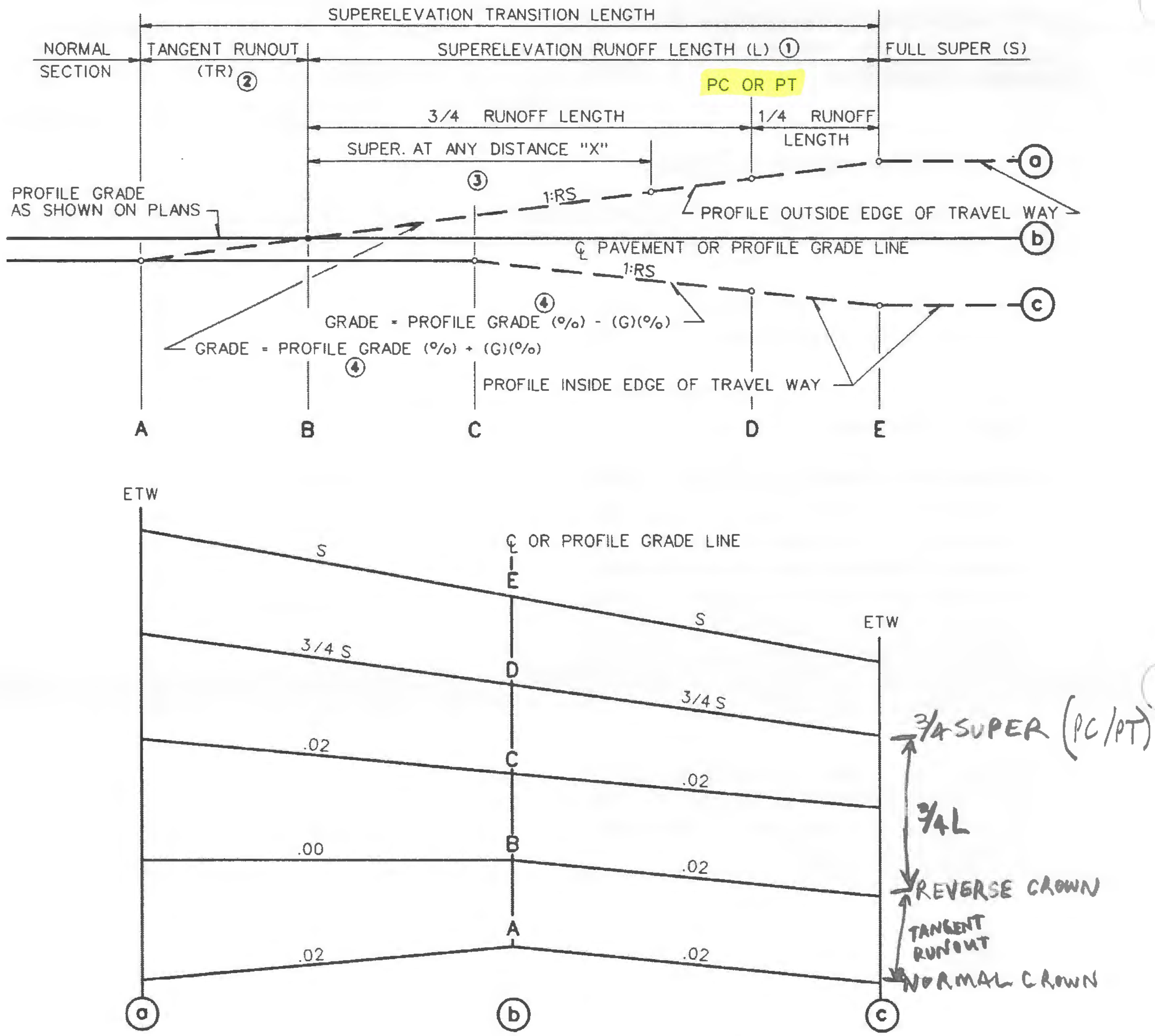
If practical, horizontal curves and superelevation transitions should be avoided on bridges. The designer should not, however, avoid placing a curve on a bridge if this results in sharp horizontal curves on either approaching roadway. Where a curve is necessary on a bridge, a simple curve should be used on the bridge and, if practical, any superelevation development should be placed on the approaching roadway. In some cases, however, superelevation transitions are unavoidable on bridges; if properly designed, most bridges will operate adequately where this occurs.

As noted in Section 6.2.3, an e_{max} = 0.06 is typically used on bridges regardless of urban/rural location.

6.2.12 Typical Superelevation Figures

The following figures illustrate superelevation development for various conditions:

1. Figure 6.2C "Axis of Rotation about Centerline (Two-Lane Roadway)."
2. Figure 6.2D "Axis of Rotation About Edge of Travelway."
3. Figure 6.2E "Axis of Rotation About Centerline (Four-Lane Undivided Highway)." This figure illustrates the method of attaining superelevation where the cross slope of an outside travel lane is steeper than the cross slope of an adjacent travel lane. See the discussion in Section 6.2.5; Section 6.2.13 presents an example calculation.
4. Figure 6.2F "Axis of Rotation About Centerline (Compound Curves)." The figure illustrates two Cases. See Section 6.2.9 for a discussion.



① $L = L_2 \times C > L_{min}$. See Section 6.2 for superelevation runoff calculations. Transition rate, $K = S/L$.

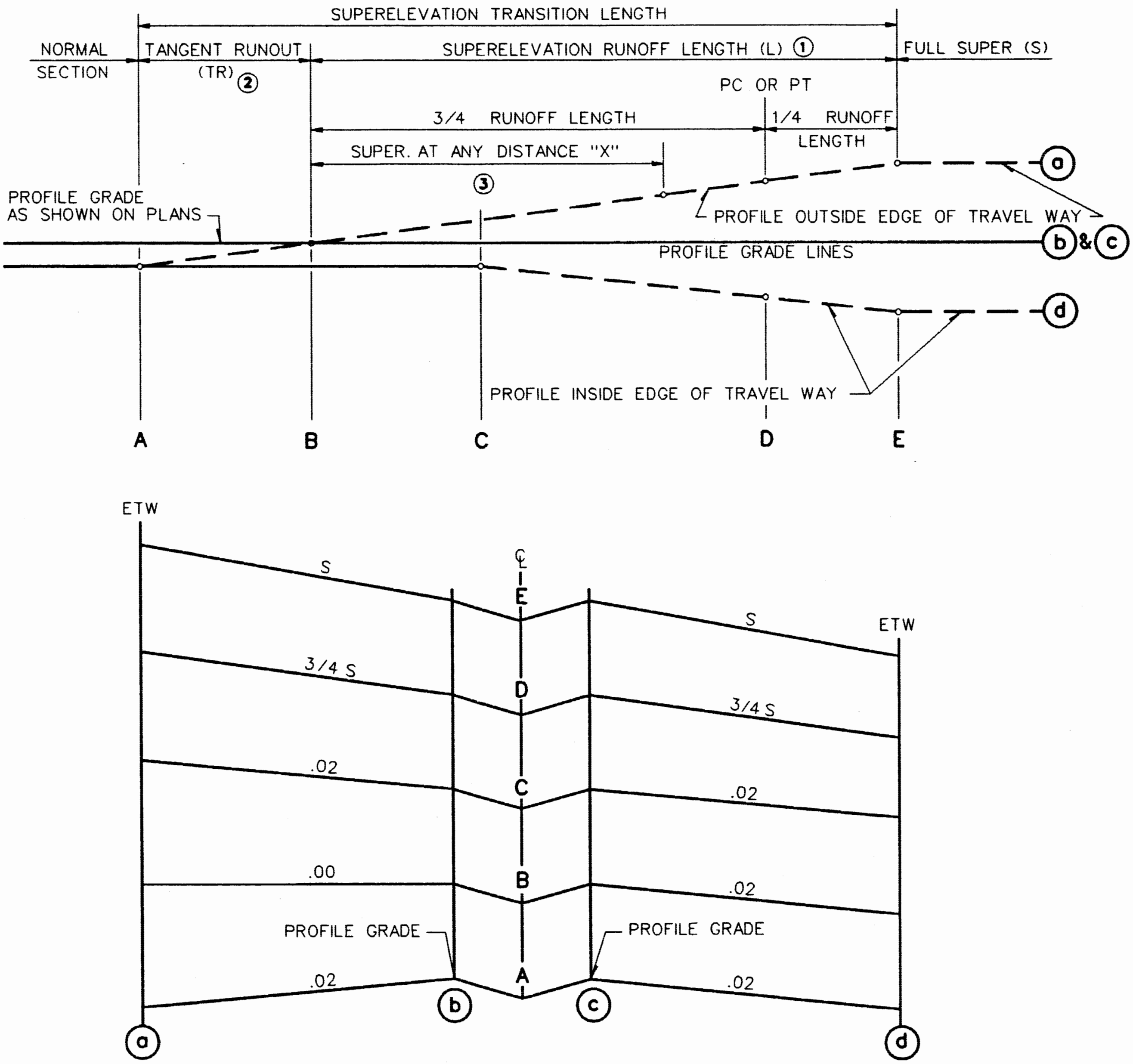
② $TR = S(normal)/K$. See Section 6.2 for tangent runout calculations.

③ $S = K \times X$. See Section 6.2 for more information.

④ Grade difference $G(\%) = 1/RS \times 100$. See Section 6.2 for values of RS.

AXIS OF ROTATION ABOUT CENTERLINE
(Two-Lane Roadway)

Figure 6.2C



① $L = L_2 \times C > L_{min}$. See Section 6.2 for superelevation runoff calculations. Transition rate, $K = S/L$.

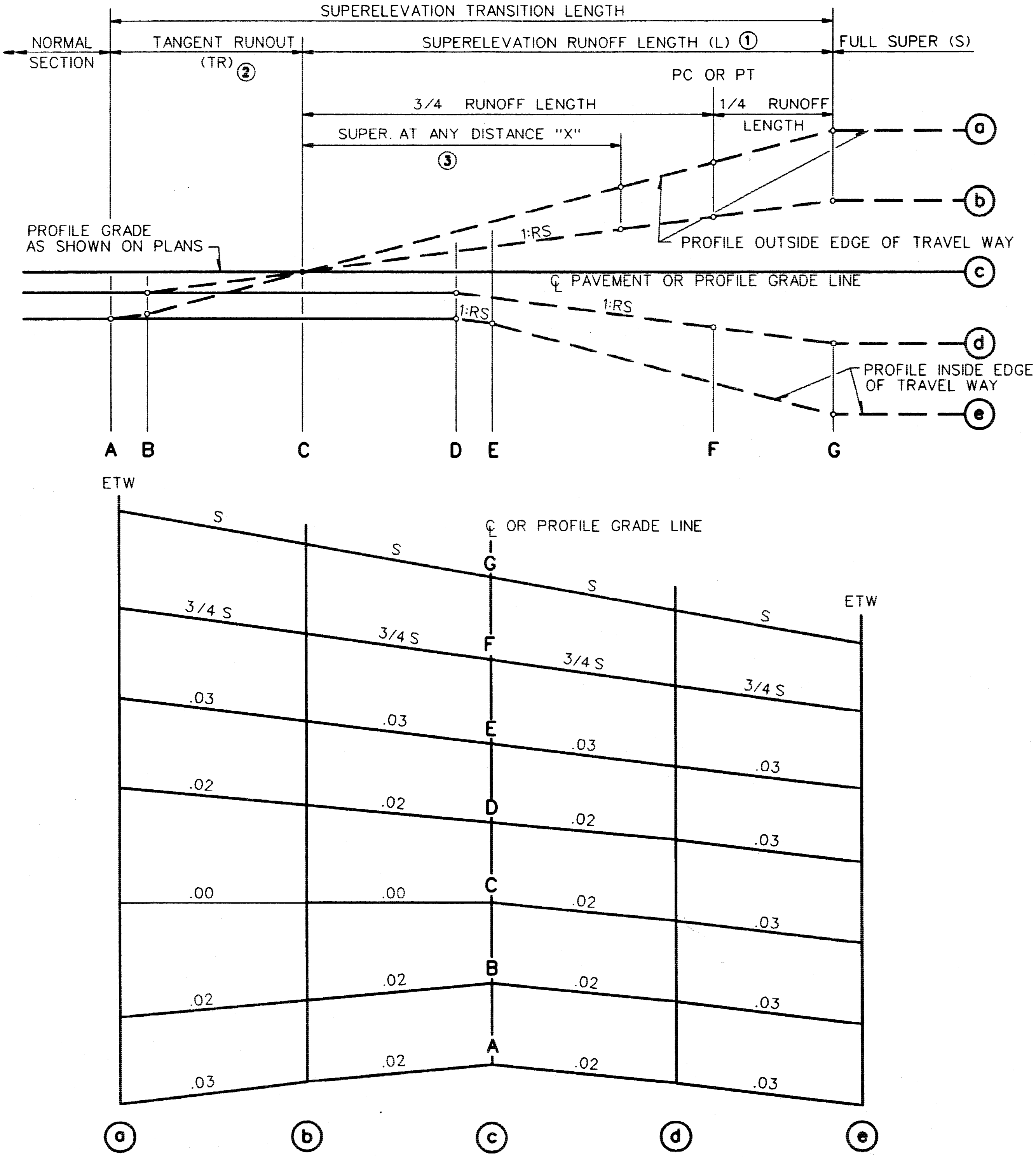
② $TR = S(normal)/K$. See Section 6.2 for tangent runout calculations.

③ $S = K \times X$. See Section 6.2 for more information.

④ Grade difference $G(\%) = 1/RS \times 100$. See Section 6.2 for values of RS.

AXIS OF ROTATION ABOUT EDGE OF TRAVELWAY

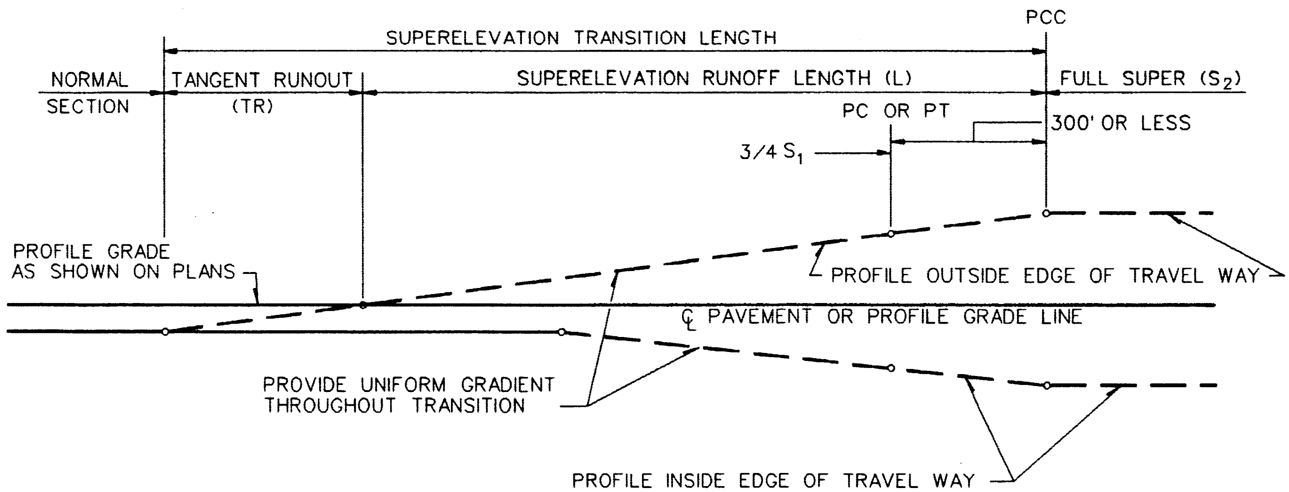
Figure 6.2D



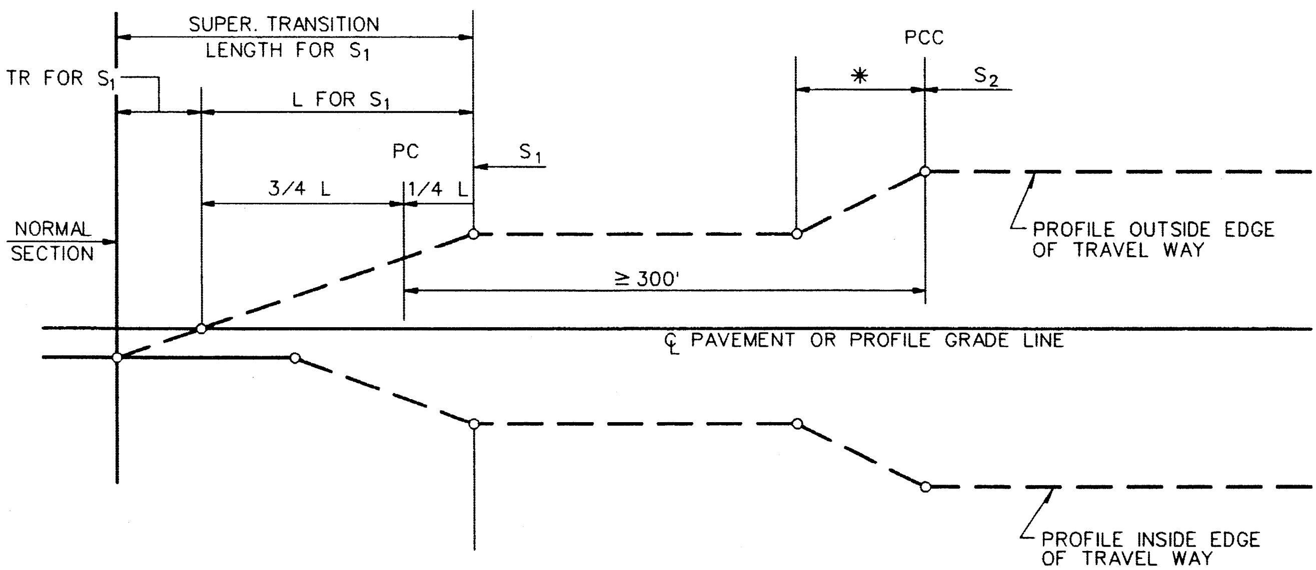
① ② ③ See Figures 6.2C and 6.2D for applicable notes.

AXIS OF ROTATION ABOUT CENTERLINE
(Four-Lane Undivided Highway)

Figure 6.2E



CASE I (a)



CASE II (b)

* This distance may be determined by application of RS for the first curve to the increase in superelevation for the second curve (S_2 - S_1).

Note: See Section 6.2.9 for a discussion on compound curves.

AXIS OF ROTATION ABOUT CENTERLINE
(Compound Curves)

Figure 6.2F

6.2.13 Examples

The following examples illustrate the application of the superelevation development criteria in Section 6.2 to specific site conditions. In all of the examples, a negative cross slope or superelevation rate slopes down from left to right, and a positive cross slope or superelevation rate slopes up from left to right.

Example 6-4 (Two-Lane Roadway)

Given: Facility — Two-lane rural arterial
Travel lane cross slope = 2% (on tangent) = 0.020 ft/ft = S_{normal}
Shoulder cross slope = 4% (on tangent) = 0.040 ft/ft
Lane width = 12 ft
Shoulder width = 8 ft
Design Speed = 60 mph
D = 3°
PC = Station 10+00
PT = Station 16+00

Problem: With the axis of rotation about the pavement centerline, determine the applicable details for superelevation development for the horizontal curve, including:

1. e_{max}
2. superelevation rate, S
3. superelevation runoff length, L
4. tangent runout length, TR
6. shoulder treatment, and
7. relative longitudinal slopes for the two outside edges of travelway.

Solution: The details of the superelevated curve are determined as follows, and Figure 6.2G presents the completed example and shows all stationing.

1. Determine e_{max}. As discussed in Section 6.2.3, e_{max} = 0.08 for rural highways and open-roadway conditions.

2. Determine Superelevation Rate (S). From Table 6.2A, S = 0.068 ft/ft for D = 3° and V = 60 mph.

3. Determine Superelevation Runoff Length (L). Runoff length, L, is equal to the greater of (L_2 x C) or L_{min}. From Table 6.2A, L_2 = 182 ft. From Table 6.2H, C = 1.0. L_2 x C = 182.0 ft x 1.0 = 182.0 ft. From Table 6.2G, L_{min} = 175.0 ft. (L_2 x C) > L_{min}. Therefore, use L = 182.0 ft.

4. Determine Transition Rate (K). K = S/L = 0.068 ft/ft / 182 ft = 0.000374 ft/ft/ft.

5. Determine Tangent Runout Length (TR). TR = S_{normal}/K = 0.020 ft/ft / 0.000374 ft/ft/ft = 53.53 ft.

6. Determine Shoulder Treatment. Desirably, the maximum shoulder breakover on the outside of curve should not exceed 7%. Therefore, with a shoulder cross slope of 4% on tangent, begin rotating the outside shoulder where the travel lanes reach a superelevation rate of -0.030 ft/ft. To determine the station where S = -0.030 ft/ft, use the equation from Figure 6.2C for superelevation at any distance "X" from point B:

   S = K *x* X
   0.030 ft/ft = 0.000374 ft/ft/ft/ *x* X
   X = 80.30 ft

   The station at point B is the PC Sta. minus ¾(L) = Station 10+00 - ¾(182.0 ft) = 8+63.50 (see Figure 6.2G). Therefore, S = -0.030 ft/ft at Station 8+63.50 + 80.30 ft = Station 9+43.80.

   A similar calculation yields S = -0.03 ft/ft at Station 17+36.50 - 80.30 ft = Station 16+56.20. Station 17+36.50 was calculated using the following equation: PT Sta. + ¾(L) = Station 16+00 + ¾(182.0 ft) = 17+36.50.

   The shoulder cross slope when the curve is fully superelevated is equal to 0.07 ft/ft - 0.068 ft/ft = 0.002 ft/ft. As stated in Section 6.2.8, the minimum continuous shoulder cross slope should be 1%. To meet the 7% maximum breakover criteria and the minimum shoulder cross slope criteria, the outside shoulder should continue to rotate until its cross slope is -1% (i.e., the shoulder slopes toward the travel lane at a rate of 1%). The rate of shoulder transition is the same as that used for the travelway (K = 0.000374 ft/ft/ft). Following is a summary of shoulder rotation:

   a. Begin shoulder rotation at Station 9+43.80, S = 0.04 ft/ft (calculated above).

   b. The shoulder will continue to rotate until S = -0.01 ft/ft. The station where S = 0.01 ft/ft is calculated using the following equations:

      Station where S = -0.01 ft/ft
      = Station where S = 0.04 ft/ft + (change in shoulder cross slope/K)
      = Station 9+43.80 + ((0.040 ft/ft - (-0.010 ft/ft))/0.000374 ft/ft/ft)
      = Station 10+77.63

      End shoulder rotation at Station 10+77.63, S = -0.010 ft/ft.

   c. The shoulder cross slope will remain at -0.010 ft/ft until Station 15+22.37. This station is calculated for the exiting portion of the curve as follows:

      Station where S = -0.010 ft/ft
      = Station where S = 0.040 ft/ft - (change in shoulder cross slope/K)

= Station 16+56.20 - ((0.040 ft/ft - (-0.010 ft/ft))/0.000374 ft/ft/ft)
= Station 15+22.37

Begin shoulder rotation at Station 15+22.37, S = -0.010 ft/ft.

d. End shoulder rotation at Station 16+56.20, S = 0.040 ft/ft (calculated above).

See Figure 6.2G.

NOTE: As stated in Section 6.2.8, a maximum breakover of 8% is acceptable. If 8% is used above, the shoulder rotation could end where S = +0.010 ft/ft (i.e., the shoulder slopes away from the travel lane).

7. Determine Relative Longitudinal Slopes (RS). The RS values are calculated using the following equation:

$$RS = \frac{Distance\ from\ Pt.\ X\ to\ Pt.\ Y}{Elev.\ @\ X\ -\ Elev.\ @\ Y}$$

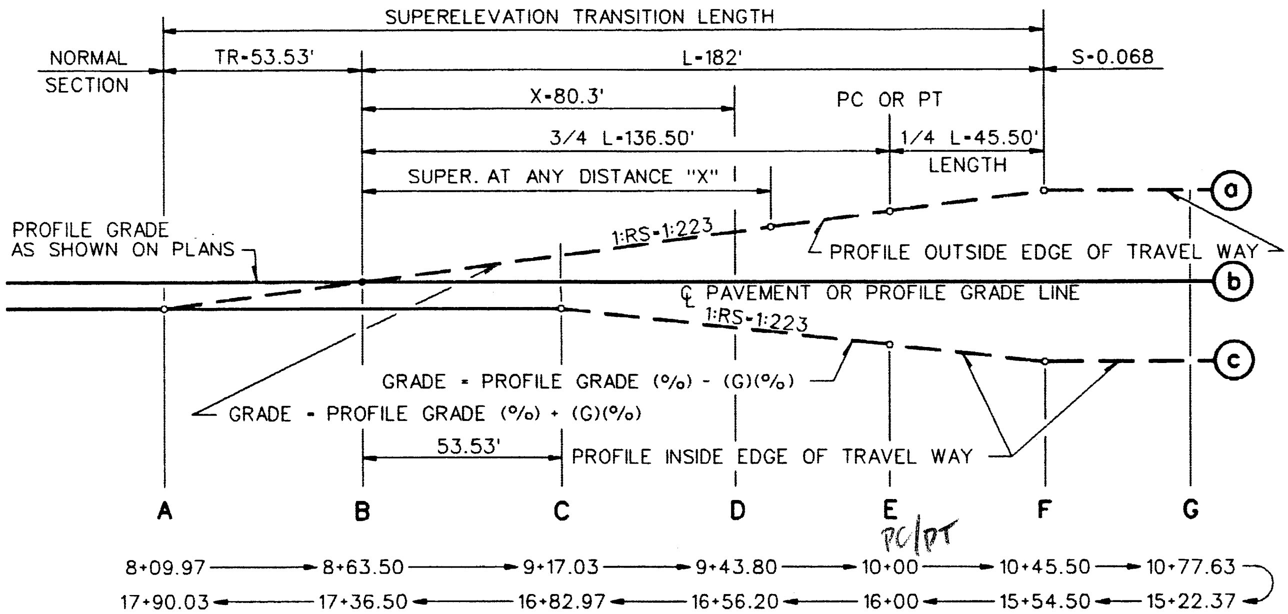
and Elevation of profile gradeline = 0.0.

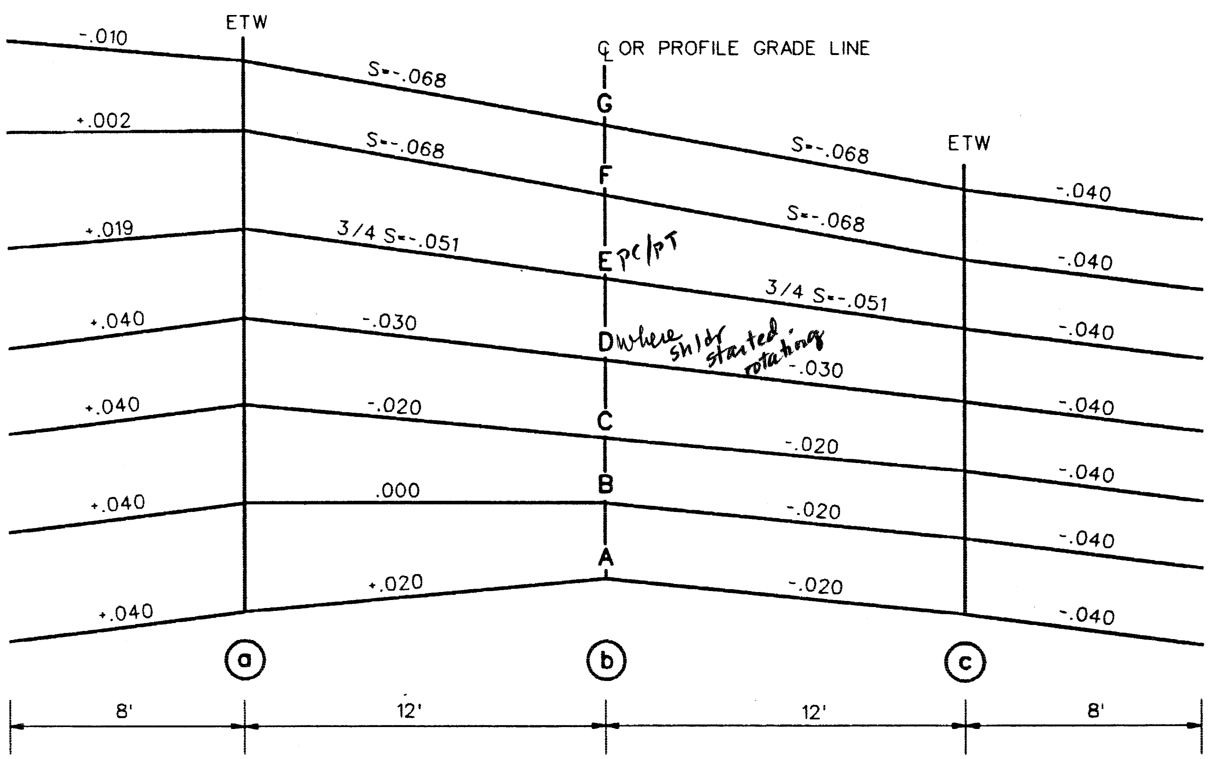
a. RS for Edge of Travelway at "a" from Point A to F:

$$RS = \frac{182.0\ ft - 53.53\ ft}{(0.020\ ft/ft)(12\ ft) - (-0.068\ ft/ft)(12\ ft)} = 223.04$$

b. RS for Edge of Travelway at "c" from Point C to F:

$$RS = \frac{182.0\ ft - 53.53\ ft}{(0.020\ ft/ft)(12\ ft) - (-0.068\ ft/ft)(12\ ft)} = 223.04$$





AXIS OF ROTATION ABOUT CENTERLINE
(Example 6-4)

Figure 6.2G

Example 6-5 (Four-Lane Divided Highway)

Given:
Facility — Four-lane divided rural arterial
Travel lane cross slope = 2% (on tangent) = 0.02 ft/ft = S_{normal}
Shoulder cross slope = 4% (on tangent) = 0.04 ft/ft
Pavement width = 24 ft
Inside shoulder width = 4 ft
Outside shoulder width = 10 ft
Median width = 40 ft
Design speed = 70 mph
D = 2°
PC = Station 10+00
PT = Station 20+00

Problem: With the axis of rotation about the median edge, determine the applicable details for superelevation development for the horizontal curve, including:

1. e_{max}
2. superelevation rate, S
3. superelevation runoff length, L
4. transition rate, K
5. tangent runout length, TR
6. shoulder treatment, and
7. relative longitudinal slopes for the two outside edges of travelway.

Solution: The details of the superelevated curve are determined as follows, and Figure 6.2H presents the completed example and shows all stationing.

1. Determine e_{max}. As discussed in Section 6.2.3, e_{max} = 0.08 for rural highways and open-roadway conditions.

2. Determine Superelevation Rate (S). From Table 6.2A, S = 0.065 ft/ft for D = 2° and V = 70 mph.

3. Determine Superelevation Runoff Length (L).
   Runoff length is equal to the greater of (L_2 x C) or L_{min}
   From Table 6.2A, L_2 = 195.0 ft
   For a divided highway with a median ≥ 40 ft, treat each direction as a separate roadway.
   From Table 6.2H, C = 1.0
   L_2 x C = 195.0 ft x 1.0 = 195.0 ft
   From Table 6.2G, L_{min} = 200.0 ft
   (L_2 x C) < L_{min}
   Therefore, L = 200.0 ft

4. Determine Transition Rate (K). K = S/L = 0.065 ft/ft / 200.0 ft = 0.000325 ft/ft/ft.

5. Determine Tangent Runout Length (TR). TR = S_{normal}/K = 0.020 ft/ft /0.000325 ft/ft/ft = 61.54 ft.

6. Determine Shoulder Treatment. Desirably, the maximum shoulder breakover on the outside of curve should not exceed 7%. Therefore, with a shoulder cross slope of 4% on tangent, begin rotating the outside shoulder where the travel lanes reach a superelevation rate of -0.030 ft/ft. To determine the station where S = 0.030 ft/ft, use the equation from Figure 6.2C for superelevation at any distance "X" from point B:

   S = K *x* X
   0.030 ft/ft = 0.000325 ft/ft/ft *x* X
   X = 92.31 ft

   The station at point B is PC Sta. minus ¾(L) = Station 10+00 - ¾(200.0 ft) = Station 8+50.00 (see Figure 6.2H). Therefore, S = -0.030 ft/ft at Station 8+50.00 + 92.31 ft = Station 9+42.31.

   A similar calculation yields S = -0.030 ft/ft at Station 21+50.00 - 92.31 ft = Station 20+57.69. Station 21+50.00 is calculated using the equation: PT Sta. + ¾(L) = Station 20+00 + ¾(200).

   The shoulder cross slope when the curve is fully superelevated is equal to 0.07 ft/ft - 0.065 ft/ft = 0.005 ft/ft. As stated in Section 6.2.8, the minimum continuous shoulder cross slope should be 1%. To meet the 7% maximum breakover criteria and the minimum shoulder cross slope criteria, the outside shoulder should continue to rotate until its cross slope is -1% (i.e., the shoulder slopes toward the travel lane at a rate of 1%). The rate of shoulder transition is the same as that used for the travelway (K = 0.000325 ft/ft/ft). Following is a summary of shoulder rotation:

   a. Begin shoulder rotation at Station 9+42.31, S = 0.040 ft/ft (calculated above).

   b. The shoulder will continue to rotate until S = -0.010 ft/ft. The station where S = -0.010 ft/ft is calculated using the following equations:

      Station where S = -0.010 ft/ft
      = Station where S = 0.040 ft/ft + (change in shoulder cross slope/K)
      = Station 9+42.31 + ((0.040 ft/ft - (-0.010 ft/ft))/0.000325 ft/ft/ft)
      = Station 10+96.16

      End shoulder rotation at Station 10+96.16, S = -0.010 ft/ft.

   c. The shoulder cross slope will remain at -0.010 ft/ft until Station 19+03.84. This station is calculated for the exiting portion of the curve as follows:

      Station where S = -0.010 ft/ft
      = Station where S = 0.040 ft/ft - (change in shoulder cross slope/K)

= Station 20+57.69 - ((0.040 ft/ft - (-0.010 ft/ft))/0.000325 ft/ft/ft)
= Station 19+03.84

Begin shoulder rotation at Station 19+03.84, S = -0.010 ft/ft.

d. End shoulder rotation at Station 20+57.69, S = 0.040 ft/ft (calculated above).

See Figure 6.2H.

NOTE: As stated in Section 6.2.8, a maximum breakover of 8% is acceptable. If 8% is used above, the shoulder rotation should end where S = +0.01 ft/ft (i.e., the shoulder slopes away from the travel lane).

7. <u>Determine Relative Longitudinal Slopes (RS)</u>. The RS values are calculated using the following equation:

$$RS = \frac{\textit{Distance from Pt. X to Pt. Y}}{\textit{Elev. @ X - Elev. @ Y}}$$

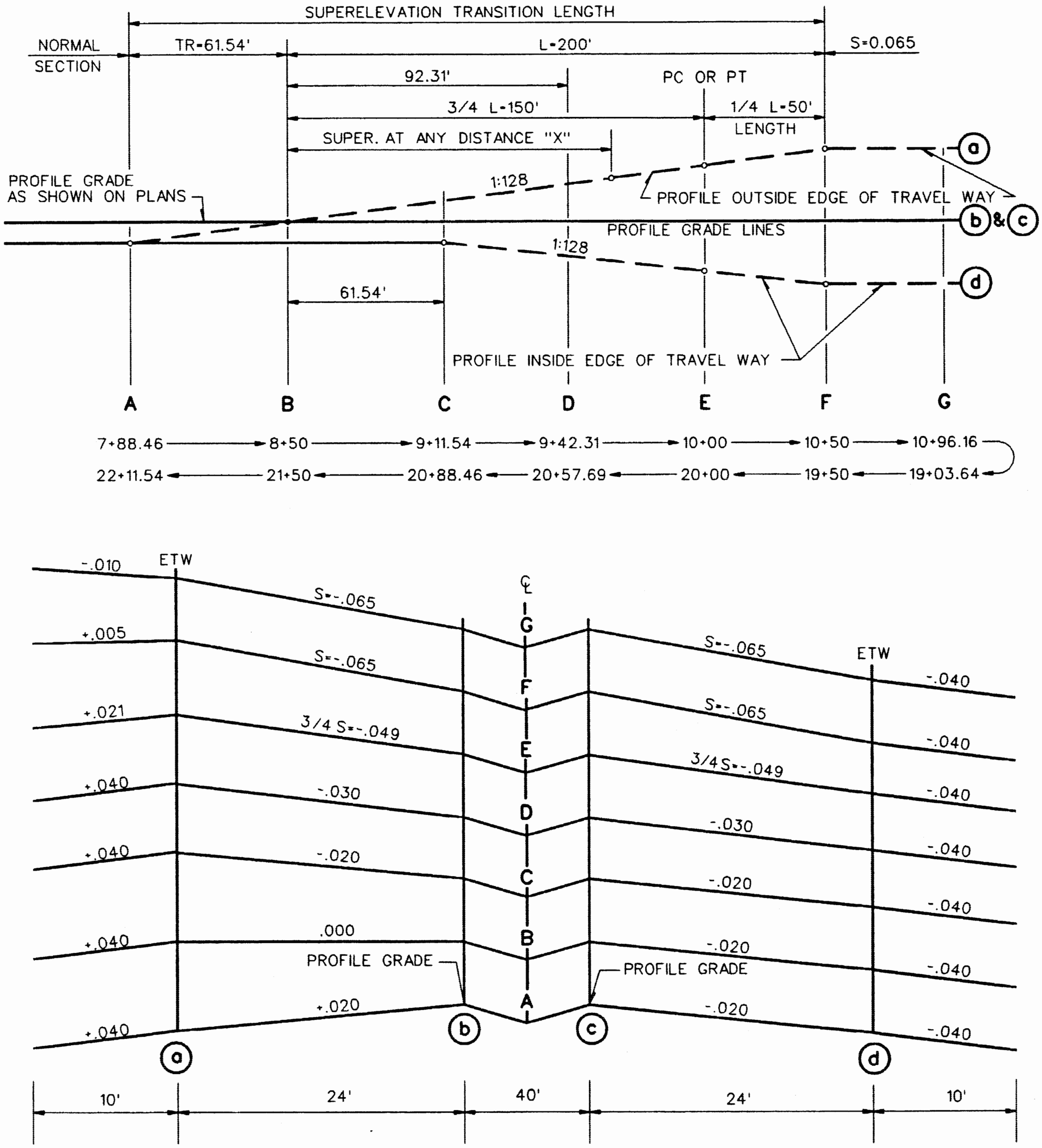
and Elevation of profile gradeline = 0.0.

a. RS for Edge of Travelway at "a" from Point A to F:

$$RS = \frac{200.0\ ft + 61.54\ ft}{(0.020\ ft/ft)(24.0\ ft) - (-0.065\ ft/ft)(24.0\ ft)} = 128.21$$

b. RS for Edge of Travelway at "d" from Point C to F:

$$RS = \frac{200.0\ ft - 61.54\ ft}{(0.020\ ft/ft)(24.0\ ft) - (-0.065\ ft/ft)(24.0\ ft)} = 128.21$$



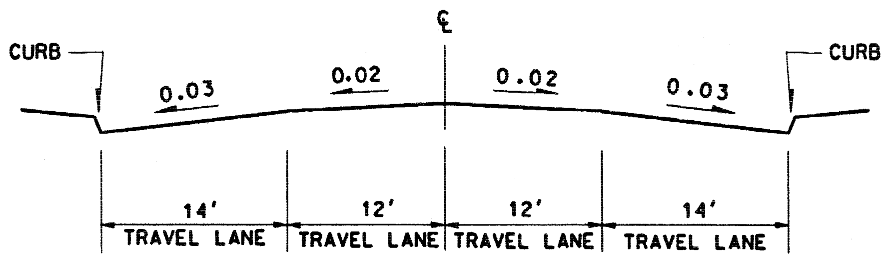
AXIS OF ROTATION ABOUT EDGE OF TRAVELWAY
(Example 6-5)

Figure 6.2H

Example 6-6 (Four-Lane Undivided Highway)

Given: Facility — Four-lane undivided urban arterial
Design Speed = 45 mph
D = 6°
PC = Station 10+00
PT = Station 16+00

Typical section as follows:



Problem: Assuming rotation about the centerline, determine the applicable details for superelevation development for the horizontal curve, including:

1. e_{max}
2. superelevation rate, S
3. superelevation runoff length, L
4. transition rate, K
5. tangent runout length, TR, and
6. relative longitudinal slopes for all travelway edges.

Solution: The details of the superelevated curve are determined as follows, and Figure 6.2I presents the completed example and shows all stationing.

1. Determine e_{max}. As discussed in Section 6.2.3, e_{max} = 0.06 for urban highways; desirably, open-roadway conditions are used.

2. Determine Superelevation Rate (S). From Table 6.2B, S = 0.055 ft/ft for D = 6° and V = 45 mph.

3. Determine Superelevation Runoff Length (L). Runoff length, L, is equal to the greater of (L_2 x C) or L_{min}. From Table 6.2B, L_2 = 125.0 ft. From Table 6.2H, C = 2.0 (desirable). L_2 x C = 125.0 ft x 2.0. From Table 6.2G, L_{min} = 135.0 ft. (L_2 x C) > L_{min}. Therefore, use L = 250.0 ft.

4. Determine Transition Rate (K). K = S/L = 0.055 ft/ft / 250.0 ft = 0.000220 ft/ft/ft.

5. Determine Tangent Runout Length (TR).

   As indicated in the "Given" information, the roadway has a variable cross slope for the travel lanes on tangent. Therefore, as described in Section 6.2.5.2, first calculate the distance needed to change the 3% cross slope of the outside travel lane on the outside of the horizontal curve to match the 2% slope of the inside travel lane:

   TR_1 = change in cross slope/K = (0.030 ft/ft - 0.020 ft/ft)/0.000220 ft/ft/ft.
   TR_1 = 45.45 ft

   Next, calculate the distance needed to rotate the two travel lanes on the outside of the horizontal curve from a 2% cross slope to a zero cross slope (i.e., level):

   TR_2 = change in cross slope/K = (0.020 ft/ft - 0.000 ft/ft)/0.000220 ft/ft/ft.
   TR_2 = 90.91 ft

   The total tangent runout distance is:

   TR = TR_1 + TR_2 = 45.45 ft + 90.91 ft
   TR = 136.36 ft

6. Determine Relative Longitudinal Slope (RS). The RS values are calculated using the following equation:

   $$RS = \frac{Distance\ from\ Pt.\ X\ to\ Pt.\ Y}{Elev.\ @\ X - Elev.\ @\ Y}$$

   and Elevation of profile gradeline = 0.0.

   a. RS of outside travel lane at Edge "a" from Point A to Point B:

$$RS = \frac{45.45\ ft}{((12\ ft)(0.020\ ft/ft) + (14\ ft)(0.030)) - ((12\ ft)(0.020\ ft/ft) + (14\ ft)(0.020\ ft/ft))} = 324.64$$

   b. RS of outside travel lane at Edge "a" from Point B to Point G:

   $$RS = \frac{90.91\ ft + 250.0\ ft}{(0.020\ ft/ft)(26.0\ ft) - (-0.055\ ft/ft)(26.0\ ft)} = 174.8$$

   c. RS of inside travel lane at Edge "b" from Point B to Point G:

   $$RS = \frac{90.91\ ft + 250.0\ ft}{(0.020\ ft/ft)(12.0\ ft) - (-0.055\ ft/ft)(12.0\ ft)} = 378.8$$

d. RS of outside travel lane at Edge "e" from Point D to Point E:

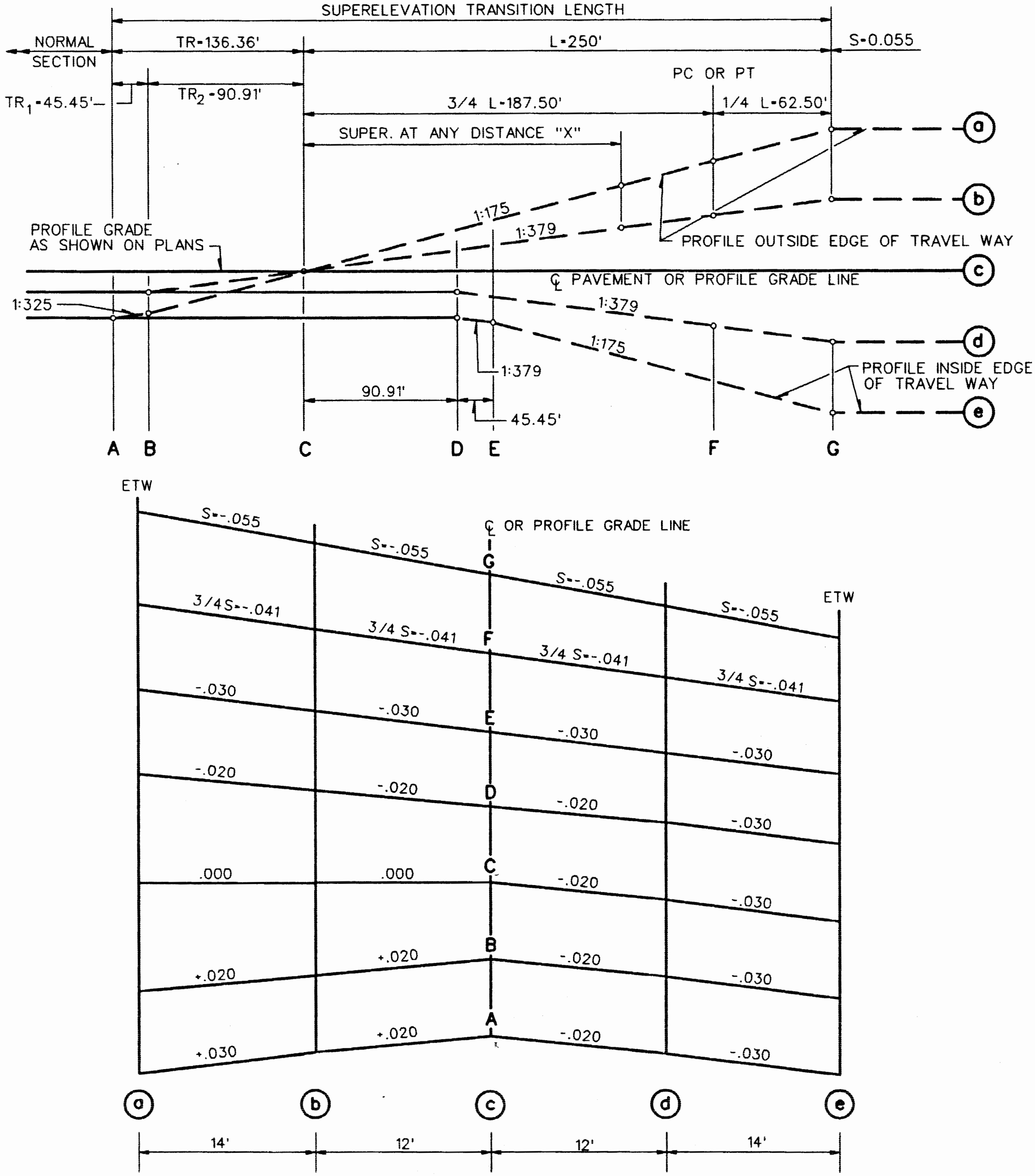
$$RS = \frac{45.45\ ft}{((12\ ft)(-0.020\ ft/ft) + (14\ ft)(-0.030\ ft/ft)) - ((12\ ft)(-0.030\ ft/ft) + (14\ ft)(-0.030\ ft/ft))} = 378.8$$

e. RS of outside travel lane at Edge "e" from Point E to Point G:

$$RS = \frac{250.0\ ft - 136.36\ ft}{(-0.030\ ft/ft)(26.0\ ft) - (-0.055\ ft/ft)(26.0\ ft)} = 174.8$$

f. RS of inside travel lane at Edge "d" from Point D to Point G:

$$RS = \frac{250.0\ ft - 90.91\ ft}{(-0.020\ ft/ft)(12.0\ ft) - (-0.055\ ft/ft)(12.0\ ft)} = 378.8$$



FOUR-LANE UNDIVIDED HIGHWAY
(Example 6-6)

Figure 6.2I

Example 6-7 (Compound Curve)

Given: Facility — Two-lane rural arterial
Travel lane cross slope = 2% on tangent
Design Speed = 60 mph
3-centered symmetrical compound curve with: $D_1 = D_3 = 2°$, $D_2 = 4°$
Distance from PC to PCC < 300 ft

Problem: Determine the superelevation details for transitioning from a normal section to the required superelevation at the PCC.

Solution: As discussed in Section 6.2.9, superelevation development on compound curves requires a process of meeting, as practical, several objectives. This example is solved using the following steps (for Case I):

1. Determine e_{max}. As discussed in Section 6.2.3, e_{max} = 0.08 for rural highways and open-roadway conditions.

2. Determine Superelevation Rates (S_1 and S_2). From Table 6.2A:

   S_1 = 0.051 ft/ft for $D_1 = 2°$ and V = 60 mph
   S_2 = 0.078 ft/ft for $D_2 = 4°$ and V = 60 mph

3. Determine Superelevation Runoff Lengths (L_1 and L_2). From Table 6.2A:

   L_1 = 136 ft for Curve #1
   L_2 = 208 ft for Curve #2

   From Table 6.2G, L_{min} = 175 ft for V = 60 mph. Therefore, the objective will be to superelevate the compound curve such that, when viewing each curve independently:

   $L_1 \geq 175$ ft
   $L_2 \geq 208$ ft

4. Determine Transition Rate (K) for Curve #1. One objective is to develop Curve #1 as if it is an independent simple curve. This yields for the transition rate:

   $K_1 = S_1/L_1$ = 0.051 ft/ft /175 ft = 0.000291 ft/ft/ft

5. Determine L_2. One objective is to provide a uniform longitudinal gradient throughout the superelevation runoff from the end of the tangent runout to S_2 at the PCC. Using the transition rate for Curve #1:

   $L_2 = S_2/K_1$ = 0.078 ft/ft /0.000291 ft/ft/ft
   L_2 = 268 ft

This distance is greater than L_2 would be if Curve #2 is developed as an independent simple curve. Therefore, using a total superelevation runoff of 268 ft is an acceptable treatment for superelevation development on the compound curve.

The superelevation runoff length (L = 268 ft) must be distributed between the tangent and Curve #1 (i.e., up to the PCC). One objective for compound curves is to provide ¾ S_1, at the PC. Therefore, the distance between the end of the tangent runout and the PC is calculated as follows:

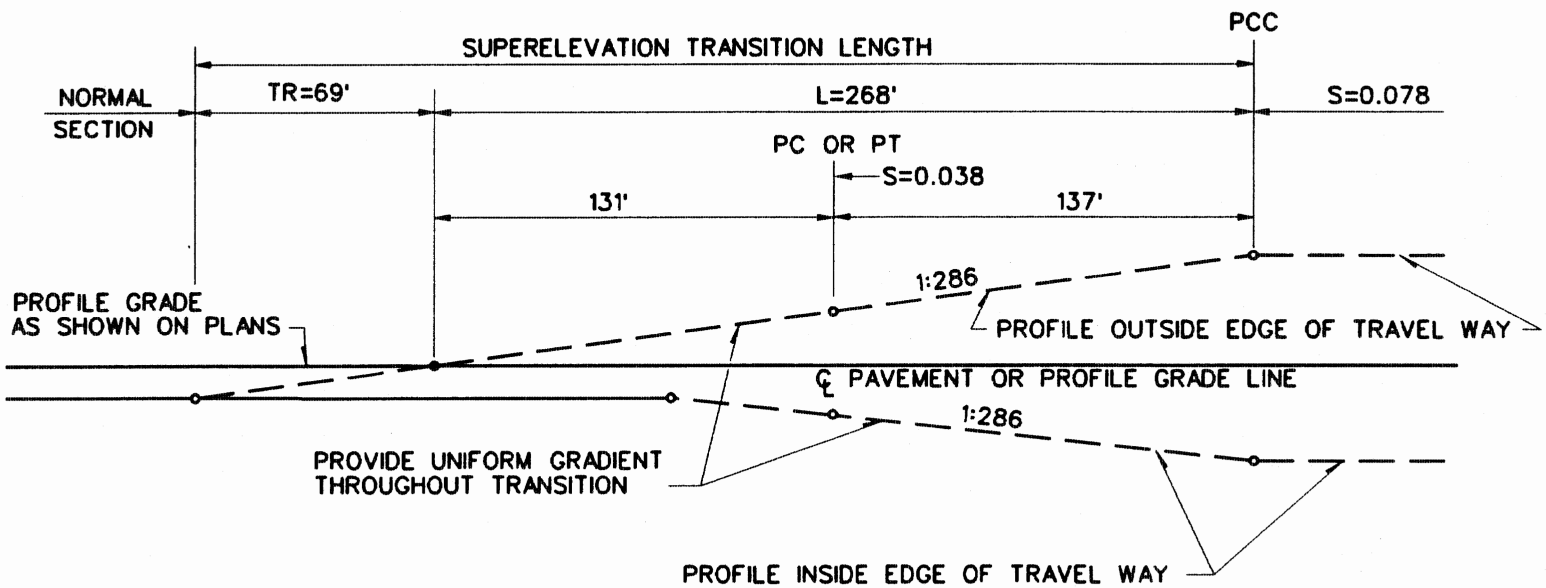
$L_{PC} = S_{PC}/K = (¾)(0.051 \text{ ft/ft}) / 0.000291 \text{ ft/ft/ft}$
$L_{PC} = 131.44 \text{ ft}$

The remainder of the runoff length (137 ft) is provided between the PC and PCC. Note that this distance is less than 300 ft, which is the guideline for using Case I for superelevation development on compound curves.

6. Determine Tangent Runout Length (TR). Section 6.2.5 describes how to calculate TR:

$TR = S_{normal}/K = 0.02 \text{ ft/ft} / 0.000291 \text{ ft/ft/ft}$
$TR = 69 \text{ ft}$

Figure 6.2J presents the completed Example 6-7.



SUPERELEVATION DEVELOPMENT ON COMPOUND CURVES
(Example 6-7, Case I)

Figure 6.2J

6.3 DESIGN PRINCIPLES AND PROCEDURES

6.3.1 General Controls

As discussed elsewhere in Chapter Six, the design of horizontal alignment involves, to a large extent, complying with specific limiting criteria. These include maximum degree of curvature, superelevation rates and sight distance around curves. In addition, the designer should adhere to certain design principles and controls which will determine the overall safety of the facility and will enhance the aesthetic appearance of the highway. These design principles include:

1. Consistency. Alignment should be consistent. Sharp curves at the ends of long tangents and sudden changes from gentle to sharply curving alignment should be avoided.

2. Directional. Alignment should be as directional as possible consistent with physical and economic constraints. On divided highways a flowing line that conforms generally to the natural contours is preferable to one with long tangents that slash through the terrain. Directional alignment will be achieved by using the smallest practical central angles.

3. Use of Maximum Curvature. The use of maximum curvature should be avoided if practical.

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

5. Alignment Reversals. Avoid abrupt reversals in alignment (S or reverse curves). Provide a sufficient tangent distance between the curves to ensure proper superelevation transitions for both curves. (See Chapter Fourteen for a discussion of horizontal alignment within detours.)

6. Broken-Back Curvature. Avoid where possible. This arrangement is not aesthetically pleasing, violates driver expectancy and creates undesirable superelevation development requirements. (See Chapter Thirteen for a discussion on existing highways.)

7. Coordination with Natural/Man-Made Features. The horizontal alignment should be properly coordinated with the natural topography, available right-of-way, utilities, roadside development and natural/man-made drainage patterns.

8. Environmental Impacts. Horizontal alignment should be properly coordinated with environmental impacts (e.g., encroachment onto wetlands). The Planning Division's Transportation Planning Branch is responsible for evaluating environmental impacts.

9. Intersections. Horizontal alignment through intersections may present special problems (e.g., intersection sight distance, superelevation development). See Chapter Nine for the design of intersections at-grade.

10. Coordination with Vertical Alignment. Chapter Seven discusses design principles for the coordination between horizontal and vertical alignment.

11. Bridges. Horizontal alignment must be coordinated with bridges. Curvature and superelevation development should be evaluated for each bridge location. Crossing angles between the mainline and roadway, railway or waterways should desirably not exceed 60°. In extreme conditions, crossing angles for waterways

may be 45° and intersecting roadways or railways may be 30°.

6.3.2 Design Procedures

6.3.2.1 General

In the design of horizontal alignment, the designer should be aware of his responsibility to communicate properly with other ODOT personnel (e.g., drafting, field survey). The following provides examples of elements of horizontal alignment which the designer should reflect in his design:

1. Preparation of Plans. Chapter Four "Plan Development" discusses the content and format of the plan sheets, abbreviations, symbols, scales and the use of ODOT's CADD system. The designer must ensure that the presentation of the horizontal alignment is consistent with ODOT practices.

2. Equations in Stationing. Departures from the line surveyed in the field may be necessary and 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.

3. Survey Lines (Divided Highways). Where the median width is less than 40 ft and remains relatively constant, the stationing and all other alignment computations are typically based on a single survey base 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.

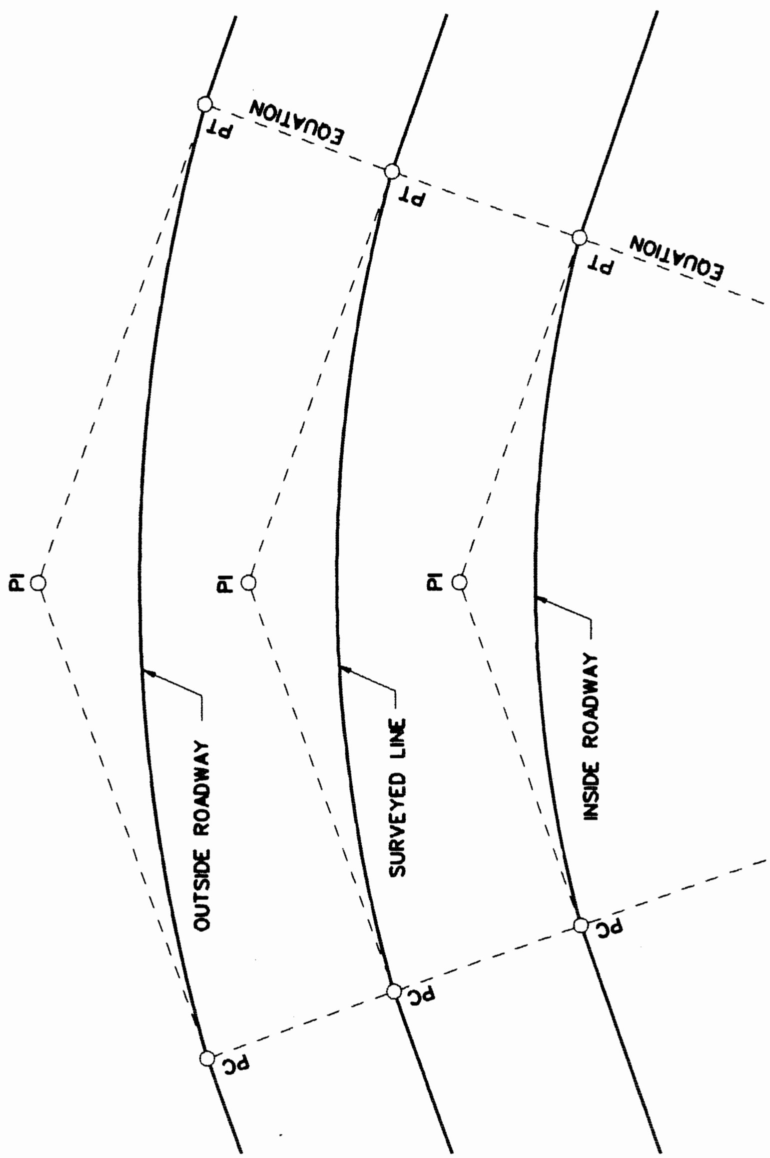
   When the median width on a divided highway exceeds 40 ft, each roadway will typically have separate horizontal and vertical controls independent of a single survey base line. Each roadway will have its own separate profile grade and horizontal curve data.

4. Curve Orientation on Divided Highways. Where independent alignment is not used, the typical practice for horizontal curvature on divided highways is the use of concentric curves (common center of radii). 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 and L will vary for each line. Equations in stationing will re-establish common stationing from the PT on both roadways as discussed in #2 above. See Figure 6.3A.

   Non-concentric curves may also be used if field conditions indicate this arrangement is advantageous.

5. Superelevation Development. The rate of superelevation should be shown on the cross section sheets. In the transition portions of the superelevated curve, superelevation data should be provided at 25-ft intervals to facilitate construction staking. These elevations and superelevation rates may be in a separate table, on the plan/profile sheets and/or on joint/spot layouts. All elevations should be the finished surface elevations.

6. Survey (General). Horizontal curves are normally staked in the field by the deflection method for simple curves. In general, the designer should ensure that the horizontal alignment is consistent with the ODOT's field survey practices. See Section Four of the ODOT's *Manual of Policies and Procedures*.



CONCENTRIC CURVES
(Divided Highways)

Figure 6.3A

6.3.2.2 Mathematical Computations

Section 6.6 presents numerous figures which provide the needed mathematical equations and techniques to make various computations for horizontal curves.

6.4 TRAVELWAY WIDENING

6.4.1 Warrants

Wherever practical, widening of the traveled way should be considered on the inside edge of horizontal curves to make operating conditions on the curves comparable to those on tangent. Travelway widening will typically be considered only on two-lane, two-way facilities. This feature may be especially applicable to 3R non-freeway projects (see Chapter Thirteen).

The designer should evaluate the need for travelway widening on a case-by-case basis considering the functional class, type of shoulder, traffic volumes, truck volumes and urban/rural location. In general, widening is more beneficial on lower-class facilities because of the typically narrower roadway width and the presence of unpaved shoulders. Travelway widening may also be warranted in known maintenance problem areas (e.g., where the inside shoulder has broken up because of traffic).

To determine warrants, two tables are presented for widths of travelway widening with application as follows:

1. High Truck Volumes. Table 6.4A presents travelway widening design criteria for facilities with high truck volumes. These values should be used according to the following guidelines:

    a. For truck AADT > 500, Table 6.4A will typically apply.

    b. For truck AADT < 200, Table 6.4A will typically not apply.

    c. For truck AADT between 200 and 500, the use of Table 6.4A will be determined on a case-by-case basis.

    d. Regardless of the truck AADT, the use of Table 6.4A should be considered where T>20%.

    On all State highways, the paved width (travel lane plus shoulder) should meet the criteria in Table 6.4A regardless of truck volumes.

2. All Other Facilities. Table 6.4B applies to all facilities where Table 6.4A does not apply.

6.4.2 Design

Widening should be applied to the inside edge of pavement only. Desirably, the transition distance for travelway widening will equal the superelevation runoff length, and it will be applied coincident with the superelevation runoff. At a minimum, the widening will occur over a distance of 100-200 ft.

Figure 6.4A illustrates a tangent line to achieve the travelway widening. This is the simplest method. The designer may insert short horizontal curves at the angle breaks to improve its appearance.

Table 6.4A

TRAVELWAY WIDENING ON HIGHWAY CURVES
(High Truck Volumes)

Degree of Curve (D)	Travelway = 24′				
	Design Speed (mph)				
	30	40	50	60	70
1°	0.0	0.0	0.0	0.0	0.0
2°	0.5	0.5	0.5	1.0	1.0
3°	1.0	1.5	1.5	2.0	2.0
4°	1.5	2.0	2.0	2.5	2.5
5°	2.0	2.5	3.0	3.0	
6°	3.0	3.0	3.5		
7°	3.5	4.0	4.0		
8°	4.0	4.5	5.0		
9°	4.5	5.0			
10°	5.5	5.5			
15°	7.5	8.0			
20°	10.0				
25°	12.5				

Notes:

1. Application. Use this table where a sufficient number of trucks use the facility to govern design. See discussion in Section 6.4.1.

2. Other Travelway Widths. Table applies directly to 24′ travelways on tangent. For 22′ travelways, increase table values by 1.0′; for 20′ travelways, increase by 2.0′.

3. Number of Lanes. Table applies to two-lane, two-way facilities.

4. Minimum Width of Widening. On all highways, the minimum width of widening should be 2.0′. Widening less than 2.0′ will not be required.

5. Basis. Values in table were calculated assuming a WB-67 design vehicle.

Table 6.4B

TRAVELWAY WIDENING ON HIGHWAY CURVES
(All Other Facilities)

Degree of Curve (D)	Travelway = 24′				
	Design Speed (mph)				
	30	40	50	60	70
1°	0.0	0.0	0.0	0.0	0.0
2°	0.0	0.0	0.0	0.5	0.5
3°	0.0	0.0	0.5	0.5	1.0
4°	0.0	0.5	0.5	1.0	1.0
5°	0.5	0.5	1.0	1.0	
6°	0.5	1.0	1.0	1.5	
7°	0.5	1.0	1.5		
8°	1.0	1.0	1.5		
9°	1.0	1.5	2.0		
10°	1.0	1.5			
15°	2.0	2.0			
20°	2.5				
25°	3.0				

Notes:

1. Application. Table 6.4B applies where Table 6.4A does not apply. Where the value of the WB-50 design vehicle is sufficient to govern design, increase the widths in Table 6.4B as follows:

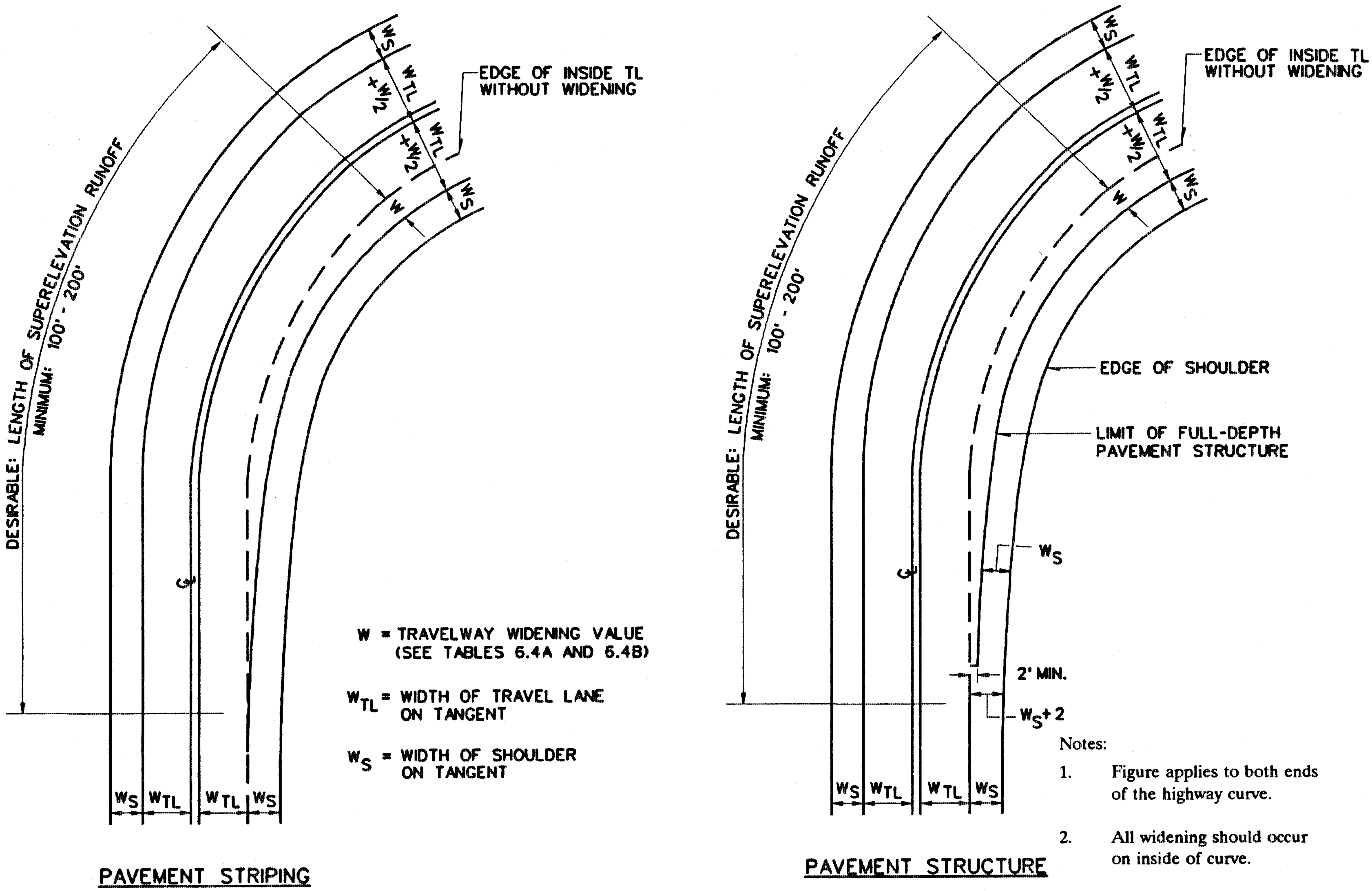
   a. D = 10°, increase by 0.5′.
   b. D ≥ 15°, increase by 1.0′.

2. Other Travelway Widths. Table applies directly to 24′ travelways on tangent. For 22′ travelways, increase table values by 1.0′; for 20′ travelways, increase by 2.0′.

3. Number of Lanes. Table applies to two-lane, two-way facilities.

4. Minimum Width of Widening. On all highways, the minimum width of widening should be 2.0′. Widening less than 2.0′ will not be required.

5. Basis. Values in table were calculated assuming a SU design vehicle.



Notes:

1. Figure applies to both ends of the highway curve.

2. All widening should occur on inside of curve.

APPLICATION OF TRAVELWAY WIDENING ON HORIZONTAL CURVES

Figure 6.4A

6.5 HORIZONTAL SIGHT DISTANCE

6.5.1 General

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, 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 on the inside of horizontal curves. 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.

6.5.2 L > S

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

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

where:

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

D = Degree of curvature

S = Sight distance, ft

Figures 6.5A and 6.5B provide the horizontal clearance criteria for various combinations of sight distance and degree of curvature. As discussed in the following sections, the designer should select a sight distance "S" which is appropriate for the given highway conditions. For those selections of S which fall outside of Figures 6.5A or 6.5B (i.e., M > 50 ft and/or D > 25°), the designer should use the basic equation to calculate the needed clearance.

6.5.2.1 Stopping Sight Distance (SSD)

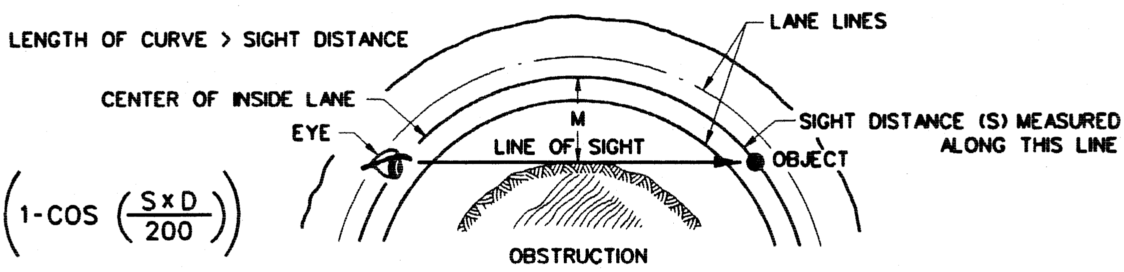
At a minimum, SSD will be available throughout the horizontal curve. The SSD values for various assumptions are presented in Section 5.7. The SSD values include:

1. Passenger Cars (Level Grade). Table 5.7A presents desirable and minimum SSD values. The minimum SSD for passenger cars on level grade represent the lowest acceptable sight distance at the horizontal curve. The designer should make every reasonable effort to provide a design which meets or exceeds the desirable SSD values.

2. Passenger Cars (Downgrade Adjustment). Table 5.7C presents minimum and desirable SSD values for passenger cars adjusted for 3% - 9% downgrades. If the downgrade on the facility is 3% or steeper, the designer should provide horizontal clearances adjusted for grade.

3. Trucks (Level Grade). Table 5.7B presents SSD values for trucks on level grades. On facilities with high truck volumes, the designer should consider providing horizontal clearances which reflect the longer braking distances needed by trucks. A high accident history with trucks may also indicate the need to consider truck SSD values.

4. Trucks (Downgrade Adjustment). Table 5.7D presents SSD values for trucks adjusted for 3% - 9% downgrades. On facilities with high truck volumes and downgrades 3% or steeper, the designer



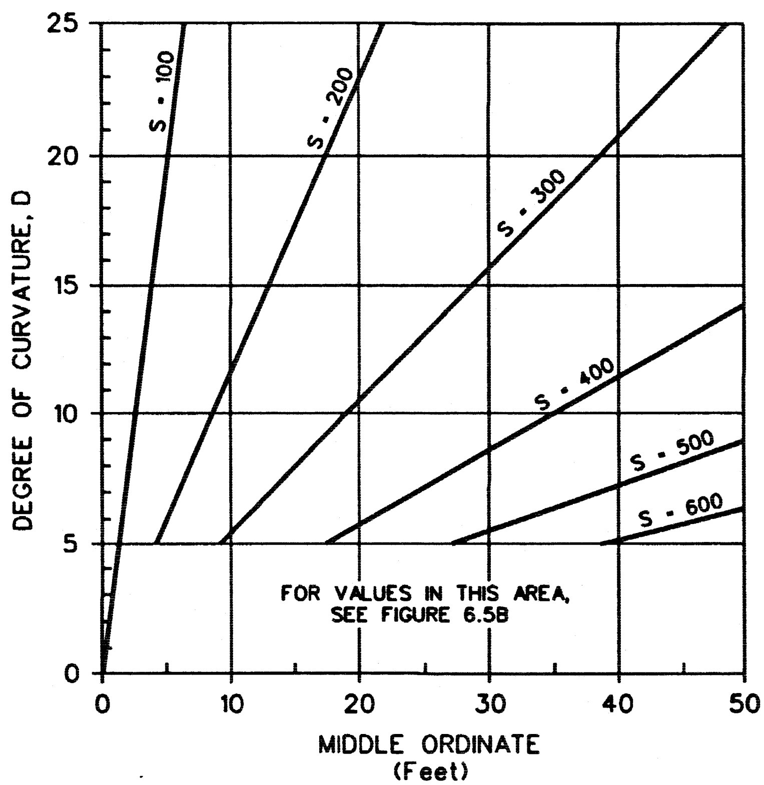
$$M = \frac{5730}{D}\left(1 - \cos\left(\frac{S \times D}{200}\right)\right)$$

WHERE:

M = MIDDLE ORDINATE, DISTANCE FROM THE LINE OF MEASURE (CENTER OF INSIDE LANE) TO THE OBSTRUCTION, FEET

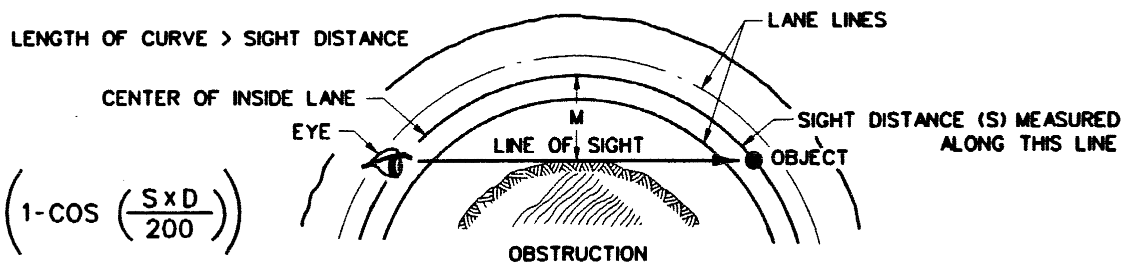
D = DEGREE OF CURVATURE

S = SIGHT DISTANCE, FEET



SIGHT DISTANCE AT HORIZONTAL CURVES
($5° \leq D \leq 25°$)

Figure 6.5A



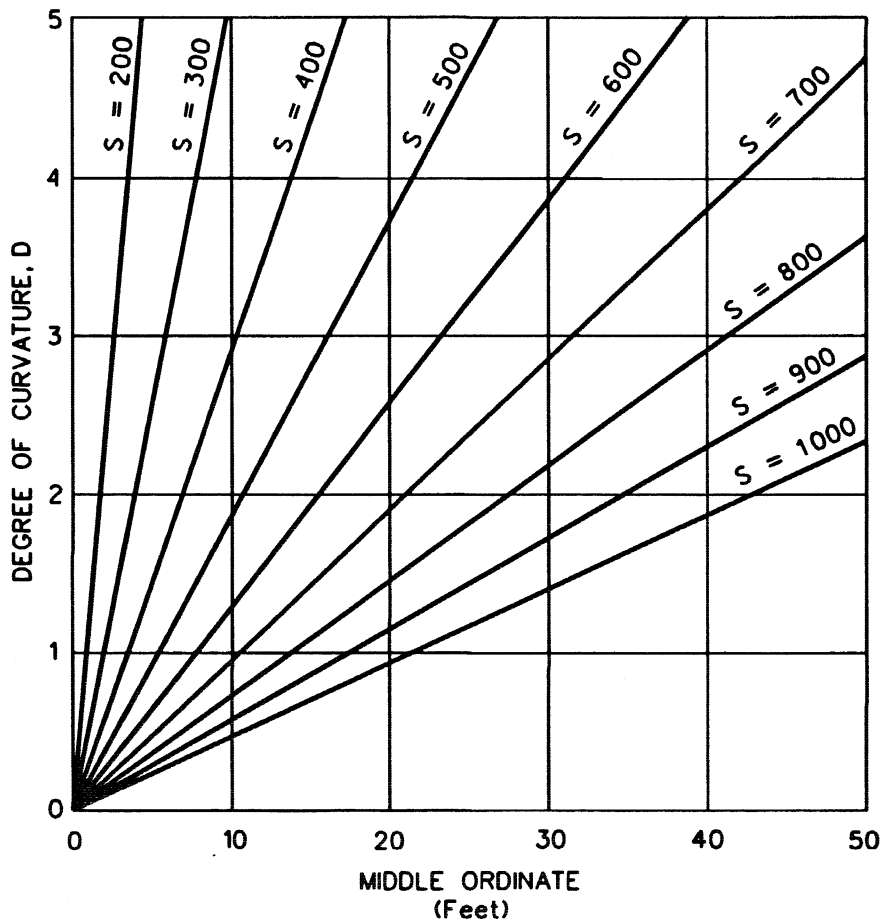
$$M = \frac{5730}{D}\left(1 - \cos\left(\frac{S \times D}{200}\right)\right)$$

WHERE:

M = MIDDLE ORDINATE, DISTANCE FROM THE LINE OF MEASURE (CENTER OF INSIDE LANE) TO THE OBSTRUCTION, FEET

D = DEGREE OF CURVATURE

S = SIGHT DISTANCE, FEET



SIGHT DISTANCE AT HORIZONTAL CURVES
(D < 5°)

Figure 6.5B

should consider providing horizontal clearances which reflect this operational condition.

The Example on Figure 6.5C illustrates the determination of clearance requirements at a horizontal curve.

6.5.2.2 Decision Sight Distance (DSD)

At some locations, it may be warranted to provide DSD at the horizontal curve. Section 5.7 discusses candidate sites and provides design values for decision sight distance. These "S" values should be used in the basic equation to calculate "M."

6.5.2.3 Entering/Exiting Portions

The M values from Figures 6.5A and 6.5B apply between the PC and PT. In addition, some transition is needed on the entering and exiting portions of the 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 S/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 Step #'s 1 and 2. The area between this line and the roadway should be clear of all continuous obstructions.

Step 4: A symmetrical application of Step #'s 1-3 should be used beyond the PT.

The Example on Figure 6.5C illustrates the determination of clearance requirements entering and exiting from a curve.

6.5.3 L < S

When the length of curve is less than the sight distance used in design, the M value from the basic equation 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 > S.

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:

$$\frac{M'}{M} = \frac{1.2\,L}{S}, \text{ and}$$

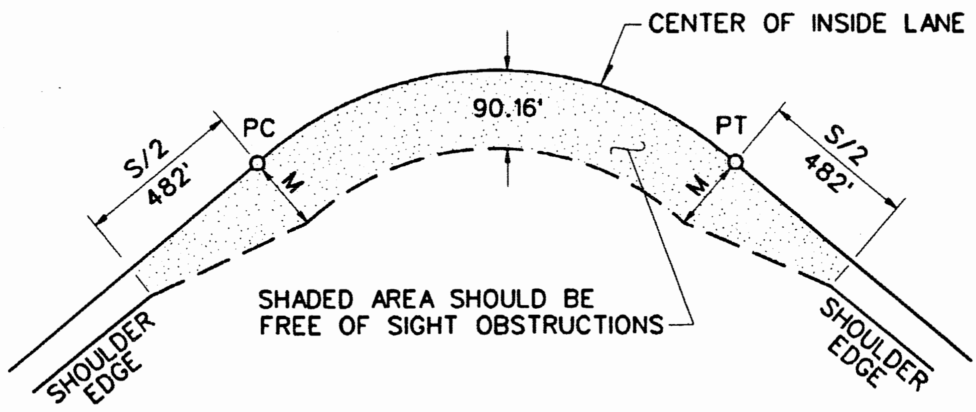
$$M' = \frac{1.2\,L \times M}{S},$$

where: $M' \leq M$

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

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

Step 5: A symmetrical application of Step #'s 2-4 should be used on the exiting portion of curve.



Example 6-8

Given: Urban Freeway
Design Speed = 60 mph
D = 4°30′, L = 1200 ft
Truck DDHV > 250 vph
Grade = 2% downgrade

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

Solution: Given the high truck volumes, the curve should desirably be designed to accommodate truck SSD values (on level grade). Table 5.7B yields a SSD of 964 ft for 60 mph. Using the equation for horizontal clearance (L > S):

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

$$M = \frac{5730}{4.5}\left(1 - \cos\frac{(964)(4.5)}{200}\right) = 90.16'$$

The above figure also illustrates the horizontal clearance requirements for the entering and exiting portion of the horizontal curve.

SIGHT CLEARANCE REQUIREMENTS FOR HORIZONTAL CURVES

Figure 6.5C

* * * * * * * * * * *

Example 6-9

Given: Design Speed = 70 mph
D = 3°00′
L = 600′
Truck DDHV < 250 vph
Grade = 5% downgrade

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

Solution: Because the downgrade is greater than 3%, the curve should desirably be designed for passenger cars adjusted for grade. Table 5.7C yields a desirable SSD of 980′ for 70 mph and a 5% downgrade. Therefore, L < S (600 ′ < 980′), and the horizontal clearance is calculated as follows:

$$M\ (L > S) = \frac{5730}{3}\left[1 - \cos\frac{(980)(3)}{200}\right]$$

$$M = 62.52'$$

$$M'\ (L < S) = \frac{1.2\ (600)(62.52)}{980}$$

$$M' = 45.93'$$

Therefore, a maximum clearance of 45.93′ should be provided at a distance of L/2 = 300′ beyond the PC. The obstruction-free triangle around the horizontal curve would be defined by M′ (45.93′) at L/2 and by points at the shoulder edge at S/2 = 490′ before the PC and beyond the PT.

* * * * * * * * * * *

6.5.4 Application

For application, the height of eye is 3.5 ft and the height of object is 0.5 ft. Both the eye and object are assumed to be in the center of the inside travel lane. The line-of-sight intercept with the obstruction is at the midpoint of the sightline and 2 ft above the center of the inside lane.

6.5.5 Longitudinal Barriers

Longitudinal barriers can cause sight distance problems at horizontal curves. These include guardrail, median barriers and bridge rails. The problem results because barriers are placed relatively close to the travel lane (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 may be no practical alternatives to their location.

The designer should, however, check the line of sight over a barrier along a horizontal curve and attempt, if practical, to locate the barrier such that it does not block the line of sight. The following should be considered:

1. Superelevation. An elevated roadway will elevate the driver eye and, therefore, improve the line of sight over the barrier.
2. Grades. The line of sight over a barrier may be improved for a driver on an upgrade and lessened on a downgrade.
3. Barrier Height. The higher the barrier, the more obstructive it will be to the line of sight.

Each barrier location on a horizontal curve will require an individual analysis to determine its impacts on the line of sight. The designer must determine the elevation of the driver eye, the elevation of the object (6 inches above the pavement surface) and the elevation of the barrier where the line of sight intercepts the barrier run. If the barrier does block the line of sight to a 6-inch object, the designer should consider relocating the barrier or revising the horizontal alignment. If the barrier blocks the sight distance needed for minimum SSD, on the mainline, it will be necessary to obtain a design exception. See Section 5.8.

6.6 MATHEMATICAL DETAILS FOR HORIZONTAL CURVES

This Section presents mathematical details used by ODOT for various applications to the design of horizontal curves. Table 6.6A summarizes the figures in Section 6.6.

Table 6.6A

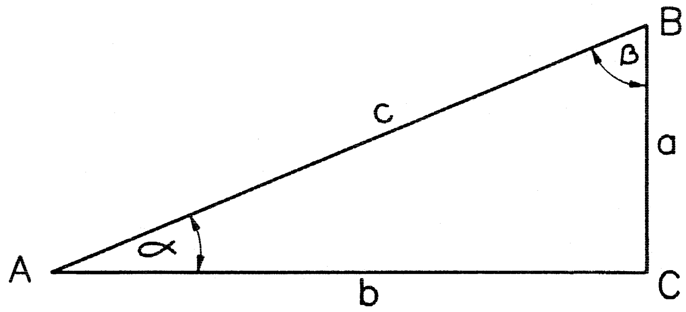
MATHEMATICAL DETAILS FOR HORIZONTAL CURVES

Figure Number	Figure Title
Figure 6.6A	Basic Trigonometric Formulas (Right Triangle Solution)
Figure 6.6B	Curve Symbols, Abbreviations and Formulas
Figure 6.6C	Arc Definition of Circular Curve
Figure 6.6D	Simple Curves (Geometric Principles)
Figure 6.6E	Simple Curves (Various Elements)
Figure 6.6F	Simple Curves (Various Elements)
Figure 6.6G	Simple Curves (Various Elements)
Figure 6.6H	Simple Curve Computation
Figure 6.6I	Simple Curves (Stationing)
Figure 6.6J	Alignment Computation (Simple Curve - Different Degree, Tangent Offset & Parallel)
Figure 6.6K	Alignment Computation (Simple Curve - Compute PC & PT, Joining Tangent Offsets)
Figure 6.6L	Curve (Between Two Fixed Curves)
Figure 6.6M	Curve (Between A Fixed Curve and Fixed Tangent)
Figure 6.6N	Curve (Establish a Tangent Between Two Curves)
Figure 6.6O	Symmetrical 3-Centered Curve
Figure 6.6P	Curve Introduction
Figure 6.6Q	Alignment (Common Point of Tangency for Two Curves)
Figure 6.6R	Common Point of Tangency for Two Curves (Sample Problem)
Figure 6.6S	POC Computation Using Right Triangles
Figure 6.6T	POC Computation Using Right Triangles (Sample Problem)
Figure 6.6U	POC Computation Using Right Triangles
Figure 6.6V	POC Computation Using Right Triangles (Sample Problem)
Figure 6.6W	POC Computation Using Oblique Triangle
Figure 6.6X	POC Computation Using Oblique Triangle (Sample Problem)
Figure 6.6Y	POC of Line 90° to Curve Tangent

Table 6.6A

MATHEMATICAL DETAILS FOR HORIZONTAL CURVES

Figure Number	Figure Title
Figure 6.6Z	Reversed Curves to Parallel Tangents
Figure 6.6AA	Reversed Curves to Parallel Tangents (Sample Problem)
Figure 6.6BB	Reversed Curves (Tangent Not Parallel)
Figure 6.6CC	Reversed Curves (Between Parallel Curves)
Figure 6.6DD	Reversed Curves (To Parallel Tangents with Tangent Between)
Figure 6.6EE	Curve Between Fixed Tangent and Fixed Curve (Case I)
Figure 6.6FF	Curve Between Fixed Tangent and Fixed Curve (Case I) (Sample Problem)
Figure 6.6GG	Curve Between Fixed Tangent and Fixed Curve (Case II)
Figure 6.6HH	Curve Between Fixed Tangent and Fixed Curve (Case III)
Figure 6.6II	Curve Between Fixed Tangent and Fixed Curve (Case IV)
Figure 6.6JJ	Curve Between Fixed Tangent and Fixed Curve (Case V)
Figure 6.6KK	Three Curves Tangent to Each Other
Figure 6.6LL	Intersection of Two Curves
Figure 6.6MM	Spiraled Curves
Figure 6.6NN	Spiraled Curve (Definitions)
Figure 6.6OO	Spiral Formulas
Figure 6.6PP	Spiral Example
Figure 6.6QQ	Parallel Transitions
Figure 6.6RR	3-Centered Curves
Figure 6.6SS	3-Centered Curve Example
Figure 6.6TT	Compass Tangent Construction
Figure 6.6UU	Compass Tangent Construction



Let BC = a, AC = b, AB = c. Then:

1. $\sin \alpha = \dfrac{a}{c}$

2. $\cos \alpha = \dfrac{b}{c}$

3. $\tan \alpha = \dfrac{a}{b}$

4. $csc \ \alpha = \dfrac{1}{\sin \alpha} = \dfrac{c}{a}$

5. $\sec \alpha = \dfrac{1}{\cos \alpha} = \dfrac{c}{b}$

6. $\cot \alpha = \dfrac{1}{\tan \alpha} = \dfrac{b}{a}$

7. $vers \ \alpha = 1 - \cos \alpha$

8. $covers \ \alpha = 1 - \sin \alpha$

9. $exsec \ \alpha = \sec \alpha - 1$

10. $coexsec \ \alpha = csc \ \alpha - 1$

11. $a^2 + b^2 = c^2$

12. $\alpha + \beta = 90°$

13. $Area = \dfrac{1}{2} ab$

BASIC TRIGONOMETRIC FORMULAS
(Right Triangle Solution)

Figure 6.6A

CURVE SYMBOLS

Δ	=	Intersection Angle
D	=	Degree of curve based on 100′ arc
T	=	Tangent Distance = distance from PC to PI = distance from PI to PT
L	=	Length of curve in ft = distance from PC to PT along curve
R	=	Radius of curve in ft
E	=	External Distance (PI to mid-point of curve)
V	=	Intersection of radii at center of circular arc
C	=	Length of long chord in ft - PC to PT
M	=	Middle Ordinate (mid-point of arc to mid-point of long chord)
a	=	Length of arc in ft
c	=	Length of any chord from PC to any point on curve, or chord for any given arc in ft
d	=	Deflection angle from tangent to any point on curve
x	=	Distance along tangent from PC to any point on curve (perpendicular to radius line)
t	=	Tangent offset to any point on curve

CIRCULAR CURVE ABBREVIATIONS

P.C.	=	PC	=	Point of Curvature (Beginning of Curve)
P.T.	=	PT	=	Point of Tangency (End of Curve)
P.I.	=	PI	=	Point of Intersection of Tangents
P.R.C.	=	PRC	=	Point of Reverse Curvature
P.C.C.	=	PCC	=	Point of Compound Curvature
P.O.S.T.	=	POST	=	Point on Sub-Tangent

CURVE FORMULA

$\Delta = LD/100$

$D = 5729.57795/R \textit{ for } 100' \textit{ Arc}$

$D = 100\ \Delta/L$

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

$L = 100\ \Delta/D$

$R = 5729.57795/D \textit{ for } 100' \textit{ Arc}$

$$E = \frac{R}{\cos(\Delta/2)} - R$$

$C = 2R(\sin(\Delta/2))$

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

$a = 200\ d/D$

$c = 2R(\sin aD/200)$

$d = aD/200$

$\cos d = (R - t)/2R$

$x = R \sin 2d = (c) \cos d$

$t = (c) \sin d$

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

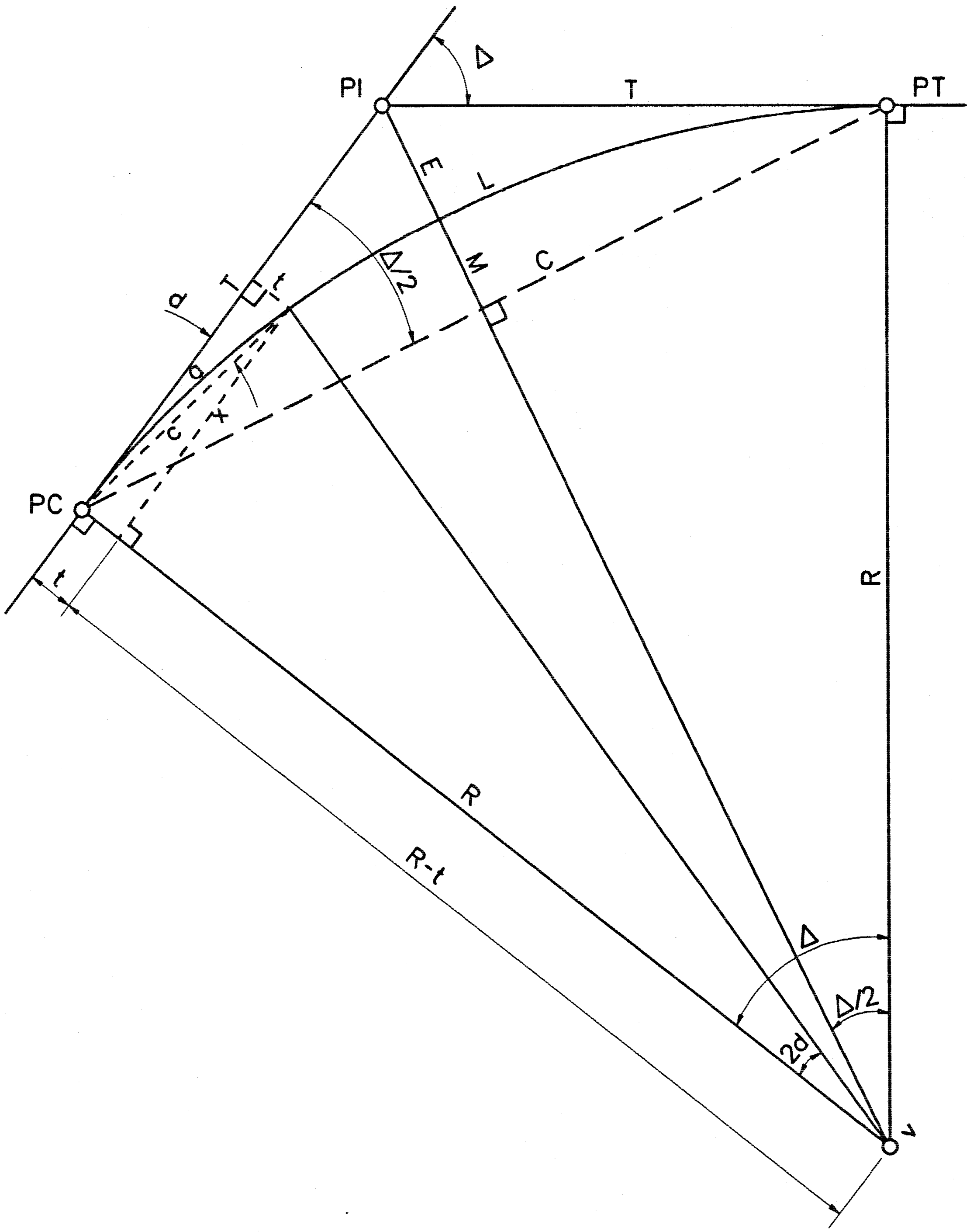
$t = R - (R \cos 2d)$

$t = R(1 - \cos 2d)$

$\pi = 3.141592653$

CURVE SYMBOLS, ABBREVIATIONS AND FORMULAS

Figure 6.6B



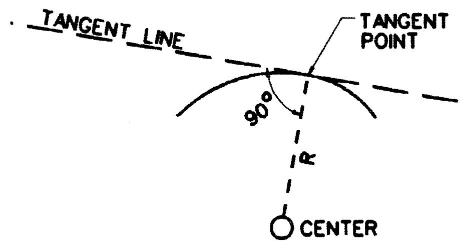
Note: All highway circular curves are based on the arc definition; most railroad circular curves are based on the chord definition.

ARC DEFINITION OF CIRCULAR CURVE

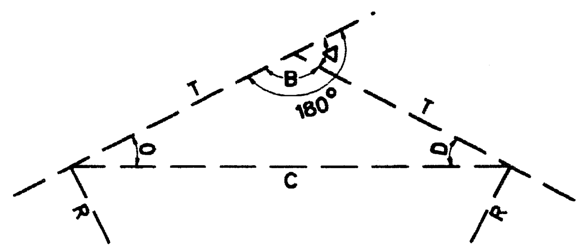
Figure 6.6C

GEOMETRIC PRINCIPLES

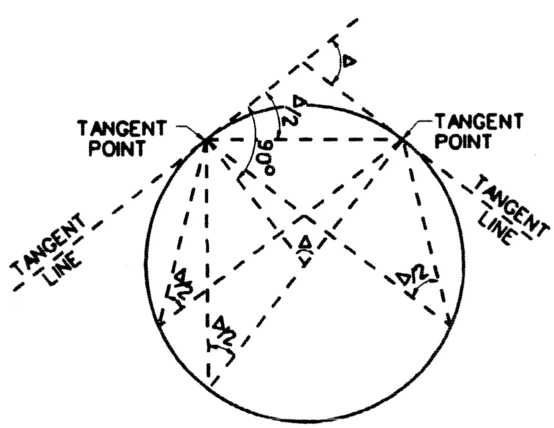
The radius of a circular curve drawn to the tangent point is perpendicular to the tangent at that point.



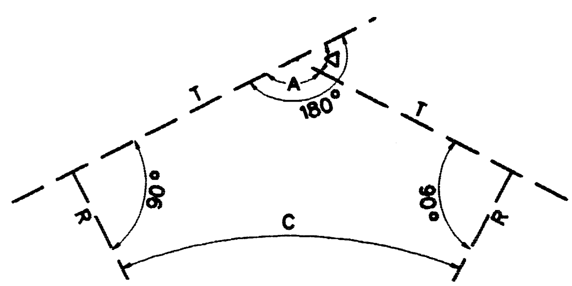
The figure below forms an isosceles triangle. Therefore, Angle O = Angle D. Also, B+D+O=180° (sum of the interior angles of a triangle). Also, Δ+B=180° (angle around a point forming a straight line). Therefore, Δ=O+D and, having shown that O=D, then Δ=O+O=2 *x* O or O=(Δ/2).



From any point on a circular curve, the angle intercepting a given arc on the same circular curve is equal to ½ the central angle (Δ) for that particular arc.



The figure below shows the 2 tangents and 2 radii of a simple curve. A+Δ=180°. Also, A+90°+90°+C=360° (sum of the interior angles in a 4-sided figure) or A+C=180°. Therefore, Δ=C. C is sometimes called the central angle but is usually designated by Δ.



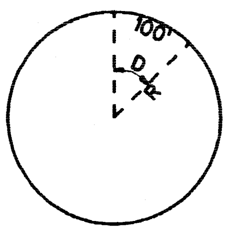
SIMPLE CURVES
(Geometric Principles)

Figure 6.6D

ARC DEFINITION

The "Arc Definition" for degree of curve states that D is the central angle subtended by an arc of 100′ (generally used in highway work).

Do not confuse this with the "Chord Definition" which states that D is the central angle subtended by a chord of 100′ (generally used in railroad work).



The illustration shows a circle with D representing the degree of curve. Thus:

$$Circumference = \frac{100 \times 360}{D}$$

Also $2\pi R$ = Circumference. Therefore:

$$2\pi R = \frac{100 \times 360}{D}$$

Where: R is in ft
D is in degrees

Solving for R:

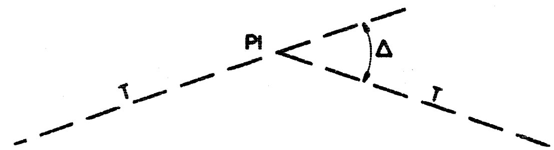
$$\pi R = \frac{50 \times 360}{D}$$

$$R = \frac{50 \times 360}{\pi D}\ ;\ R = \frac{5729.58}{D}$$

Also: $D = \dfrac{5729.58}{R}$

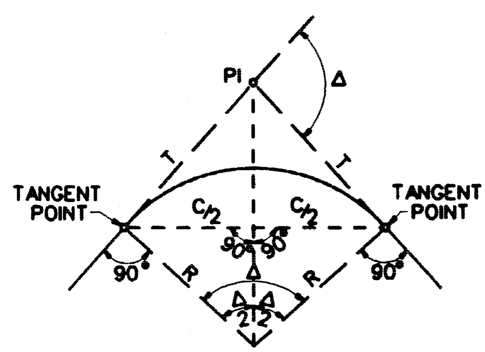
(It can be readily seen that the radius of a 1° curve is 5729.58′).

Δ = DELTA ANGLE OR INTERSECTION ANGLE



Δ = The deflection angle from the first tangent extended. This is the same angle as the angle between radii (central angle). This should be known before the other parts of the curve are computed.

R = RADIUS



Formulas: $R = \dfrac{5729.58}{D}$

OR $R = \dfrac{T}{\tan(\Delta/2)}$

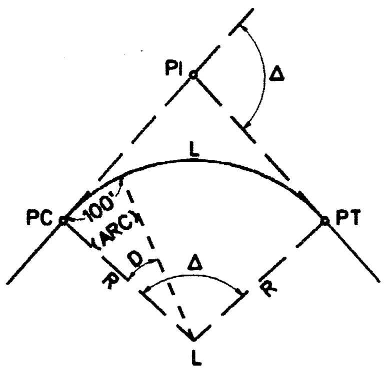
OR $R = \dfrac{C}{2 \sin(\Delta/2)}$

Where R is in ft and D and Δ are in degrees.

SIMPLE CURVES
(Various Elements)

Figure 6.6E

<u>L = LENGTH OF CURVE</u>
<u>D = DEGREE OF CURVE</u>



With a constant Δ, L increases or decreases in direct ratio to R.

Formulas for D: $D = \dfrac{5729.58}{R}$

$D = \dfrac{\Delta}{L} x\ 100$

Formulas for L: $L = \dfrac{\Delta}{D} x\ 100$

Where Δ and D are in degrees.

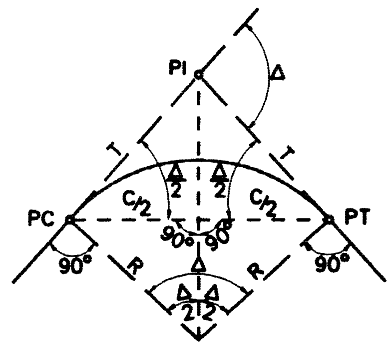
Or, L is in direct ratio to Δ. Thus:

$$L = \frac{\Delta}{360}\, x\ 2\pi R$$

Reducing to: $L = .0174533\ \Delta\, R$

Where L and R are in ft, and Δ is in degrees.

<u>T = TANGENT LENGTH</u>

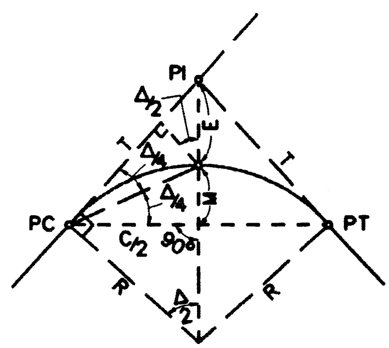


Formulas: $T = R \tan\ (\Delta/2)$

$$T = \frac{C}{2 \cos\ (\Delta/2)}$$

Where T, L & R are in ft, and D is in degrees.

<u>M = MIDDLE ORDINATE</u>



Where M, C, R and E are in ft.

Formulas:

$M = \dfrac{C}{2} \tan\ (\Delta/4)$ Also: $M = E \cos\ (\Delta/2)$

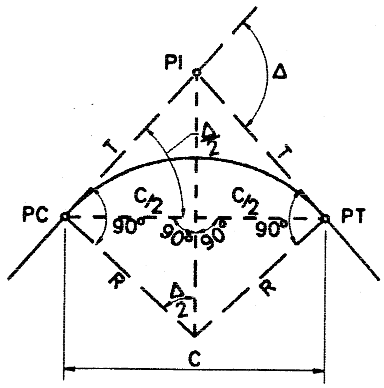
or

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

SIMPLE CURVES
(Various Elements)

Figure 6.6F

C = CHORD



Formulas:

$$\frac{C}{2} = R \sin(\Delta/2); \quad \text{or}$$

$$C = 2R \sin(\Delta/2)$$

Also: $C = 2T \cos(\Delta/2)$

The main chord and short chords are often convenient to use in laying out the curve.

Figure can be applied to the whole chord or to the chord of any part of the curve. Δ would then be the central angle of the arc of whatever part of the curve is being considered.

E = EXTERNAL

Formulas:

$$E = R\left(1 - \frac{1}{\cos(\Delta/2)}\right)$$

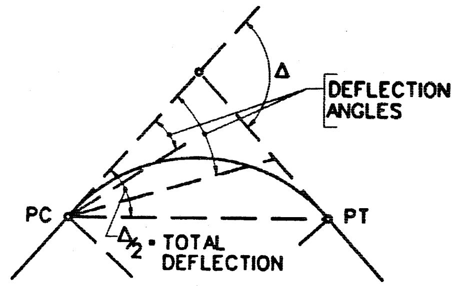
Also: $E = \dfrac{M}{\cos(\Delta/2)}$; or

$$E = T \tan(\Delta/4); \text{or}$$

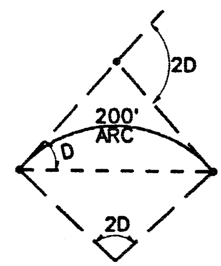
$$E = \frac{C}{2}(\tan(\Delta/2) - \tan(\Delta/4))$$

DEFLECTION ANGLES

In circular curves for highways, the deflection angle to a point on a curve is usually turned from the tangent with a set-up on the PC. (See figure below.)



The deflection angle in degrees for 200 ft of arc on a circular curve is equal to D (See figure below.)



The deflection angle (in degrees):

For $100' = \dfrac{D}{2}$ For $50' = \dfrac{D}{4}$

For $25' = \dfrac{D}{8}$ For $10' = \dfrac{D}{20}$

For $1' = \dfrac{D}{200}$

The deflection angle in minutes per ft

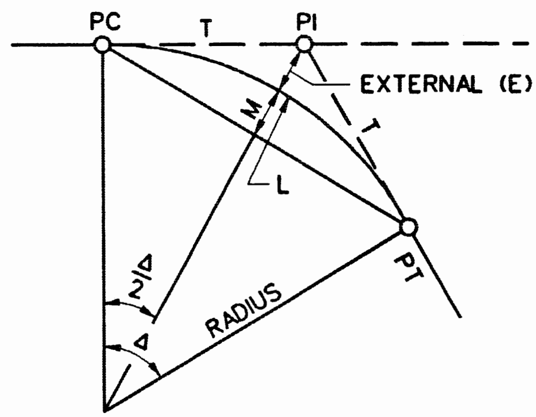
$$= \frac{60D}{200} = 0.3D$$

The deflection angle in minutes for any arc = 0.3D x arc length.

SIMPLE CURVES
(Various Elements)

Figure 6.6G

Δ = Central Angle
R = Radius
D = Degree of Curve
T = Tangent
L = Length of Curve
M = Middle Ordinate
E = External
R = For 1° = 5729.578′



$$R = \frac{5729.578}{D} \quad \text{or} \quad R = \frac{T}{\tan(\Delta/2)}$$

$$D = \frac{5729.578}{R} \qquad T = R \tan(\Delta/2)$$

$$L = \frac{\Delta}{D} \, x \, 100$$

$$M = R - (R \cos(\Delta/2)) \text{ or}$$

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

When R is known, $E = \dfrac{R}{\cos(\Delta/2)} - R$

When T is known, $E = T(\tan(\Delta/4))$

Example:

Given:

$PI = Sta.161 + 60.35;\ \Delta = 62°10';\ D = 8°20'$

To find: Sta. of PC and PT

1. Change 8°20′ to decimals or 8.3333°:

$$R = \frac{5729.578}{8.3333} = 687.5493'$$

2. $T = R \tan(\Delta/2) = 687.5493 \, x \, 0.60284$

$T = 414.48$

3. $L = \dfrac{\Delta}{D} \, x \, 100 = \dfrac{62.1666}{8.3333} \, x \, 100 = 746.00'$

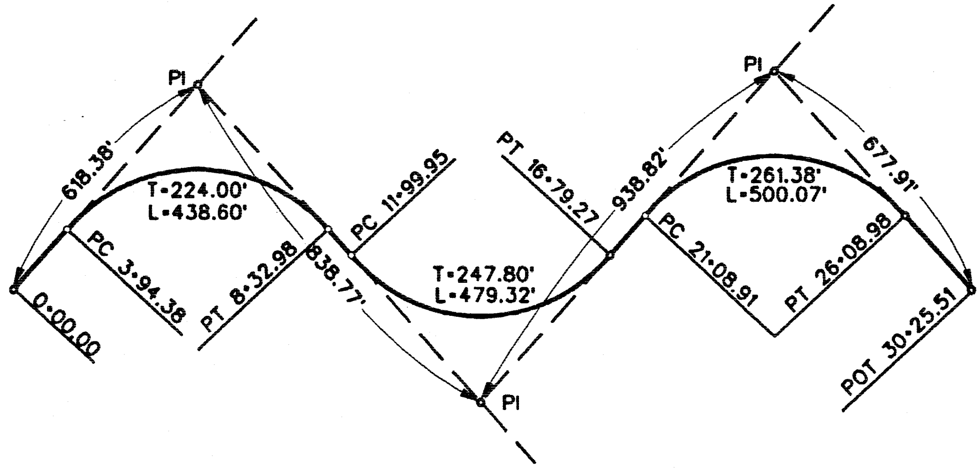
4. Therefore:

PI	=	Sta. 161+60.35
T	=	-414.48
PC	=	Sta. 157+45.87
L	=	+746.00
PT	=	Sta. 164+91.87

SIMPLE CURVE COMPUTATION

Figure 6.6H

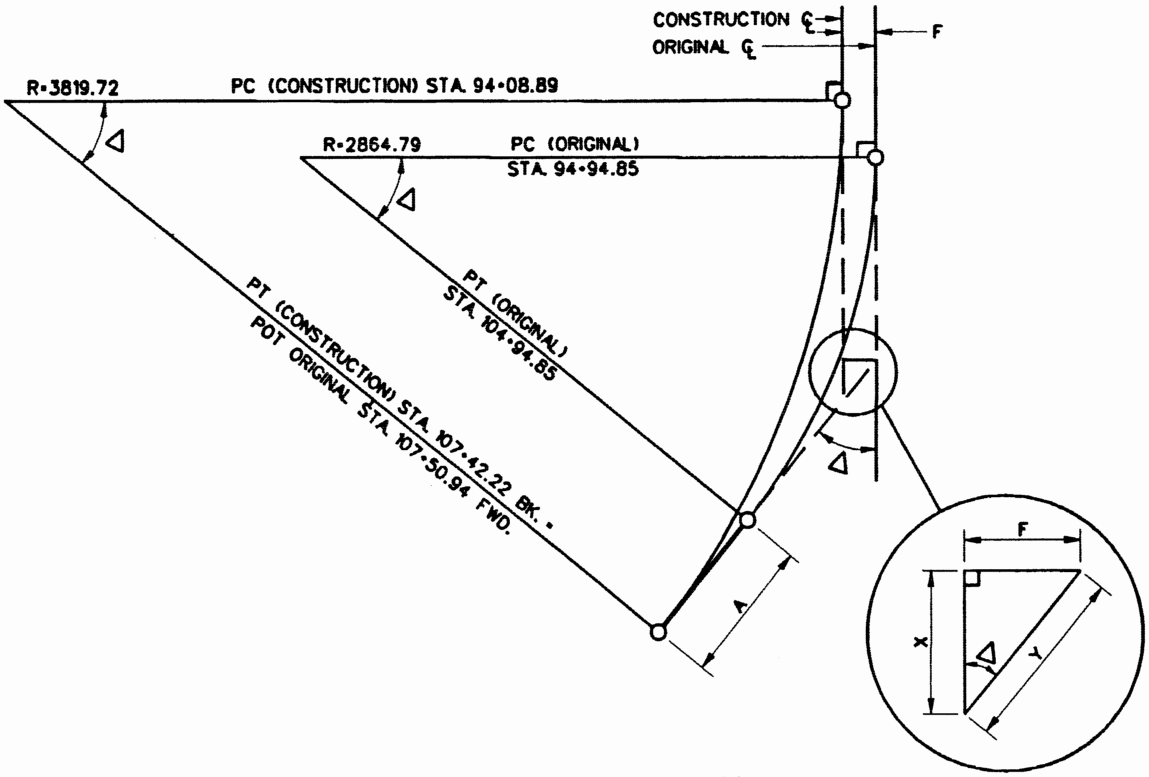
STATIONING



1. The station at the first PI is 6 + 18.38.
2. The station at the first PC = 6 + 18.38 - 224.00 = 3 + 94.38.
3. The station at the first PT = 394.38 + 438.60 = 8 + 32.98.
4. The station at the second PC = 832.98 + (838.77 - 224.00 - 247.80) = 11 + 99.95.
5. The station at the second PI = 11 + 99.95 + 247.80 = 14 + 47.75.
6. The station at the second PT = 1199.95 + 479.32 = 16 + 79.27.
7. The station at the third PC = 1679.27 + (938.82 - 247.80 - 261.38) = 21 + 08.91.
8. The station at the third PI = 2108.91 + 261.38 = 23 + 70.29.
9. The station at the third PT = 2108.91 + 500.07 = 26 + 08.98.
10. The station at the final POT = 2608.98 + (677.91 - 261.38) = 30 + 25.51.
11. Check: (618.38 + 838.77 + 938.82 + 677.91) - (2 *x* 224.00 + 2 *x* 247.80 + 2 *x* 261.38 - 438.60 - 479.32 - 500.07) = 30 + 25.51.

SIMPLE CURVES
(Stationing)

Figure 6.6I



Given: Simple Curve

Original PI	=	100 + 00.00
Δ	=	20°00′ Rt.
D	=	2°
T	=	505.15′
L	=	1000.00′
R	=	2864.79′

Problem:

Compute: Simple curve of different degree where one tangent is offset a specified distance from and parallel to the original tangent.
D = 1°30′, F = 30.0′ Rt.

Solution:

Y = F/sin Δ = 30.0/sin 20° = 30.0/0.34202 = 87.71′
X = F/tan Δ = 30.0/tan 20° = 30.0/0.36397 = 82.42′
T = (R)tan (Δ/2) = 3819.72 tan 10° = 673.53′
L = 100 Δ/D = (100)(20°)/1°30′ = 1333.33′

CONSTRUCTION CURVE DATA

PI	=	100 + 82.42
Δ	=	20°00′ Rt.
D	=	1°30′
T	=	673.53′
L	=	1333.33′
R	=	3819.72′

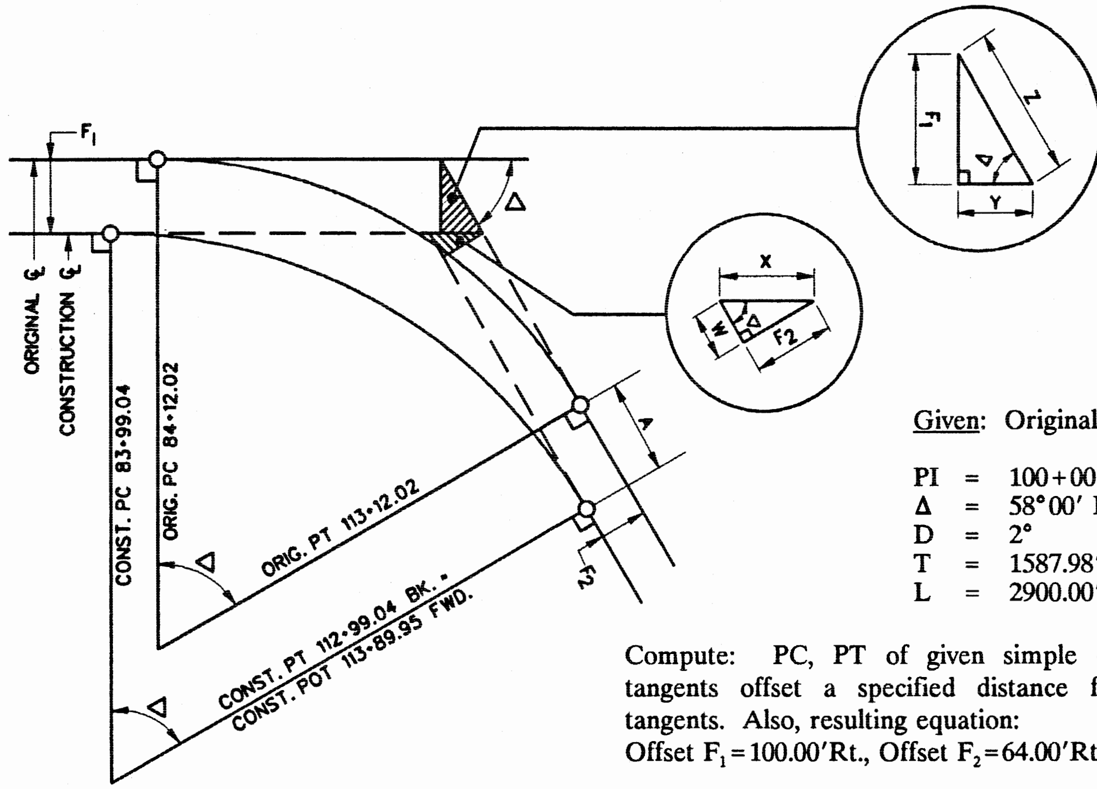
STATIONING

Orig. PI =	100+00.00	Const. T =	6+73.53
+x =	82.42	+Y =	87.71
Const. PI =	100+82.42		7+61.24
-T =	673.53	-Orig. T =	- 5+05.15
Const. PC =	94+08.89	A =	2+56.09
+L =	1333.33	Orig. PT =	104+94.85
Const. PT =	107+42.22	Orig. POT =	107+50.94

Equation Const. PT 107+42.22 Bk = Orig. POT 107+50.94 Fwd

ALIGNMENT COMPUTATION
(Simple Curve - Different Degree, Tangent Offset & Parallel)

Figure 6.6J



Given: Original Curve Data:

PI	=	100+00.00
Δ	=	58°00′ Rt.
D	=	2°
T	=	1587.98′
L	=	2900.00′

Compute: PC, PT of given simple curve joining tangents offset a specified distance from original tangents. Also, resulting equation:
Offset F_1 = 100.00′Rt., Offset F_2 = 64.00′Rt.

$Z = F_1/\sin\Delta = 100.00'/\sin 58° = 100.00'/0.84805 = 117.92'$
$Y = F_1/\tan\Delta = 100.00'/\tan 58° = 100.00'/1.60033 = 62.49'$
$X = F_2/\sin\Delta = 64.00'/\sin 58° = 64.00'/0.84805 = 75.47'$
$W = F_2/\tan\Delta = 64.00'/\tan 58° = 64.00'/1.60033 = 39.99'$
Const. PI = Orig. PI + Y - X
= 100+00.00 + 62.49′ - 75.47′ = 99+98.02

Note: In many cases, Y>X & W>Z or offsets may be to other side of original tangents causing the picture of problem to look different, but the principles of the problem remain the same.

Const. PI	=	99+87.02	
-T	=	15+87.98	
Const. PC	=	83+99.04	
+L	=	29+00.00	
Const. PT	=	112+99.04	
A = Z - W = 117.92′ - 39.99′	=	77.93	
Orig. PT	=	113+12.02	
+A	=	77.93	
		113+89.95	(Orig. POT 64′ Lt. of Const. PT 113+89.95)

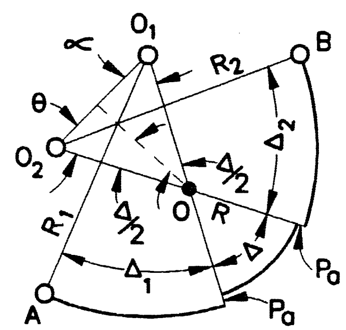
Equation: Const. PT 112+99.04 Bk = Sta. 113+89.95 Fwd

ALIGNMENT COMPUTATION
(Simple Curve - Compute PC & PT, Joining Tangent Offsets)

Figure 6.6K

CASE I - TO INTRODUCE A CURVE OF SELECTED DEGREE BETWEEN TWO FIXED CURVES

Part A: Fixed curves of equal degree.



$R_1 = R_2$

Given:

R_1 and R_2 = Radii of fixed curves with radials AO_1 and BO_2 fixed on coordinate system.

R = Radius selected for intermediate curve.

Pa, Pa_1 and Pa_2 = Offset ("p" distance) to permit insertion of selected spirals; without spirals, Pa = 0.

Problem:

To determine Δ, Δ_1 and Δ_2 and the remaining properties of each curve.

Solution:

Determine length and bearing of O_1O_2 from given coordinates of O_1 and O_2.

$$\Delta = 2 \sin^{-1} \frac{O_1O_2}{2(R_1 - R - Pa)*}$$

$$\propto = \theta = 90° - \frac{\Delta}{2}$$

Determine bearing O_1O by applying ∝ to bearing O_1O_2.

Determine bearing O_2O by applying θ to bearing O_1O_2.

Δ_1 = difference in bearings of O_1O and O_1A.
Δ_2 = difference in bearings of O_2O and O_2B.

Determine remaining properties of each curve through usual procedure.

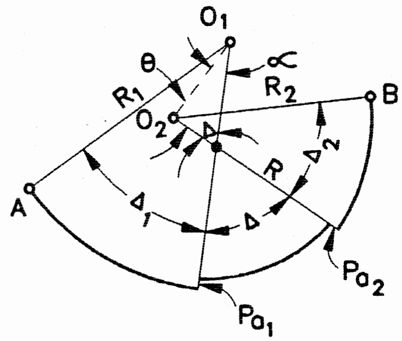
* Where R is greater than R_1 and/or R_2:

$$\Delta = 2 \sin^{-1} \frac{O_1O_2}{2(R - R_1 - Pa)}$$

CURVE
(Between Two Fixed Curves)

Figure 6.6L

Part B: Fixed curves of unequal degree.



Given:

R_1 and R_2 = Radii of fixed curves with radials AO_1 and BO_2 fixed on coordinate system.

R = Radius selected for intermediate curve.

Pa, Pa_1 and Pa_2 = Offset ("p" distance) to permit insertion of selected spirals; without spirals, Pa=0.

Problem:

To determine Δ, Δ_1 and Δ_2 and the remaining properties of each curve.

Solution:

Determine length and bearing of O_1O_2 from given coordinates of O_1 and O_2.

$$OO_1 = R_1 - (R + Pa_1)*$$

$$OO_2 = R_2 - (R + Pa_2)*$$

$$\Delta = \cos^{-1}\frac{OO_1^2 + OO_2 - O_1O_2^{\ 2}}{2\ x\ OO_1\ x\ OO_2}$$

$$\propto = \cos^{-1}\frac{OO_1^2 + OO_2^2 - O_1O_2^{\ 2}}{2\ x\ OO_1\ x\ O_1O_2}$$

$$\theta = 180° - (\Delta + \propto)$$

Determine bearing O_1O by applying $\propto$ to bearing O_1O_2.

Determine bearing O_2O by applying θ to bearing O_1O_2.

Δ_1 = difference in bearings of O_1O and O_1A.
Δ_2 = difference in bearings of O_2O and O_2B.

Determine remaining properties of each curve through usual procedure.

* Where R is greater than R_1 and/or R_2:

$$OO_1 = R - (R_1 + Pa_1)$$

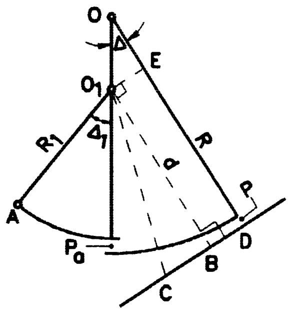
$$OO_2 = R - (R_2 + Pa_2)$$

CURVE
(Between Two Fixed Curves)

Figure 6.6L (Continued)

CASE II - TO INTRODUCE A CURVE OF SELECTED DEGREE BETWEEN A FIXED CURVE AND A FIXED TANGENT

Part A: Selected curve of lesser degree than the fixed curve.



Given:

R_1 = Radius of fixed curve with coordinated radial AO_1.

C = Any coordinated point on fixed tangent of known bearing.

R = Radius of selected curve.

P & Pa = Offset ("p" distance) to permit insertion of selected spirals; without spirals, P and Pa = 0.

Problem:

To determine Δ, Δ_1 and remaining properties of each curve.

Solution:

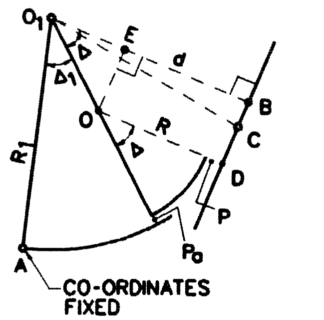
In ΔO_1CB, solve for O_1B, or d.

$OE = OD - ED = (R + P) - d$

$$\Delta = \cos^{-1}\frac{OE}{OO_1} = \cos^{-1}\frac{(R + P) - d}{R - (R_1 + Pa)}$$

Determine remaining properties of each curve through usual procedure.

Part B: Selected curve of greater degree than the fixed curve.



Given:

R_1 = Radius of fixed curve with coordinated radial AO_1.

C = Any coordinated point on fixed tangent of known bearing.

R = Radius of selected curve.

P & Pa = Offset ("p" distance) to permit insertion of selected spirals; without spirals, P and Pa = 0.

Problem:

To determine Δ, Δ_1 and remaining properties of each curve.

Solution:

In ΔO_1CB, solve for O_1B, or d.

$O_1E = d - (R + P)$

$$\Delta = \cos^{-1}\frac{O_1E}{O_1O} = \cos^{-1}\frac{d - (R + P)}{R_1 - (R + Pa)}$$

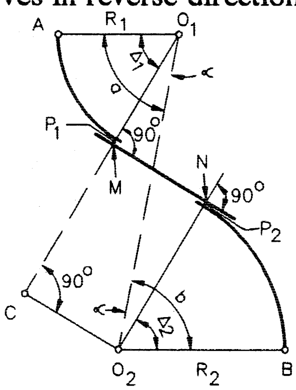
Determine remaining properties of each curve through usual procedure.

CURVE
(Between a Fixed Curve and Fixed Tangent)

Figure 6.6M

CASE III - TO ESTABLISH A TANGENT BETWEEN TWO CURVES

Part A: Curves in reverse direction.



Given:

R_1 & R_2 = Radii of fixed curves with coordinated radials AO_1 & BO_2.

P_1 & P_2 = Offset ("p" distance) to permit insertion of selected spirals; without spirals, P_1 and $P_2 = 0$.

Problem:

To determine length and bearing of tangent MN, Δ_1, Δ_2 and the remaining properties of each curve.

Solution:

Determine length and bearing of O_1O_2 from known coordinates; then, in ΔO_1O_2C:

$$\propto = \cos^{-1} \frac{(R_1 + P_1)* + (R_2 + P_2)*}{O_1O_2}$$

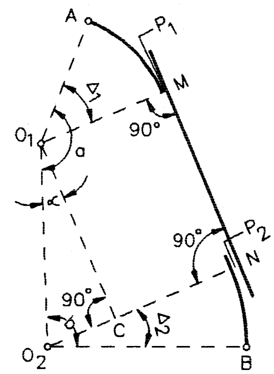
$$MN = [(R_1 + P_1)* + (R_2 + P_2)*] \tan \propto$$

Determine angles a and b from bearings of O_1A, O_2B and O_2O_1:

$$\Delta_1 = a - \propto; \Delta_2 = b - \propto$$

Determine remaining properties of each curve through usual procedure.

Part B: Curves in same direction.



Given:

R_1 & R_2 = Radii of fixed curves with coordinated radials AO_1 & BO_2.

P_1 & P_2 = Offset ("p" distance) to permit insertion of selected spirals; without spirals, P_1 and $P_2 = 0$.

Problem:

To determine length and bearing of tangent MN, Δ_1, Δ_2 and the remaining properties of each curve.

Solution:

Determine length and bearing of O_1O_2 from known coordinates; then, in ΔO_1O_2C:

$$\propto = \cos^{-1} \frac{(R_2 + P_2)* - (R_1 + P_1)*}{O_1O_2}$$

$$MN = [(R_2 + P_2)* - (R_1 + P_1)*] \div \tan \propto$$

Determine angles a and b from bearings of O_1A, O_2B and O_2O_1:

$$\Delta_1 = a - 90° - \propto; \Delta_2 = b - 90° + \propto$$

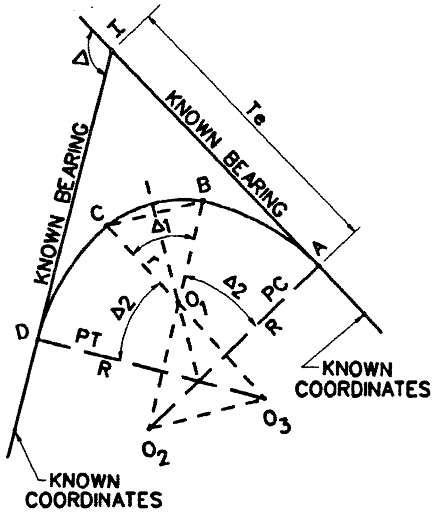
Determine remaining properties of each curve through usual procedure.

*P_1 and P_2 = 0 when spirals are not used.

CURVE
(Establish a Tangent Between Two Curves)

Figure 6.6N

CASE IV - TO INTRODUCE A SYMMETRICAL 3-CENTERED CURVE OF SELECTED RADII BETWEEN TWO GIVEN TANGENTS OF KNOWN INTERSECTION



Given:

Radii r and R and angle Δ.

Problem:

To determine the external tangent, Te.

Solution:

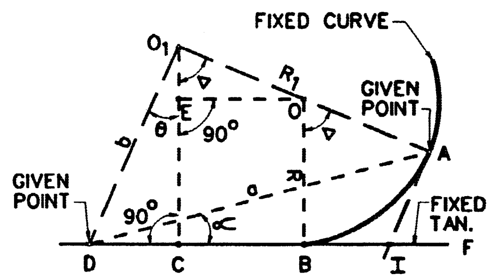
$\Delta_2 = (\Delta - \Delta_1) + 2.$

Compute remaining properties (L, T, etc.) for each curve.

Establish PC (Point A) and PT (Point D) using distance Te.

$$Te = \frac{R \sin(\Delta/2) - [(R - r) \sin(\Delta_1/2)]}{\cos(\Delta/2)}$$

CASE V - TO INTRODUCE A CURVE, AND TO DETERMINE ITS RADIUS, BETWEEN A GIVEN POINT ON A FIXED CURVE AND SOME POINT ON A FIXED TANGENT



Given:

Tangent DF, its bearing and coordinates of D.

R_1 and coordiantes of O_1 and A.

Problem:

To determine R and Δ.

SYMMETRICAL 3-CENTERED CURVE

Figure 6.6O

Solution:

Method 1: Determine bearing and length of DA or a from known coordinates. Determine α from bearings DA and DF. Determine Δ from bearings O_1A and O_1C.

From solution of triangle DIA:

$$R = \frac{a \sin \alpha}{\sin (180° - \Delta) \tan (\Delta/2)}$$

Method 2: Determine bearing and length of DO_1, or b, from known coordinates. Determine θ from bearings O_1D and O_1C. Determine Δ from bearings O_1A and O_1C. From solution of triangles DO_1C and EO_1O:

$$R = \frac{b \cos \theta - R_1 \cos \Delta}{1 - \cos \Delta}$$

Note: If spiral is used at B, Method 2 must be employed; then:

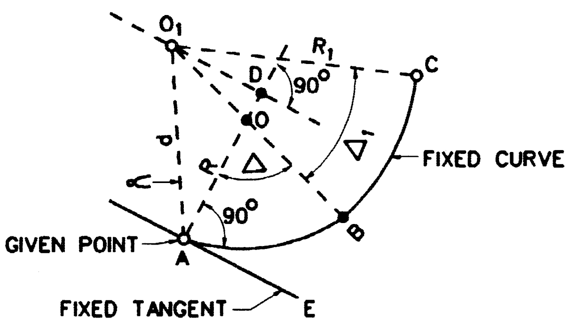
$$R = \frac{b \cos \theta - R_1 \cos \Delta - P}{1 - \cos \Delta}$$

Where P is offset of curve for spiral at B, use expression for R without P first, and find approximate R; then substitute in latter equation, using value of P for the approximate R and find new R. Same formulas apply whether $R <$ or $> R_1$.

SYMMETRICAL 3-CENTERED CURVE (Continued)

Figure 6.6O

CASE VI - TO INTRODUCE A CURVE AND TO DETERMINE ITS RADIUS BETWEEN A GIVEN POINT ON A FIXED TANGENT AND SOME POINT ON A FIXED CURVE



Given:

R_1 and coordinates of O_1 and C. Tangent AE, its bearing and coordinates of A.

Problem:

To determine R, Δ and Δ_1.

Solution:

Erect right triangles AO_1D and O_1OD. Determine length and bearing of AO_1 or d from known coordinates of A and O_1. From solution of triangles AO_1D and O_1OD:

$$R = \frac{d^2 - R_1^2}{2\,(d \cos \alpha - R_1)}$$

and

$$\Delta = \sin^{-1} \frac{d \sin \alpha}{R_1 - R}$$

Determine bearing BO_1 by application of Δ to bearing AD_1, then Δ_1 = difference in bearings of BO_1 and CO_1.

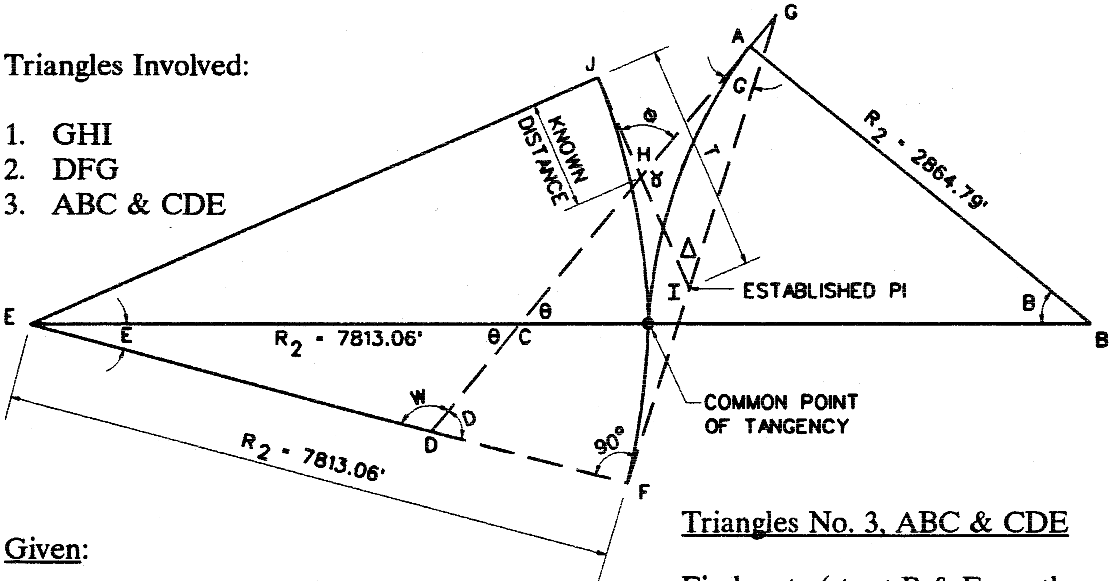
Note: This solution is applicable whether R < or > R_1.

CURVE INTRODUCTION

Figure 6.6P

Triangles Involved:

1. GHI
2. DFG
3. ABC & CDE



Given:

One of the curves already established (JF, Δ, T): Radii of curves (R_1 & R_2). Intersection point and angle of the two centerlines (H & φ).

Distance JH is thereby known and HI can be determiend by subtracting JH from T for established curve.

Distance between curve centers (BE = R_1+R_2).

Triangle No. 1, GHI

Find: ∠G & GI

1. $\gamma = 180° - \phi$
2. $\angle G = 180° - (\gamma + \Delta)$
3. HI = T (est. curve) - JH
4. $GI = \dfrac{HI \sin \gamma}{\sin \angle G}$

Triangle No. 2, DGF

Find: ∠D & DF

5. FG = T (est. curve) + GI
6. DF = FG tan ∠G
7. $\angle D = 90° - \angle G$

Triangles No. 3, ABC & CDE

Find: ∠e (∠s at B & E are thus determined)

$BC + CE = R_1 + R_2$

In triangle ABC:

$$BC = \frac{R_1}{\sin \theta}$$

$$BC + CE = \frac{R_1}{\sin \theta} + \frac{DE \sin W}{\sin \theta} = R_1 + R_2$$

In triangle CDE:

8. $DE = R_2 - DF$
9. $W = 180° - \angle D$

$$CE = \frac{DE \sin W}{\sin \theta}$$

10. $\sin \theta = \dfrac{R_1 + DE \sin W}{R_1 + R_2}$

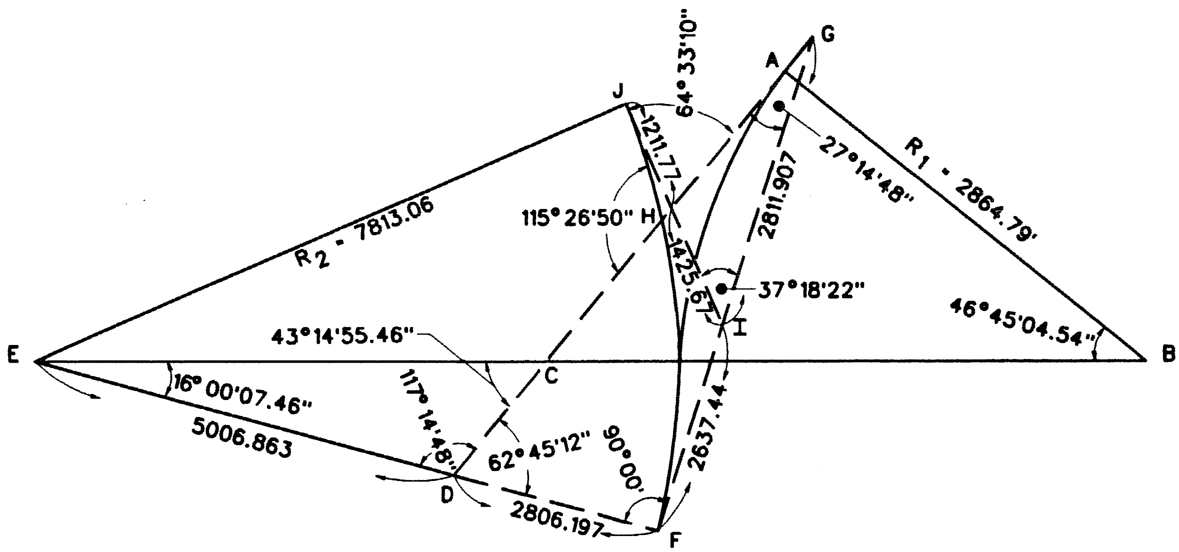
11. $\angle B = 90° - \theta$

12. $\angle E = 180° - (\theta + W)$

Length of curves, tangent length, etc., are determined.

ALIGNMENT
(Common Point of Tangency for Two Curves)

Figure 6.6Q



Given:

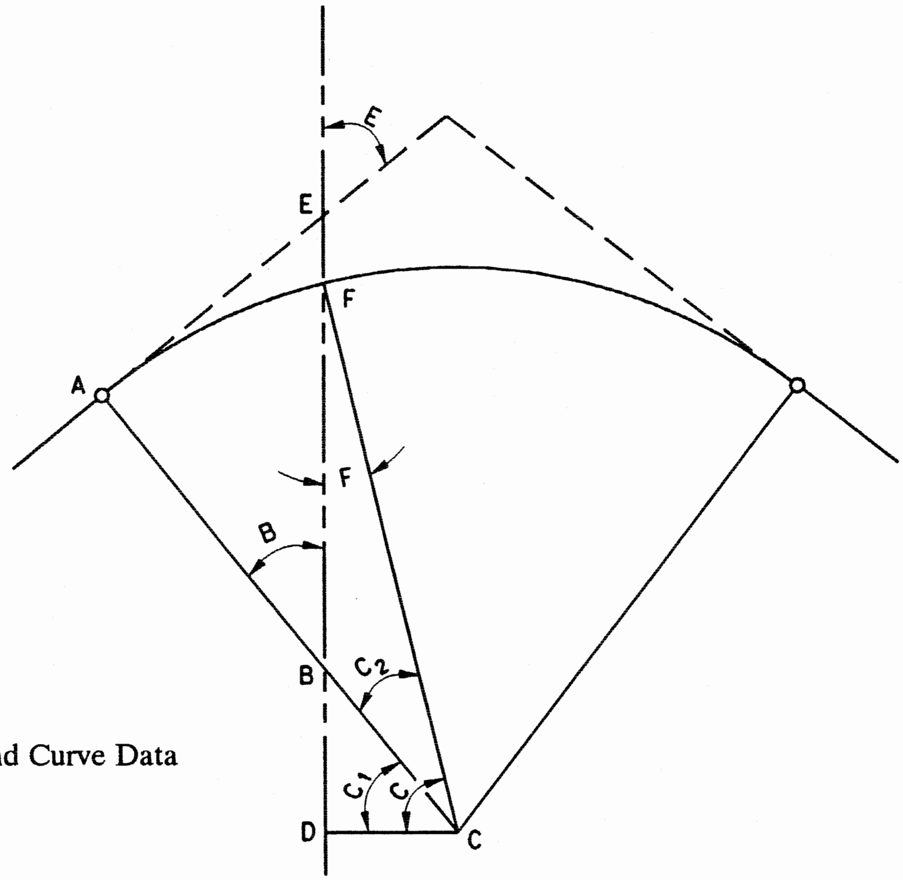
ϕ = 64°33′10″
JH = 1211.77
Δ = 37°18′22″
T = 2637.44
R_2 = 7813.06
R_1 = 2864.79

1. γ = 180°00′00″ - 64°33′10″ = 115°26′50″
2. ∠G = 180°00′00″ - (115°26′50″ + 37°18′22″) = 27°14′48″
3. HI = 2637.44-1211.77 = 1425.67
4. GI = $\dfrac{(1425.67)(.90298146)}{.45782219}$ = 2811.907
5. FG = 2637.44 + 2811.907 = 5549.347
6. DF = (5449.347)(.5149602) = 2806.197
7. ∠D = 90°00′00″ - 27°14′48″ = 62°45′12″
8. DE = 7813.06 - 2806.197 = 5006.863
9. ∠W = 180°00′00″ - 62°45′12″ = 117°14′48″
10. sin θ = $\dfrac{2864.79 + (5006.863)(.88904378)}{2864.79 + 7813.06}$ = .68516695

    θ = 43° 14′55.46″
11. ∠B = 90°00′00″ - 43°14′55.46″ = 46°45′04.54″
12. ∠E = 180°00′00″ - (46°45′04.54″ + 117°14′48″) = 16°00′07.46″

COMMON POINT OF TANGENCY FOR TWO CURVES
(Sample Problem)

Figure 6.6R



Given:

AE, Angle E and Curve Data

Required:

EF and Arc Dist. AF

Solution:

From triangle ABE:

1. $B = 90° - E$
2. $BE = \dfrac{AE}{\sin B}$
3. $AB = BE \cos B$

From triangle BCD:

4. $BC = \text{Radius} - AB$
5. $CD = BC \sin B$
6. $BD = BC \cos B$
7. $C_1 = 90° - B$

From triangle CDF:

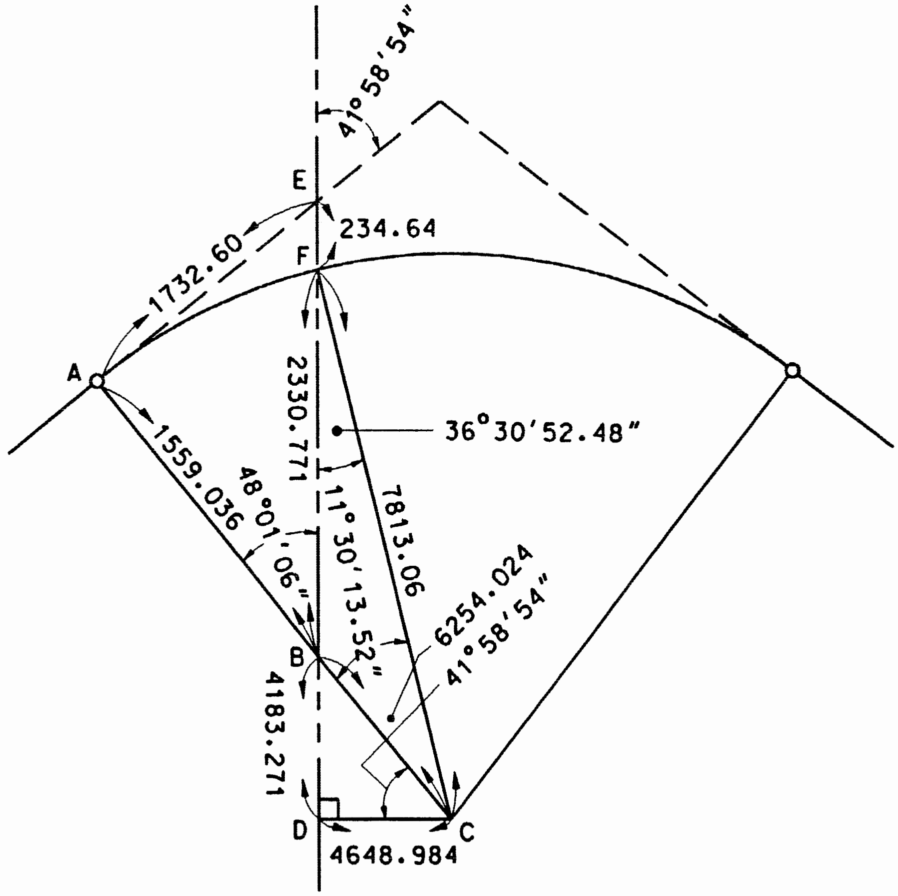
8. $CF = \text{Radius}$
9. $\sin F = \dfrac{DC}{CF}$
10. $C = 90° - F$
11. $C_2 = C - C_1$
12. $DF = CF \cos F$
13. $EF = BD + BE - DF$
14. $\text{Arc Dist. } AF = \dfrac{C_2}{\text{Deg. of Curve}} \times 100$

POC COMPUTATION USING RIGHT TRIANGLES

Figure 6.6S

<u>Given</u>:

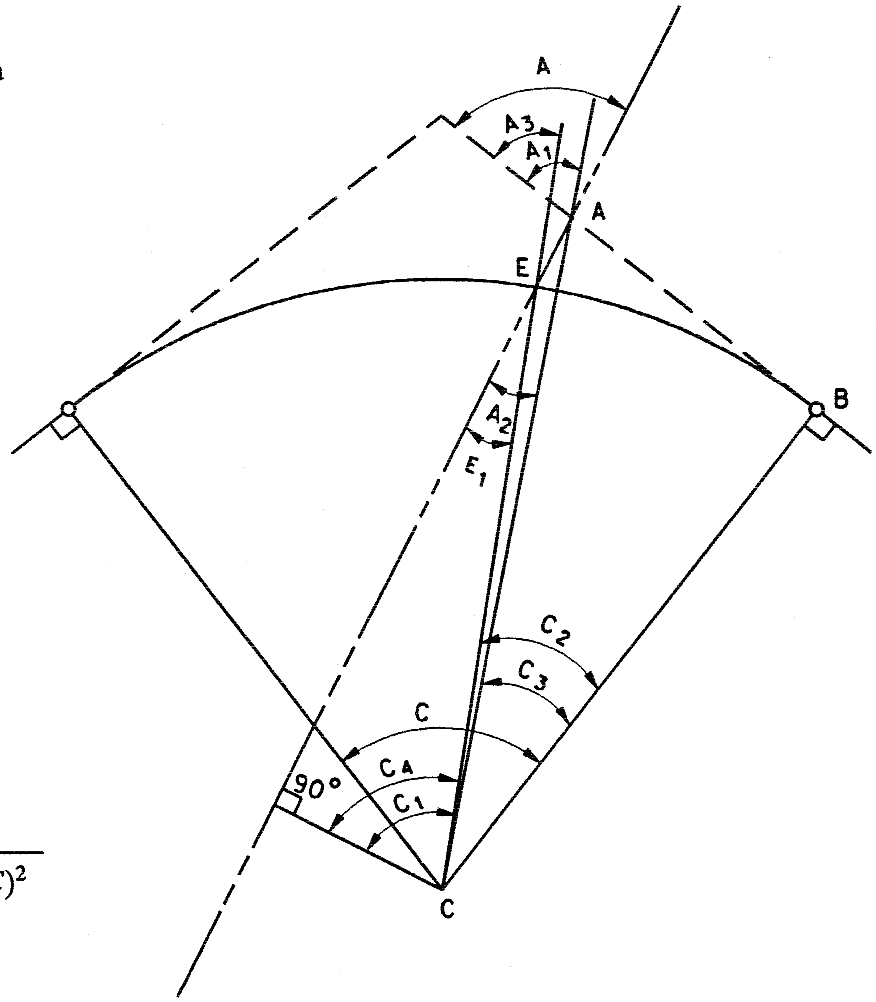
Angle E	=	41°58′54″
AE	=	1732.60
Radius	=	7813.06
Curve D	=	0°44′



1. B = $90°00'00'' - 41°58'54'' = 48°01'06''$
2. BE = $\dfrac{1732.60}{.74335889} = 2330.771$
3. AB = $(2330.771)(.66889278) = 1559.036$
4. BC = $7813.06 - 1559.036 = 6254.024$
5. CD = $(6254.024)(.74335889) = 4648.984$
6. BD = $(6254.024)(.66889278) = 4183.271$
7. C_1 = $90°00'00'' - 48°01'06'' = 41°58'54''$
8. CF = 7813.06
9. sin F = $\dfrac{4648.984}{7813.06} = .59502730$ F = $36°30'52.48''$
10. C = $90°00'00'' - 36°30'52.48'' = 53°29'07.52''$
11. C_2 = $53°29'07.52'' - 41°58'54'' = 11°30'13.52''$
12. DF = $(7813.06)(.80370550) = 6279.399$
13. EF = $4183.271 + 2330.771 - 6279.399 = 234.64$
14. Arc AF = $\dfrac{11.5037556°}{.73333333°} x\ 100 = 1568.69$

POC COMPUTATION USING RIGHT TRIANGLES
(Sample Problem)

Figure 6.6T



Given:

AB, Angle A and Curve Data

Required:

AE and Arc Dist. BE

Solution:

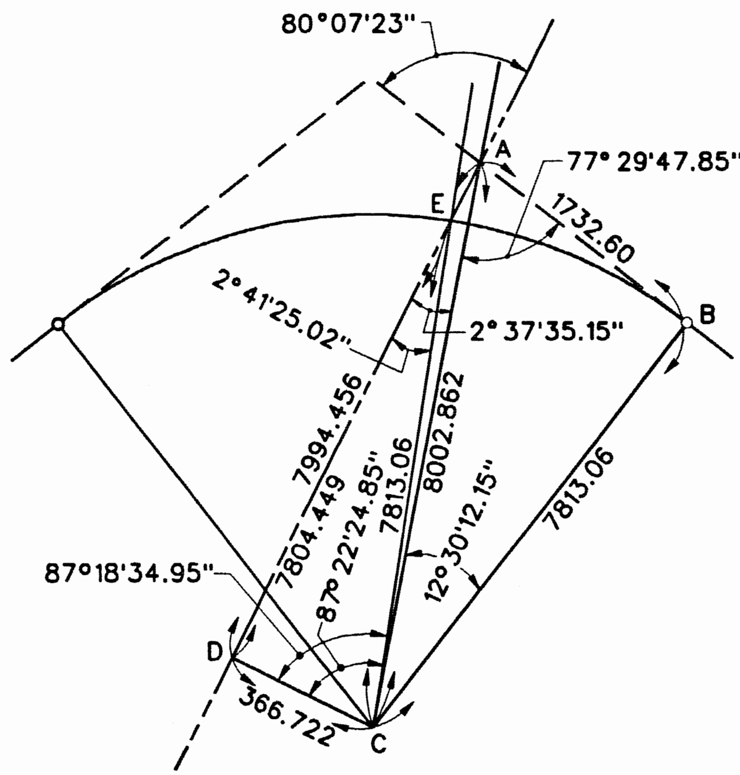
1. BC = Radius
2. AC = $\sqrt{(AB)^2 + (BC)^2}$
3. $\sin C_3 = \dfrac{AB}{AC}$
4. $A_1 = 90° - C_3$
5. $A_2 = A - A_1$
6. $CD = AC \sin A_2$
7. $AD = AC \cos A_2$
8. EC = Radius
9. $\sin E_1 = \dfrac{CD}{EC}$
10. $DE = EC \cos E_1$
11. $C_1 = 90° - A_2$
12. $C_4 = 90° - E_1$
13. $C_2 = C_3 + C_1 - C_4$
14. AE = AD - DE
15. Arc Dist. BE = $\dfrac{C_2 \, x \, (100)}{Degree\ of\ Curve}$

POC COMPUTATION USING RIGHT TRIANGLES

Figure 6.6U

Given:

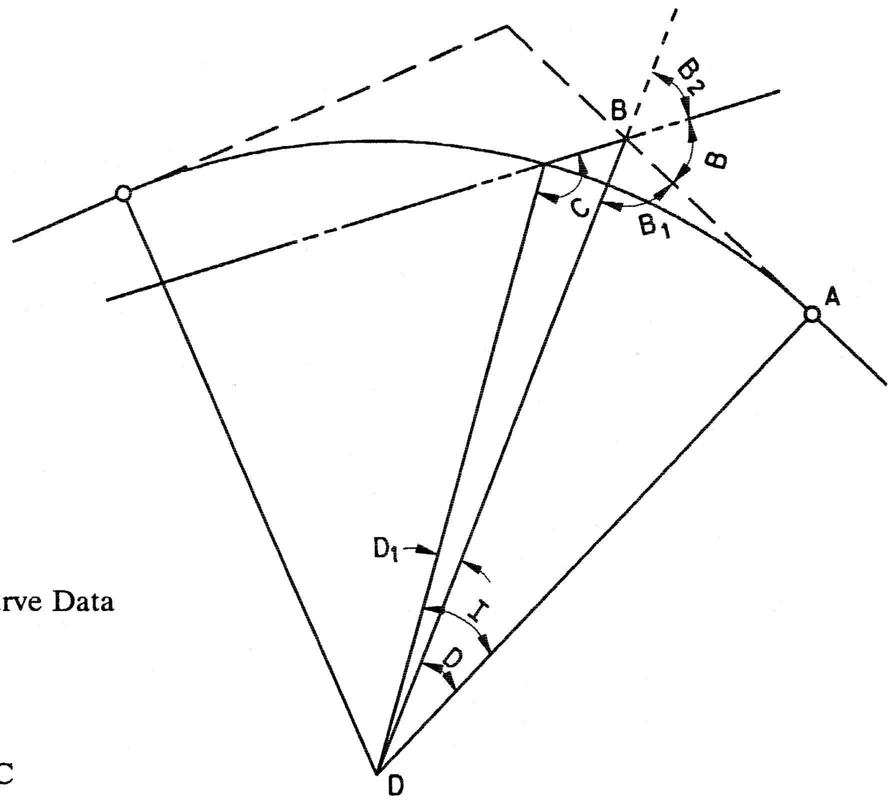
Angle A = 80°07′23″
AB = 1732.60
Radius = 7813.06
Curve D = 0°44′



1. BC = 7813.06
2. AC = $\sqrt{(7813.06)^2 + (1732.60)^2}$ = 8002.862
3. $\sin C_3 = \dfrac{1732.60}{8002.862} = .2164975 \qquad C_3 = 12°30'12.15''$
4. A_1 = 90°00′00″ - 12°30′12.15″ = 77°29′47.85″
5. A_2 = 80°07′23.00″ - 77°29′47.85″ = 2°37′35.15″
6. CD = (8002.862)(.04582381) = 366.722
7. AD = (8002.262)(.99894954) = 7994.456
8. EC = 7813.06
9. $\sin E_1 = \dfrac{366.722}{7813.06} = .04693705 \quad E_1 = 2°41'25.02''$
10. DE = (7813.06)(.99889785) = 7804.449
11. C_1 = 90°00′00″ - 2°37′35.15″ = 87°22′24.85″
12. C_4 = 90°00′00″ - 2°41′25.02″ = 87°18′34.98″
13. C_2 = 12°30′12.15″ + 87°22′24.85″ - 87°18′34.98″ = 12°34′02.02″
14. AE = 7994.456 - 7804.449 = 190.01
15. $Arc\ BE = \dfrac{12.567228° \ (100)}{.73333333°} = 1713.71$

POC COMPUTATION USING RIGHT TRIANGLES
(Sample Problem)

Figure 6.6V



Given:

AB, Angle B and Curve Data

Required:

BC and Arc Dist. AC

Solution:

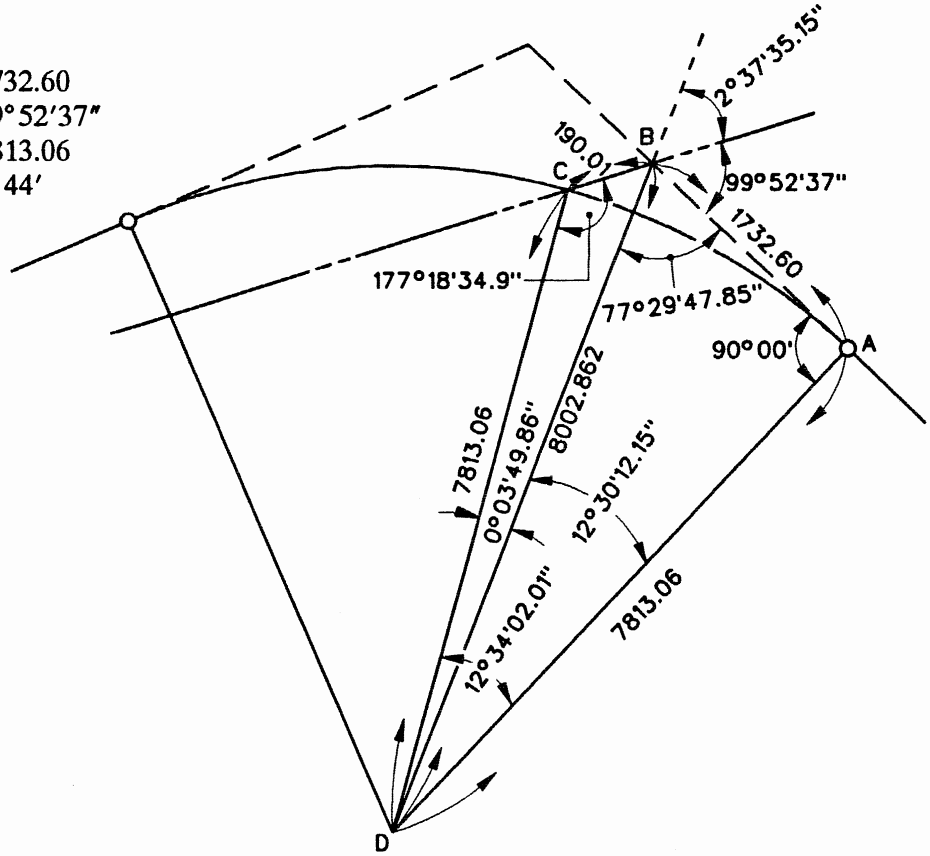
1. AD = Radius
2. BD = $\sqrt{(AB)^2 + (AD)^2}$
3. sin D = $\dfrac{AB}{BD}$
4. B_1 = 90°00′ - D
5. B_2 = 180°00′ - (B + B1)
6. sin C = $\dfrac{BD \sin B_2}{Radius}$
7. D_1 = 180° - (B2 + C)
8. BC = $\dfrac{Radius \sin D_1}{\sin B_2}$
9. I = D + D_1
10. Arc Dist. AC = $\dfrac{I \times 100}{\text{Degree of Curve}}$

POC COMPUTATION USING OBLIQUE TRIANGLE

Figure 6.6W

Given:

AB	=	1732.60
Angle B	=	99°52′37″
Radius	=	7813.06
Curve D	=	0°44′

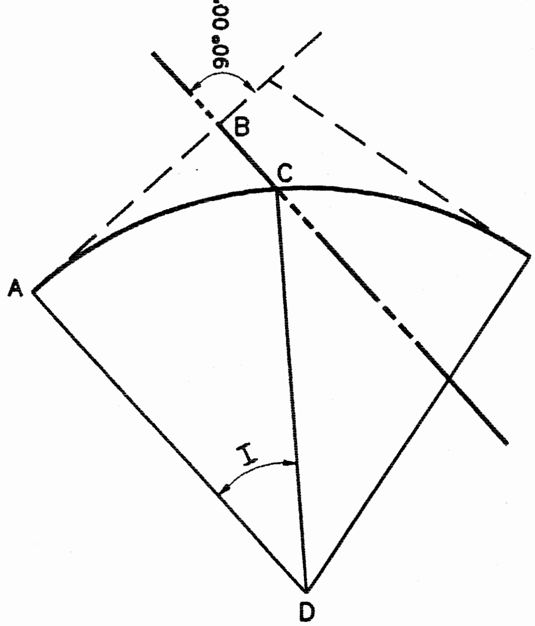


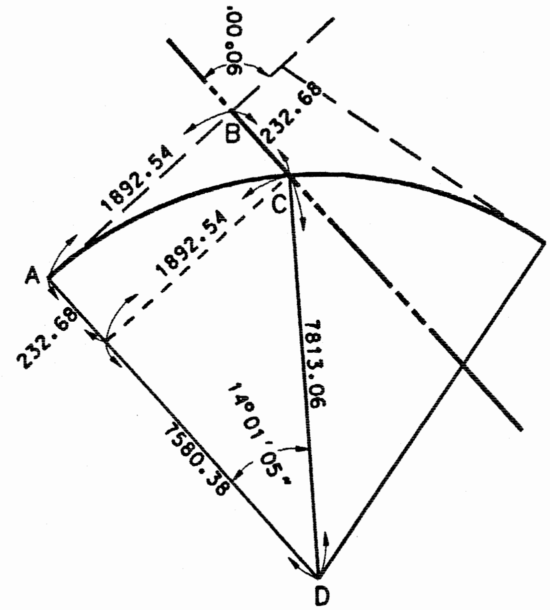
Solution:

1. AD = 7813.06
2. BD = $\sqrt{(1732.60)^2 + (7813.06)^2} = 8002.862$
3. sin D = $\dfrac{1732.60}{8002.862} = .2164975 \qquad D = 12°\,30'\,12.15''$
4. B_1 = 90°00′ - 12°30′12.15″ = 77°29′47.85″
5. B_2 = 180°00′ - (99°52′37″ + 77°29′47.85″) = 2°37′35.15″
6. sin C = $\dfrac{(8002.862)(.04582381)}{7813.06} = .0469370 \qquad C = 177°\,18'\,34.99''$
7. D_1 = 180°00′ - (2°37′35.15″ + 177° 18′34.99″) = 0°03′49.86″
8. BC = $\dfrac{(7813.06)(.00111439)}{.04582381} = 190.01$
9. I = 12°30′12.15″ = 0°03′49.86″ = 12°34′02.01″
10. Arc AC = $\dfrac{12.567225°}{.73333333°} \times 100 = 1713.71$

POC COMPUTATION USING OBLIQUE TRIANGLE (Sample Problem)

Figure 6.6X





<u>Given</u>:

AB and Curve Data.

<u>Required</u>:

BC and Arc Dist. AC.

<u>Solution</u>:

1. CD = Radius

2. $\sin I = \dfrac{AB}{CD}$

3. $BC = CD - \sqrt{(CD)^2 - (AB)^2}$

4. $\text{Arc AC} = \dfrac{I \times 100}{\text{Degree of Curve}}$

<u>EXAMPLE</u>

<u>Given</u>:

AB = 1892.54
Radius = 7813.06
Curve D = 0°44′

<u>Solution</u>:

1. CD = 7813.06

2. $\sin I = \dfrac{1892.54}{7813.06} = .2422277$

   $I = 14°01'05.00''$

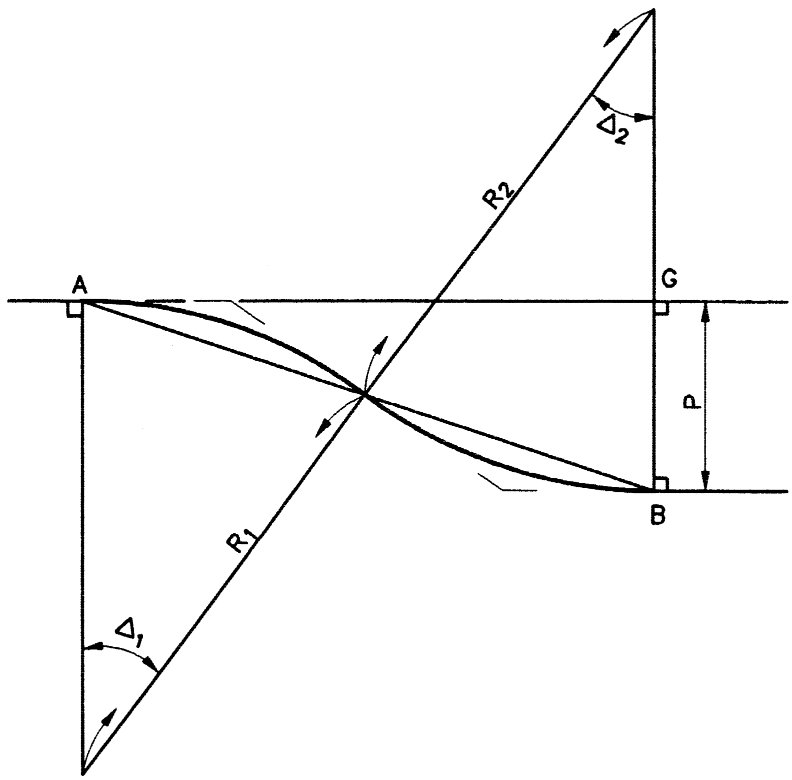
3. $BC = 7813.06 - \sqrt{(7813.06)^2 - (1892.54)^2}$

   $= 7813.06 - 7580.38 = 232.68$

4. $\text{Arc AC} = \dfrac{14.018056°}{.73333333°} \times 100 = 1911.55$

POC OF LINE 90° TO CURVE TANGENT

Figure 6.6Y



EQUAL RADII

Given: Radius & BG

1. $R_1 = R_2 \quad \Delta_1 = \Delta_2$
2. $BG = P$
3. $\cos \Delta_1 = \dfrac{R_1 - \frac{1}{2}P}{R_1}$
4. $AG = \sqrt{4PR_1 - P^2}$
5. $\sin \Delta_1 = \dfrac{AG}{R_1 + R_2}$
6. $\tan \Delta_1 = \dfrac{AG}{R_1 + R_2 - P}$

UNEQUAL RADII

Given: R1, AG & P

1. $\Delta_1 = \Delta_2$
2. $AB = \sqrt{AG^2 + P^2}$
3. $R_2 = \dfrac{AB^2}{2P} - R_1$
4. $\sin \Delta_1 = \dfrac{AG}{R_1 + R_2}$
5. $\cos \Delta_1 = \dfrac{R_1 + R_2 - P}{R_1 + R_2}$
6. $\tan \Delta_1 = \dfrac{AG}{R_1 + R_2 - P}$

REVERSED CURVES TO PARALLEL TANGENTS

Figure 6.6Z

EQUAL RADII

$R_1 = R_2 = 7813.06$
$P = 50.00$

1. $\cos \Delta_1 = \dfrac{7813.06 - 25.00}{7813.06} = .9968002$

   $\Delta_1 = \Delta_2 = 4°35'05.00''$

2. $AG = \sqrt{(4)(50.00)(7813.06) - (50.00)^2} = 1249.0444$

3. $\sin \Delta_1 = \dfrac{1249.0444}{7813.06 + 7813.06} = .07993311$

   $\Delta_1 = 4°35'05.00''$

4. $\tan \Delta_1 = \dfrac{1249.0444}{7813.06 + 7813.06 - 50.00} = .08018970$

   $\Delta_1 = 4°35'05.00''$

UNEQUAL RADII

$R_1 = 7813.06$
$P = 50.00$
$AG = 1000.00$

1. $AB = \sqrt{(1000.00)^2 + (50.00)^2} = 1001.249$

2. $R_2 = \dfrac{(1001.249)^2}{(2)(50.00)} - 7813.06 = 2211.94$

3. $\sin \Delta_1 = \dfrac{1000.00}{7813.06 + 2211.94} = .09975062$

   $\Delta_1 = \Delta_2 = 5°43'29.32''$

4. $\cos \Delta_1 = \dfrac{7813.06 + 2211.94 - 50.00}{7813.06 + 2211.94} = .99501246$

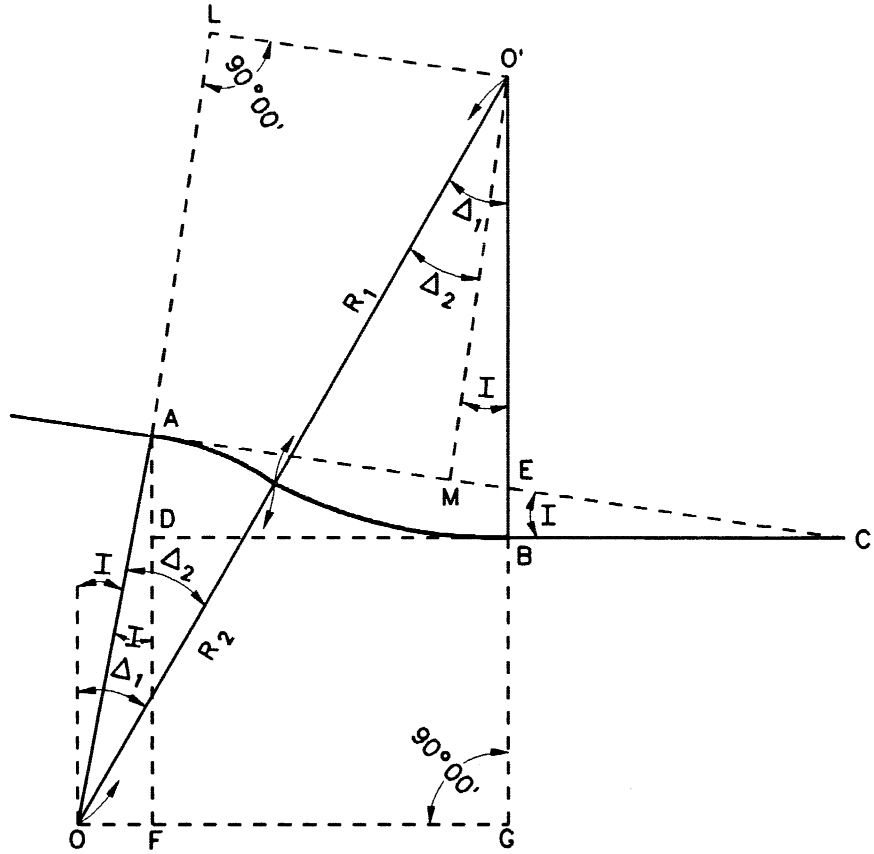
   $\Delta_1 = \Delta_2 = 5°43'29.32''$

5. $\tan \Delta_1 = \dfrac{1000.00}{7813.06 + 2211.94 - 50.00} = .1002506$

   $\Delta_1 = \Delta_2 = 5°43'29.32''$

REVERSED CURVES TO PARALLEL TANGENTS
(Sample Problem)

Figure 6.6AA



Given:
I, AD, R_1 and R_2

Required:
Δ_1 and Δ_2

1. AC $= \dfrac{AD}{\sin I}$

2. BG = DF = $R_2 \cos I$ - AD

3. $\cos \Delta_1 = \dfrac{R_1 + BG}{R_1 + R_2}$

4. $\Delta_2 = \Delta_1 - I$

EXAMPLE:

Given:

I	=	2° 13′16″
AD	=	53.17
R_1	=	17,188.73
R_2	=	21,485.92

1. AC $= \dfrac{53.17}{.03875599} = 1371.917$

2. BG = (21,485.92)(.99924870) - 53.17
= 21,416.61

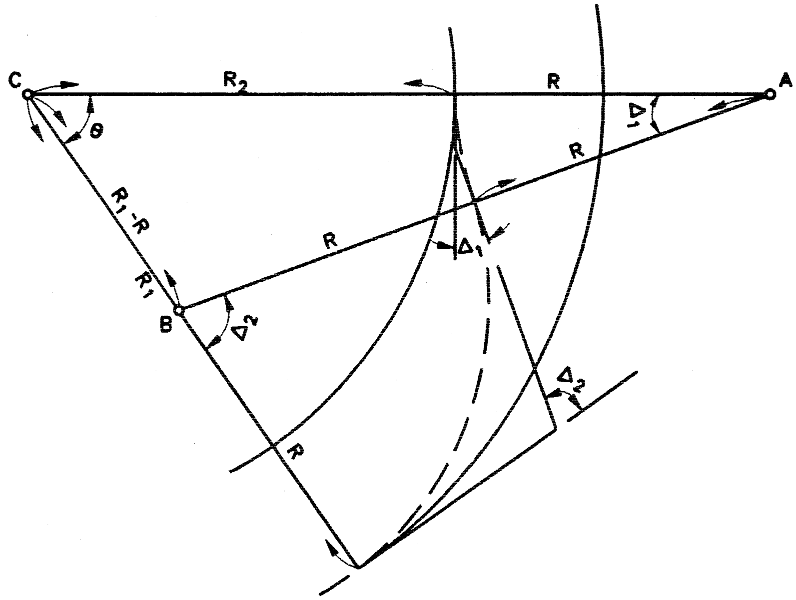
3. $\cos \Delta_1 = \dfrac{17{,}188.73 + 21{,}416.61}{17{,}188.73 + 21{,}485.92} =$
.9982078

Δ_1 = 3°25′51.00″

4. Δ_2 = 3°25′51.00″ - 2° 13′16.00″ =
1° 12′35.00″

REVERSED CURVES
(Tangents Not Parallel)

Figure 6.6BB



EQUATIONS:

1. $$\frac{\sin\Delta_1}{2} = \sqrt{\frac{(S-b)(S-c)}{bc}}$$

Where:
$a = R_1 - R$
$b = R_2 + R$
$c = 2R$
$s = \tfrac{1}{2}(a+b+c)$

2. $$\sin\theta = \frac{2R\sin\Delta_1}{R_1 - R}$$

3. $$\Delta_2 = \Delta_1 + \theta$$

Given:

R_2 = Inside Curve Radius
R_1 = Outside Curve Radius
R = Equal Radii of Reverse Curve

Required:

Δ_1 & Δ_2

EXAMPLE:

R_2 = 10,692.96, R_1 = 10,792.96, R = 1909.86

a = 10,792.96 - 1909.86 = 8,883.10
b = 10,692.96 + 1909.86 = 12,602.82
c = (2)(1909.86) = 3,819.72
s = ½(8,883.10 + 12,602.82 + 3,819.72) = 12,652.82

$$\frac{\sin\Delta_1}{2} = \sqrt{\frac{(50.00)(8{,}883.10)}{(12{,}602.82)(3{,}819.72)}} = \sqrt{\frac{441{,}655.00}{48{,}139{,}243.6104}}$$
$= 0.09578377$

$$\frac{\Delta_1}{2} = 5°29'47.155'',\ \Delta_1 = 10°59'34.31''$$

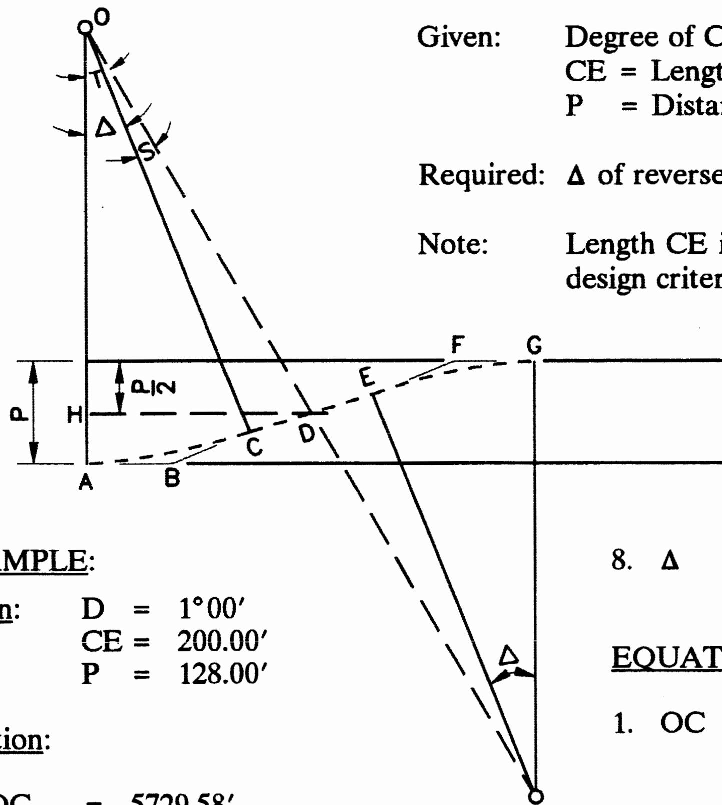
$$\sin\theta = \frac{2(1909.86)(0.19068675)}{10{,}792.96 - 1909.86} = \frac{728.36996}{8883.10}$$
$= 0.08199502$

$\theta = 4°42'11.70''$

$\Delta_2 = 10°59'34.31'' + 4°42'11.70'' = 15°41'46.01''$

REVERSED CURVES (Between Parallel Curves)

Figure 6.6CC



Given: Degree of Curvature of Reversed Curve
CE = Length of tangent between curves
P = Distance between parallel tangents

Required: Δ of reversed curves

Note: Length CE is governed by superelevation runoff design criteria.

EXAMPLE:

Given:
D = 1°00′
CE = 200.00′
P = 128.00′

Solution:

1. OC = 5729.58′

2. CD $= \dfrac{200.00}{2} = 100.00'$

3. OH = 5729.58 - 64.00 = 5665.58′

4. OD $= \sqrt{(100)^2 + (5729.58)^2}$
$= \sqrt{32{,}838{,}086.9764} = 5730.4526$

5. sin S $= \dfrac{100.00}{5730.4526} = 0.01745062$
S = 0°59′59.63″

6. HD $= \sqrt{(5730.4526)^2 - (5665.58)^2}$
$= \sqrt{739{,}290.2634} = 859.8198$

7. sin T $= \dfrac{859.8198}{5730.4526} = 0.15004395$
T = 8°37′46.10″

8. Δ = 8°37′46.10″ - 0°59′59.63″ = 7°37′46.47″

EQUATIONS:

1. OC = Radius for given degree of curvature

2. CD $= \dfrac{CE}{2}$

3. OH $= OC - \dfrac{P}{2}$

4. OD $= \sqrt{CD^2 + OC^2}$

5. sin S $= \dfrac{CD}{OD}$

6. HD $= \sqrt{OD^2 - OH^2}$

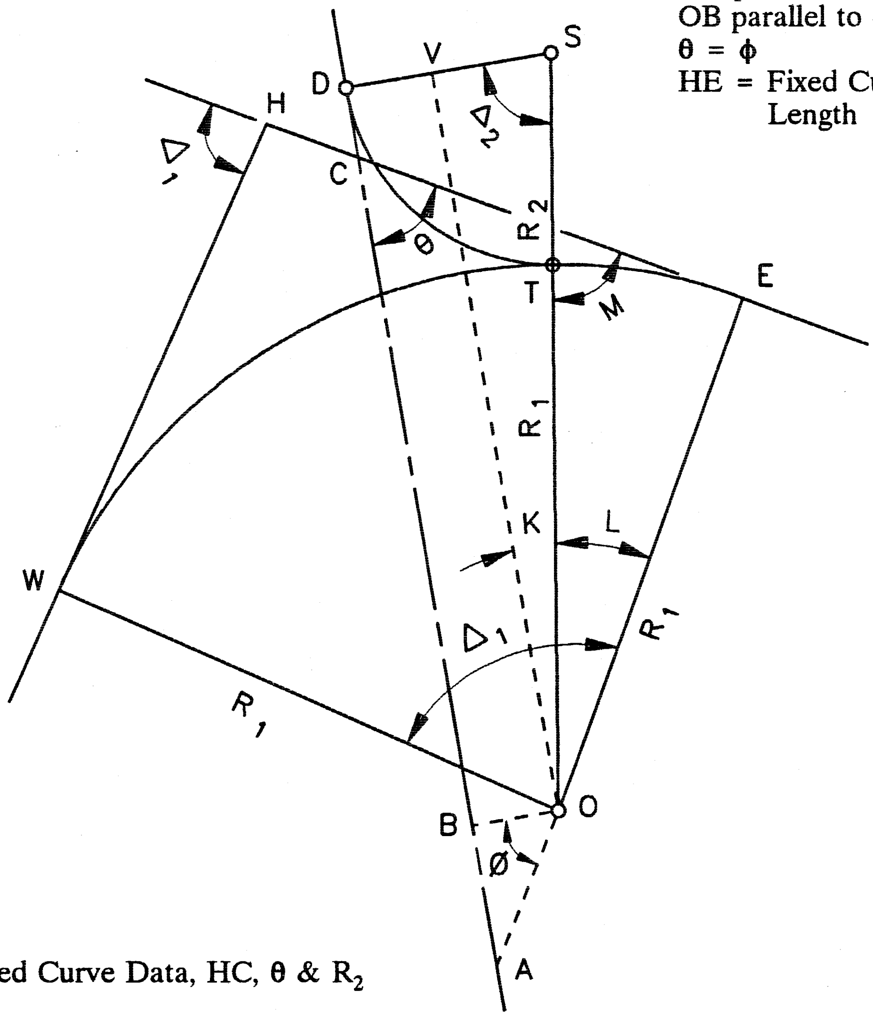
7. sin T $= \dfrac{HD}{OD}$

8. Δ = T - S

REVERSED CURVES
(To Parallel Tangents with Tangent Between)

Figure 6.6DD

OV parallel to BD &
OB parallel to SD
$\theta = \phi$
HE = Fixed Curve Tangent Length



Given:

Fixed Curve Data, HC, θ & R_2

Note:

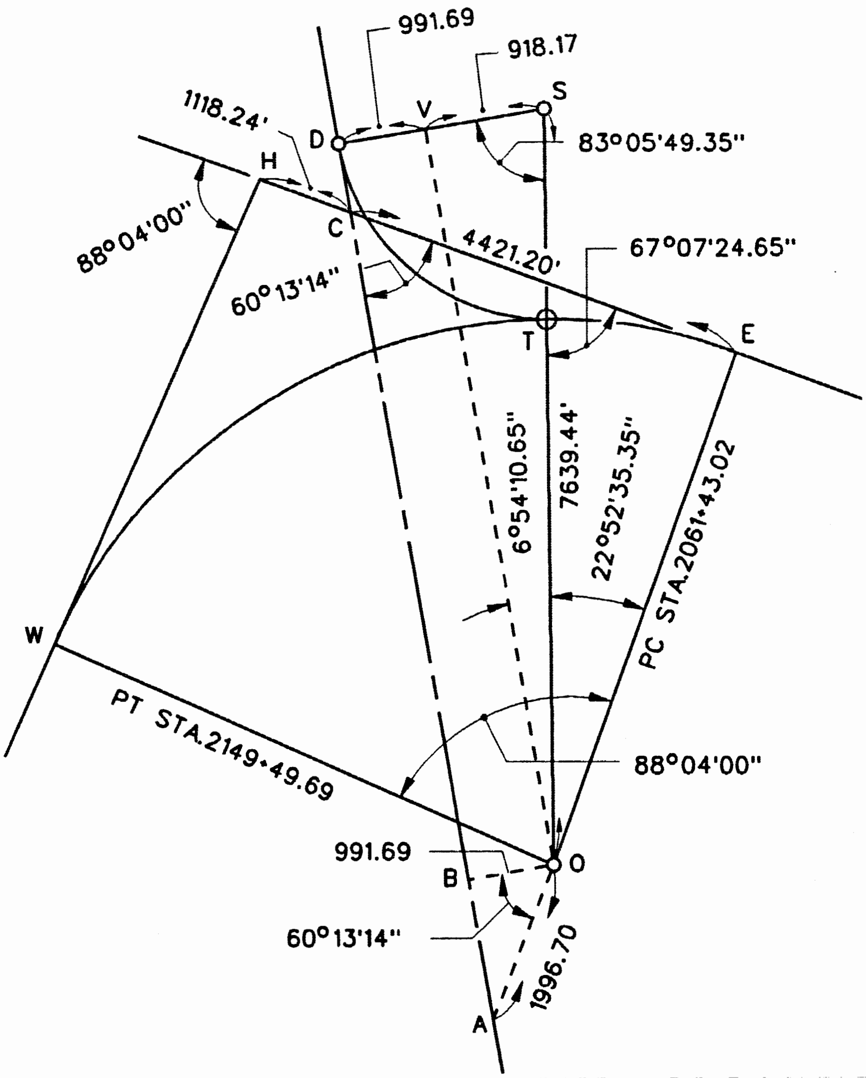
θ must be less than 90° for this solution.
WE = fixed curve

Equations:

1. $CE = HE - HC$
2. $OA = CE \tan\theta - R_1$
3. $OB = DV = OA \cos\phi$
4. $SV = R_2 - DV$
5. $\sin K = \dfrac{SV}{R_1 + R_2}$
6. $\Delta_2 = 90° - K$
7. $M = \theta + K$
8. $L = 90° - M$
9. $\dfrac{L \times 100}{\textit{Degree of Curve}} = \textit{Arc Length ET}$

CURVE BETWEEN FIXED TANGENT AND FIXED CURVE
(Case 1)

Figure 6.6EE



PI Sta. 2116+82.46
Δ_1 = 88°04'00"
D = 1°00'
T = 5539.44
L = 8806.67
R_1 = 5729.58
E = 2239.95

HC = 1118.24
θ = 60°13'14"
R_2 = 1909.86'

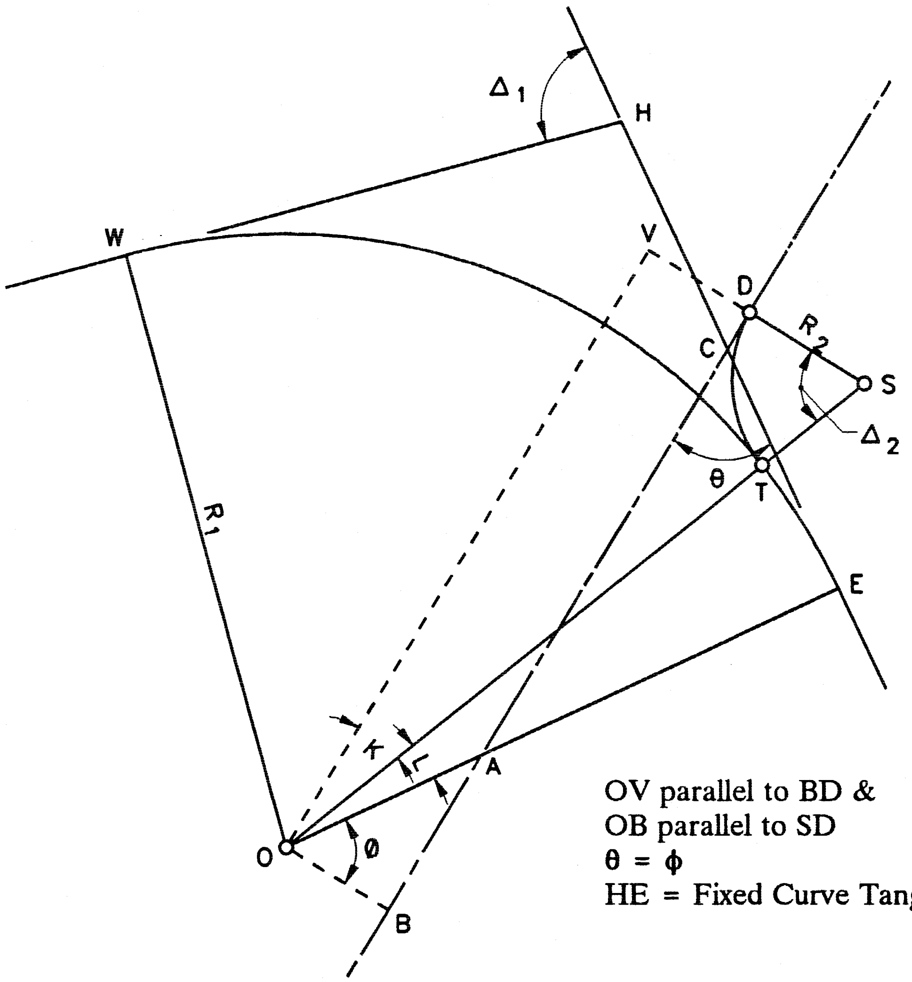
1. CE = 8806.67 - 1118.24 = 7688.43
2. OA = (4421.20)(1.7475519) - 5729.58 = 1996.70
3. OB = DV = (1996.70)(0.49666261) = 991.69
4. SV = 1909.86 - 991.69 = 918.17
5. $\sin K = \dfrac{918.17}{7639.44} = 0.12018812$

   K = 6°54'10.65"
6. Δ_2 = 90° - 6°54'10.65" = 83°05'49.35"
7. M = 60°13'14.00" + 6°54'10.65" = 67°07'24.65"
8. L = 90° - 67°07'24.65" = 22°52'35.35"

$$\frac{L \times 100}{\text{Degree of Curve}} = \text{Arc Length ET}$$

CURVE BETWEEN FIXED TANGENT AND FIXED CURVE (CASE 1)
(Sample Problem)

Figure 6.6FF



OV parallel to BD &
OB parallel to SD
$\theta = \phi$
HE = Fixed Curve Tangent Length

<u>Given</u>:

Fixed Curve Data, HC, θ & R_2

<u>Note</u>:

θ must be less than 90° for this solution
WE = Fixed Curve

<u>Equations</u>:

1. CE = HE - HC
2. OA = R_1 - CE tan θ
3. OB = DV = OA cos ϕ
4. SV = R_2 + DV
5. $\sin K = \dfrac{SV}{R_1 + R_2}$
6. $\Delta_2 = 90° - K$
7. L = $90° - (K + \phi)$

$$\frac{L \times 100}{Degree\ of\ Curve} = Arc\ Length\ ET$$

CURVE BETWEEN FIXED TANGENT AND FIXED CURVE
(Case II)

Figure 6.6GG

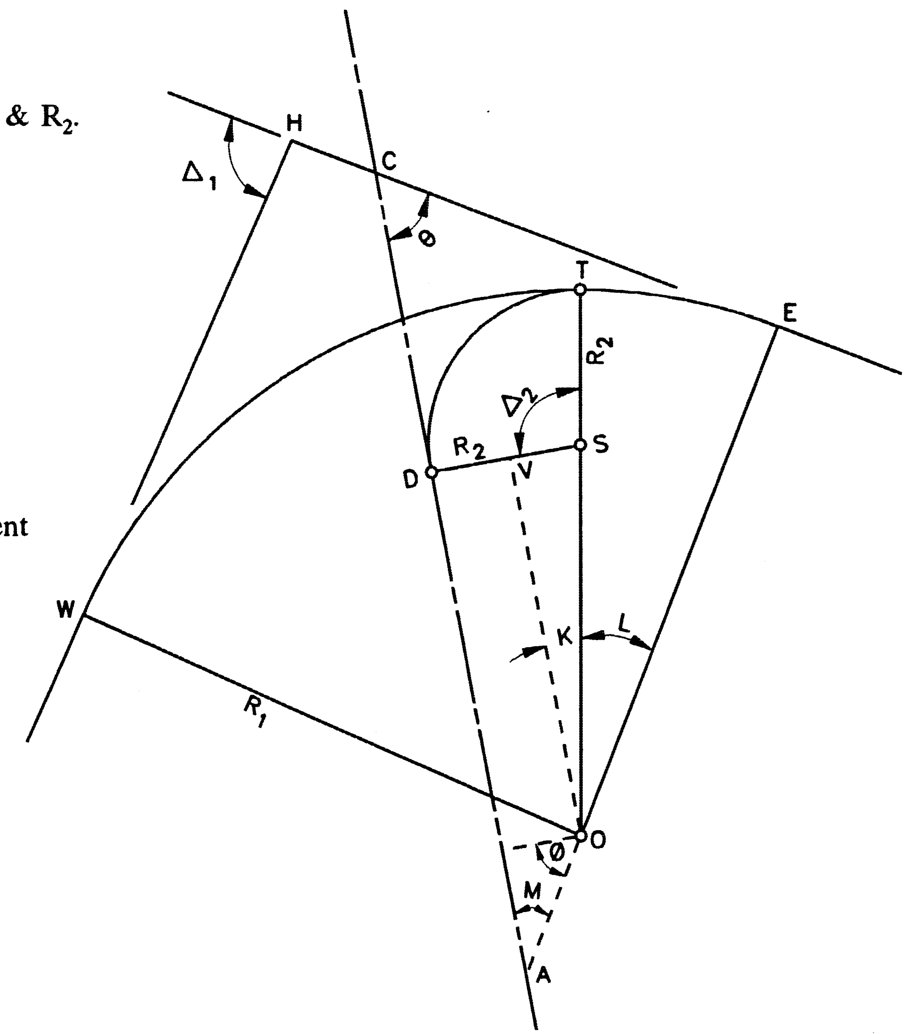
<u>Given</u>:

Fixed Curve Data, HC, θ & R_2.

<u>Note</u>:

θ must be less than 90°
WE = Fixed Curve

OV parallel to BC &
OB parallel to SD
$\theta = \phi$
HE = Fixed Curve Tangent Length



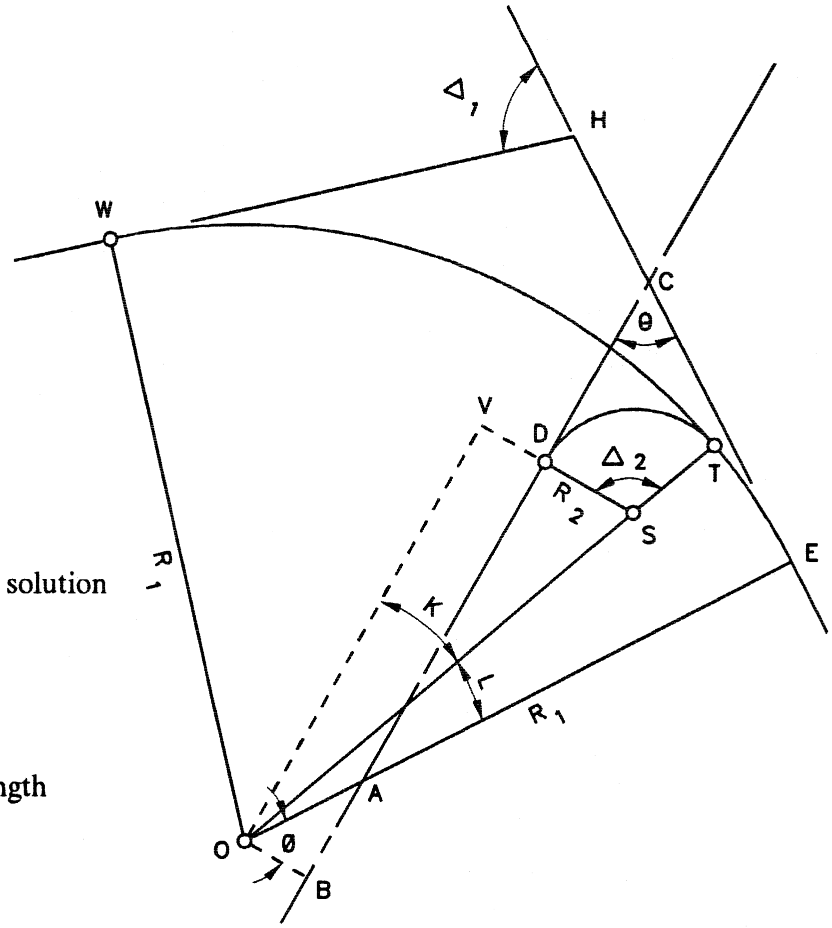
<u>Equations</u>:

1. CE = HE - HC
2. OA = CE tan θ - R_1
3. OB = DV = OA cos ϕ
4. SV = R_2 - DV
5. sin K = $\dfrac{SV}{R_1 - R_2}$
6. Δ_2 = 90° + K
7. M = 90° - ϕ
8. L = M - K

$$\frac{L \times 100}{\text{Degree of Curve}} = \text{Arc Length } ET$$

CURVE BETWEEN FIXED TANGENT & FIXED CURVE (Case III)

Figure 6.6HH



Given:

Fixed Curve Data, HC, θ & R_1

Note:

θ must be less than 90° for this solution
WE = Fixed Curve

OV parallel to BC &
OB parallel to SD
$\theta = \phi$
HE = Fixed Curve Tangent Length

Equations:

1. CE = HE - HC
2. OA = R_1 - CE tan θ
3. OB = DV = OA cos ϕ
4. SV = R_2 - DV
5. sin K = $\dfrac{SV}{R_1 - R_2}$
6. Δ_2 = 90° + K
7. L = 90° - (K + ϕ)

$$\frac{L \times 100}{Degree\ of\ Curve} = Arc\ Length\ ET$$

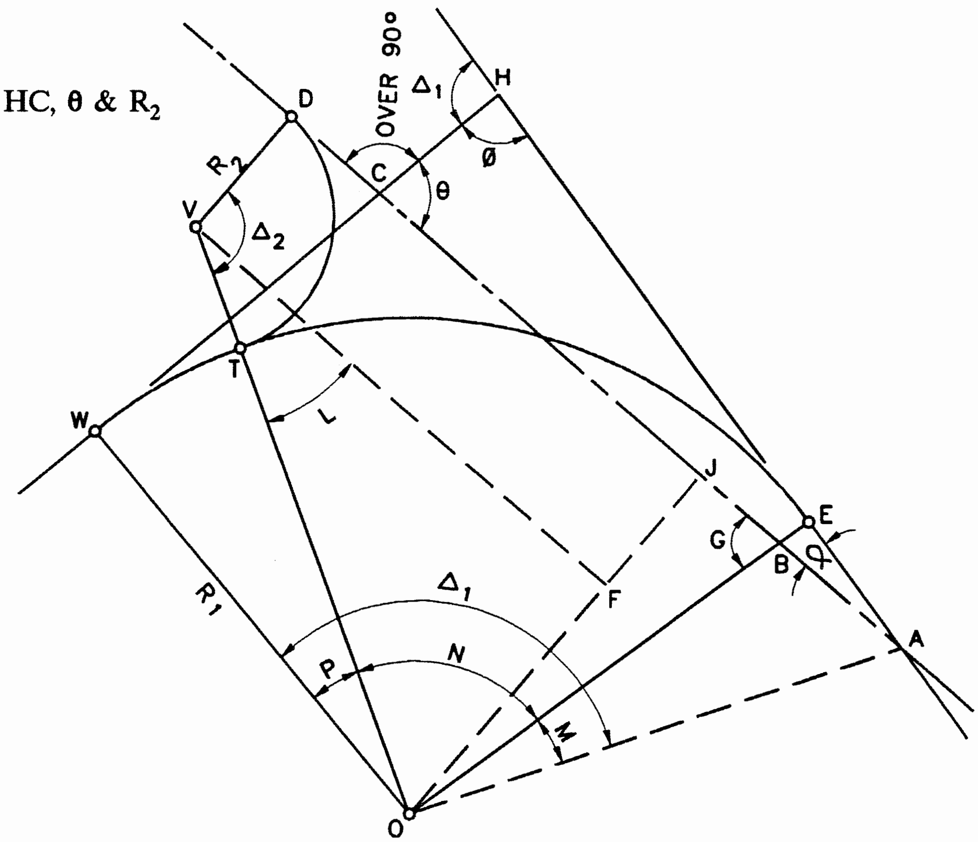
CURVE BETWEEN FIXED TANGENT & FIXED CURVE (CASE IV)

Figure 6.6II

Given:

"WE" Curve Data, HC, θ & R_2

OF parallel to VD
VF parallel to AD



Equations:

1. $\alpha = 180° - (\theta + \phi)$
2. $HA = \dfrac{HC \sin \theta}{\sin \alpha}$
3. $AE = HA - HE$
4. $BE = AE \tan \alpha$
5. $OB = R_1 - BE$
6. $OJ = OB \sin G$
7. $OF = OJ - R_2$
8. $\sin L = \dfrac{OF}{R_1 + R_2}$
9. $\Delta_2 = 90° + L$
10. $M = 90° - G$
11. $N = 90° - L$
12. $P = \Delta_1 - M - N$

$$Arc\ Length\ WT = \frac{P \times 100}{Degree\ of\ fixed\ curve\ (WE)}$$

CURVE BETWEEN FIXED TANGENT & FIXED CURVE
(Case V)

Figure 6.6JJ

Given:

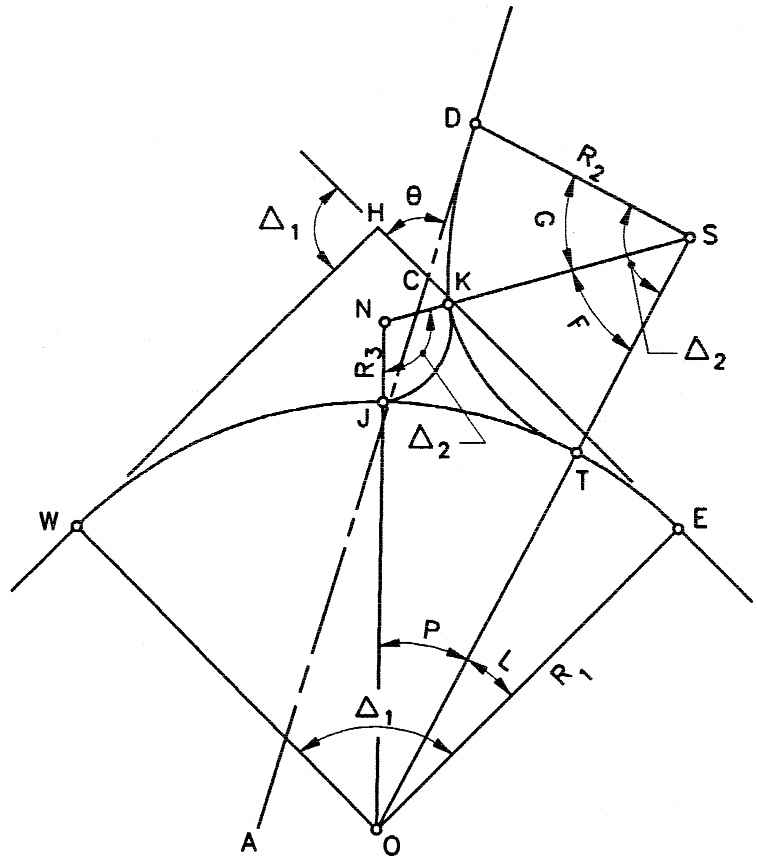
"WE" Curve Data, HC, θ, R_2, R_3

Note:

θ must be less than 90°

AD = Fixed Tangent
WE = Fixed Curve

Δ_2 & L computed according to example in Case I



Equations:

1. $\dfrac{\sin \Delta_2}{2} = \sqrt{\dfrac{(s-b)(s-c)}{bc}}$

2. $\sin P = \dfrac{(R_1 + R_2) \sin \Delta_2}{R_1 + R_3}$

3. $\sin F = \dfrac{(R_1 + R_3) \sin \Delta_2}{R_1 + R_3}$

4. $\textit{Arc Length EJ} = \dfrac{(L + P) \times 100}{\textit{Degree of Curve with Arc EJ}}$

5. $G = \Delta_2 - F$

6. $\textit{Arc Length DK} = \dfrac{G \times 100}{\textit{Degree of Curve with Arc DK}}$

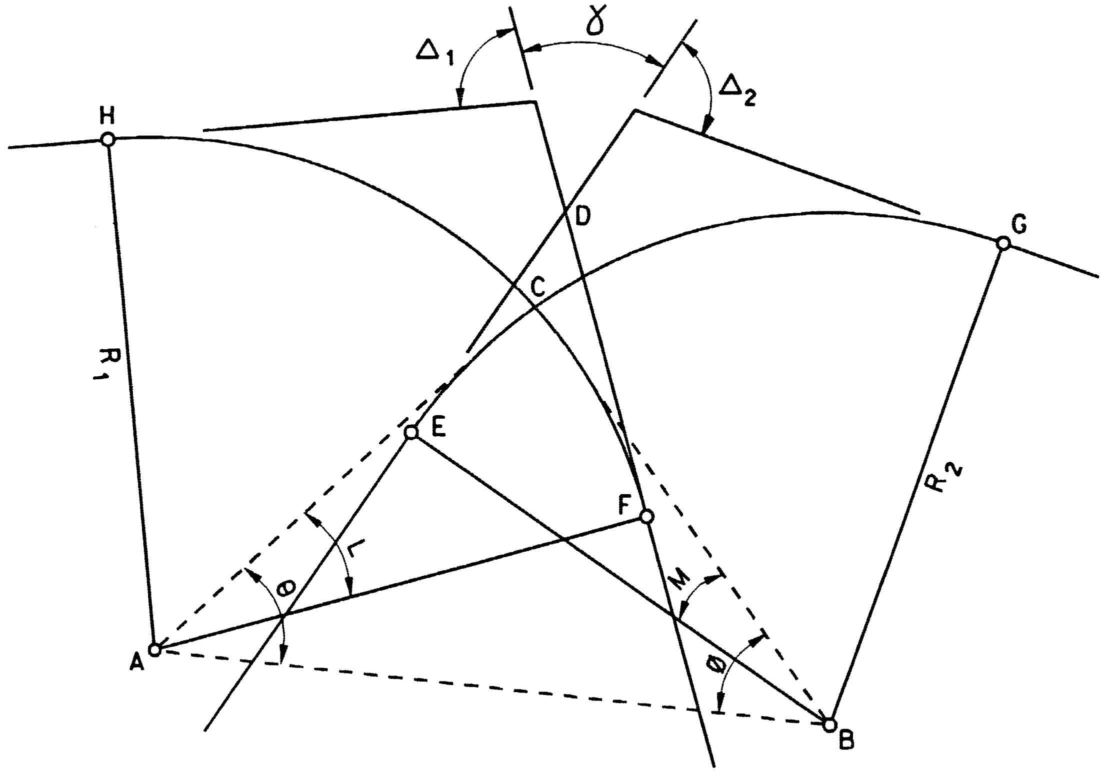
7. $\textit{Arc Length KJ} = \dfrac{\Delta_2 \times 100}{\textit{Degree of Curve with Arc KJ}}$

Where:

a = OS = $R_1 + R_2$
b = NS = $R_2 + R_3$
c = ON = $R_1 + R_3$
s = ½ (a + b + c)

THREE CURVES TANGENT TO EACH OTHER

Figure 6.6KK



Given:

Both curve data, azimuth or bearing of curve tangents, DE, DF & ∝.

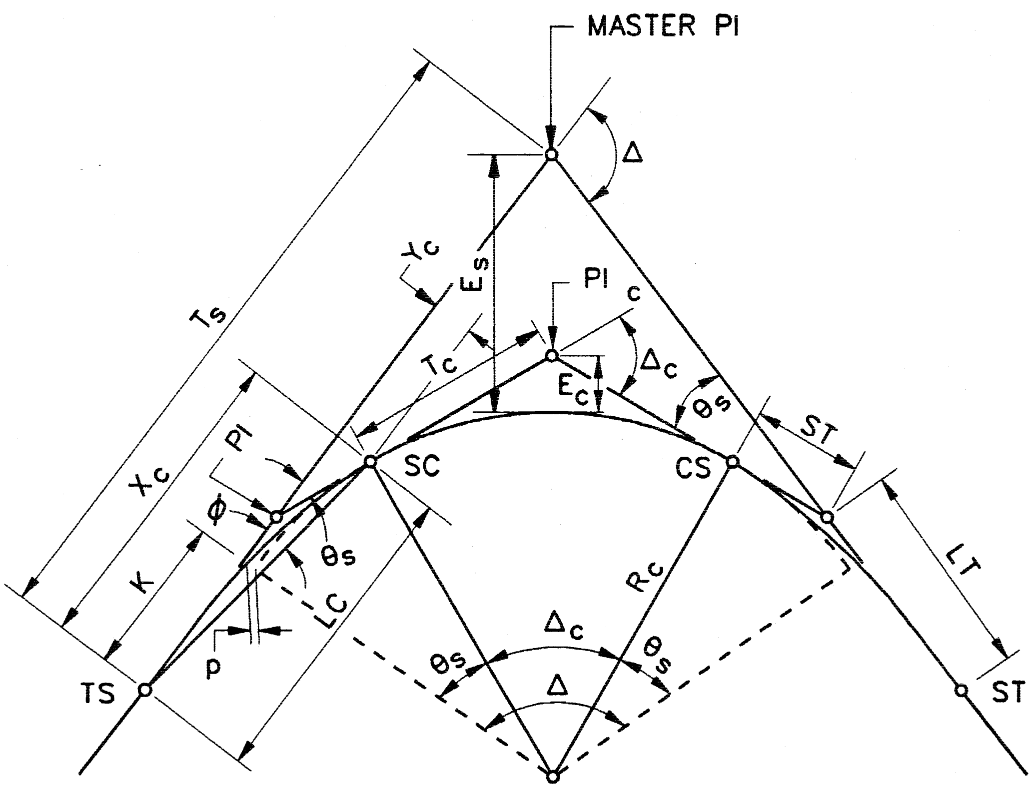
Required:

Arc EC and Arc FC

1. Coordinates at D either given or assume grid system
2. Determine coordinates at centers of curves (A & B)
3. Determine length and bearing AB
4. BC = R_2 = a, AC = R_1 = b, AB = c
5. S = ½ (a+b+c)
6. $\frac{\sin\theta}{2} = \sqrt{\frac{(s-b)(s-c)}{bc}}$
7. $\sin\phi = \frac{R_1 \sin\theta}{R_2}$
8. Determine bearings of AC & BC
9. Determine L & M from bearings
10. $Arc\ EC = \frac{M\ x\ 100}{Degree\ of\ Curve\ EG}$
11. $Arc\ FC = \frac{L\ x\ 100}{Degree\ of\ Curve\ FH}$

INTERSECTION OF TWO CURVES

Figure 6.6LL



Note:

All highway spirals to be computed from data shown in *Transition Curves for Highways* by Public Roads Administration (Joseph Barnett).

All railroads in Oklahoma except M.K. & T. R.R. use 10-chord spiral definition. M.K. & T. R.R. uses Searles spiral. For additional information, contact the Rail Planning Branch within the Planning Division.

PI_c to circular curve must be set on External (E_s) and not from spiral PI (PI_s) and spiral end point (SC or CS).

See next page for definition of terms.

SPIRALED CURVED

Figure 6.6MM

SPIRAL TRANSITION CURVE NOMENCLATURE

Master PI = Point of intersection of the main tangents.

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.

R_c = Radius of the circular curve.

L_s = Length of spiral.

T_s = Tangent distance PI to TS or ST, or tangent distance of completed curve.

E_s = External distance Master PI to midpoint of circular curve portion.

LT = Long tangent of spiral only.

ST = Short tangent of spiral only.

LC = Long chord of spiral.

P = Offset distance from the main tangent to the PC or PT of the circular curve produced.

K = Distance from TS to point on main tangent opposite the PC of the circular curve produced.

Δ = Intersection angle between tangents of the entire curve.

Δ_c = Intersection angle between tangents at the SC and the CS or the central angel of the circular curve portion of the curve.

θ_s = Intersection angle between the tangent of the complete curve and the tangent at the SC, the spiral tangents intersection angle.

D_c = Degree of the circular curve. Same as degree of the spiral at the SC (arc definition).

ϕ = Deflection angle from tangent at TS to SC.

X_cY_c = Coordinates of SC from the TS.

SPIRALED CURVE
(Definitions)

Figure 6.6NN

1. P is obtained from *Transition Curves for Highways* Book.

2. $E_s = (R_c + P)\,(1/\cos(\Delta/2) - 1) + P =$

$$\left[\frac{(R_c + P)}{\cos(\Delta/2)} - (R_c + P)\right] + P$$

3. $\theta_s = \dfrac{L_s}{200} D_c$

4. $\phi = \dfrac{\theta}{3}$,

   If $\theta_s < 15°00'$.

5. $\phi = \dfrac{\theta}{3} - C$,

   If $\theta_s \geq 15°00'$.

Corrections for C in Formula $\phi = \frac{\theta}{3} - C$								
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

6. $Y_c = LC \sin\phi$
7. $X_c = LC \cos\phi$
8. $K = X_c - R_c \sin\theta_s$
9. $ST = \dfrac{Y_c}{\sin\theta_s}$
10. $LT = X_c - \left(\dfrac{Y_c}{\tan\theta_s}\right)$
11. $LC = \dfrac{X_c}{\cos\phi}$
12. $T_s = (R_c + P)\tan(\Delta/2) + K$
13. $\theta = \dfrac{L^2}{L_s^2}\theta_s$
14. $L_c = \dfrac{(\Delta - 2\theta_s)\,x\,100}{D_c}$ = *Length of Circular Curve*
15. $\Delta_c = \Delta - 2\theta_s$
16. Deflection angle from TS or ST to any point "n" on Spiral.

$$\phi = \frac{\theta_s}{3}\left[\frac{L}{L_s}\right]^2$$

17. The exact values of ϕ by coordinates

$$\tan\phi = \frac{y}{x}$$

Note: All spiral functions should be taken from *Transition Curves for Highways* by Joseph Barnett for all highway computations.

Minimum Transition Lengths (ft)						
D_c	30 MPH L_s	40 MPH L_s	50 MPH L_s	60 MPH L_s	70 MPH L_s	D_c
1°30'	150	150	150	150	150	1°30'
2°00'	150	150	150	150	200	2°00'
2°30'	150	150	150	150	250	2°30'
3°00'	150	150	150	150	300	3°00'
3°30'	150	150	150	200	350	3°30'
4°00'	150	150	150	250	400	4°00'
5°00'	150	150	150	300		
6°00'	150	150	200	350		
7°00'	150	150	250			
8° to 9°	150	150	300			
10° to 12°	150	200				
13° to 14°	150	250				
15° to 23°	150					

Based on $L_s = \dfrac{1.6}{R_c} V^3$

Min L_s = 150 ft

Where V = 0.75 design speed in mph

SPIRAL FORMULAS

Figure 6.600

MASTER PI STA. 273+84.00

Δ = 76°00'00" Rt.
T_s = 1592.82
E_s = 514.89

SPIRAL DATA

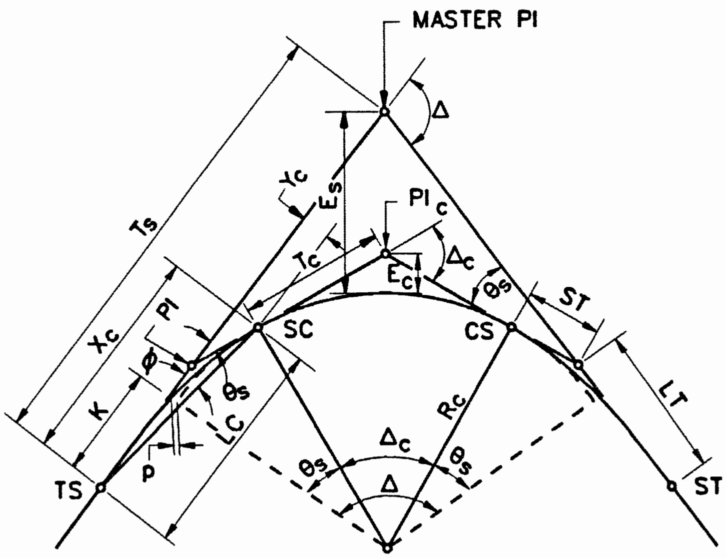
θ_s = 3°00'
LC = 200.00
LT = 133.35
ST = 66.68
X_c = 199.95
Y_c = 3.49

CIRCULAR CURVE DATA

PI Sta. 278+28.49
Δ_c = 70°00'00" Rt.
D_c = 3°00'
T_c = 1337.30
L_c = 2333.33
R_c = 1909.86
E_c = 421.65

Given:

PI = 273+84.00
D_c = 3°00'
Δ = 76°00' Rt.
L_s = 200.00



Solution:

1. p = 200' Spiral Table = 0.87 Unit Length Table (200) (.00435) = 0.87

2. $E_s = \left[\dfrac{(R_c + p)}{\cos(\Delta/2)} - (R_c + p)\right] + p = \left[\dfrac{(1909.86 + 0.87)}{.78801075} - (1909.86 + 0.87)\right] + 0.87 = 514.89$

3. $\theta_s = \dfrac{(L_s)\ D_c}{200} = \dfrac{(200)}{(200)}\ 3°\,00' = 3°\,00'$

4. $\phi = \dfrac{\theta_s}{3} = \dfrac{3°\,00'}{3} = 1°\,00'$

5. Y_c = 200' Spiral (Table V) = 3.49 Unit Length (Table II) = (200)(.01745) = 3.49
6. X_c = 200' Spiral (Table V) = 199.95 Unit Length (Table II) = (200)(.99973) = 199.95
7. K = 200' Spiral (Table V) = 99.99 Unit Length (Table II) = (200)(.49995) = 99.99
8. ST = 200' Spiral (Table V) = 66.68 Unit Length (Table II) = (200)(.33342) = 66.68
9. LT = 200' Spiral (Table V) = 133.35 Unit Length (Table II) = (200)(.66676) = 133.35
10. LC = 200' Spiral (Table V) = 199.98 Unit Length (Table II) = (200)(.99988) = 199.98
11. T_s = $(R_c + p)\tan(\Delta/2) + K = (1909.86 + 0.87)\tan 38°00' + 99.99 = (1910.73)(.78128564) + 99.99 = 1592.82$
12. Δ_c = $\Delta - 2\theta_s$ = 76°00' - 6°00' = 70°00' (All circular curve data computed as usual.)

SPIRAL EXAMPLE

Figure 6.6PP

Sometimes it is necessary to stake a line parallel to the centerline of the roadway. Generally, this is done by offset from the centerline. When this is impractical, the parallel line may be treated as a separate alignment with each circular curve concentric with the corresponding curve of the centerline with a radius equal to the radius of the centerline circular curve plus or minus the offset distance. The value of p is the same as that of the centerline and the PC of the circular curve produced is opposite that of the centerline, but the transition for a curve inside of the centerline must be shorter due to the increase in D_c, and the transition for a curve outside of the centerline must be longer due to the decrease in D_c. The length of transition may be found by trial, using the ratio of the value of D_c as a guide in choosing the length for first trial, as illustrated in the example.

Example:

Calculations for transition curve parallel to centerline with transition.

Given:

L_s = 300 ft, D_c = 9°, R = 636.62 ft
O_s = 13°30′ and p = 300 x 0.01960 (Table 11) = 5.88 ft.

Design a parallel line 30 ft inside the centerline.

Try L_s a little larger than $\frac{9.00}{9.45}$ x 300.

Try L_s = 290	O_s = 1.45 x 9.45° = 13.70°	p = 290 x 0.01989 = 5.76
Try L_s = 292	O_s = 1.46 x 9.45° = 13.80°	p = 292 x 0.02003 = 5.85
Try L_s = 293	O_s = 1.465 x 9.45° = 13.84°	p = 293 x 0.02009 = 5.89 O.K.

The TS of the offset curve is forward of the TS of the centerline curve a distance equal to the difference of the k distances. For this example:

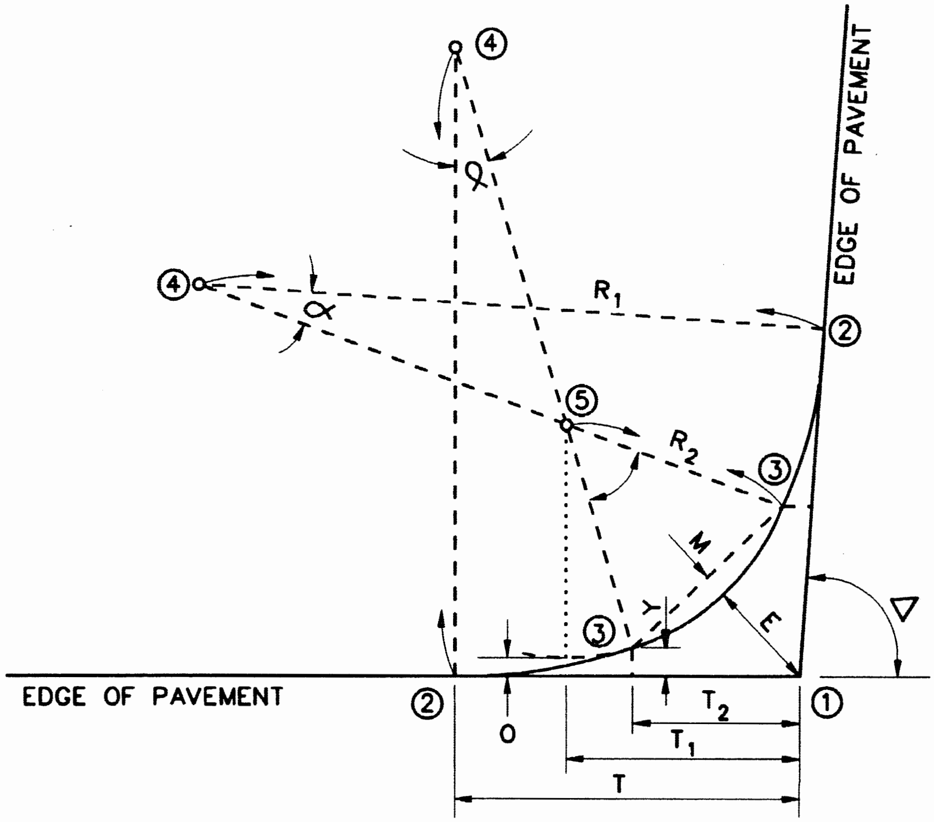
k for centerline	=	300 x 0.49908 (Table II)	= 149.72
k for offset line	=	293 x 0.49903	= 146.22

The resulting curve is very nearly but not exactly parallel to the centerline and is as smooth and graceful in appearance as the centerline with transition.

When pavements are widened on curves, the inside edge may be given transition by the method illustrated in the example except that the throw, p, is the sum of the p of the centerline and the widening of the pavement on the inside of the curve. This procedure results in a smooth curve without any break at the SC, but a transition much longer than the centerline transition is required. The central circular portion and the tangents leading to the centerline TS and ST must be long enough to permit the extra length of transition.

PARALLEL TRANSITIONS

Figure 6.6QQ



Given:

R_1 = 180′
R_2 = 65′
0 = 45′

The 3-centered curve is used to reduce the area of pavement required for longer, simple curves and yet provide sufficient space for the turning movement of the design vehicle.

The determination of the proper radii and offset (O) is based on the type and frequency of the design vehicle. This determination is a design rather than a technical function. See Chapter Nine.

Using the given information and the Tables - "3-Centered Curve," a State of Oklahoma publication, the curves are laid out as shown on the following page.

3-CENTERED CURVES

Figure 6.6RR

1. Intersect the edge of pavements as shown on the preceding detail (Point **(1)**).

2. Use the bearings (if given) or a protractor to determine Δ. From example, $\Delta = 86°30'$.

3. From page 52, 3-Centered Curve Tables, $T = 97.23'$, $T_2 = 47.37'$, $Y = 7.04'$, area (shaded portion) = 1394.51 sq ft.

4. Locate points **(2)** and points **(3)**.

5. Points **(4)** are perpendicular and R_1 distance from points **(2)**. Construct curve of R_1 radius from points **(2)** to **(3)** using points **(4)** as center of radius.

6. Connect points **(3)** and **(4)**, the intersection being point **(5)** (the center of the curve with radius R_2).

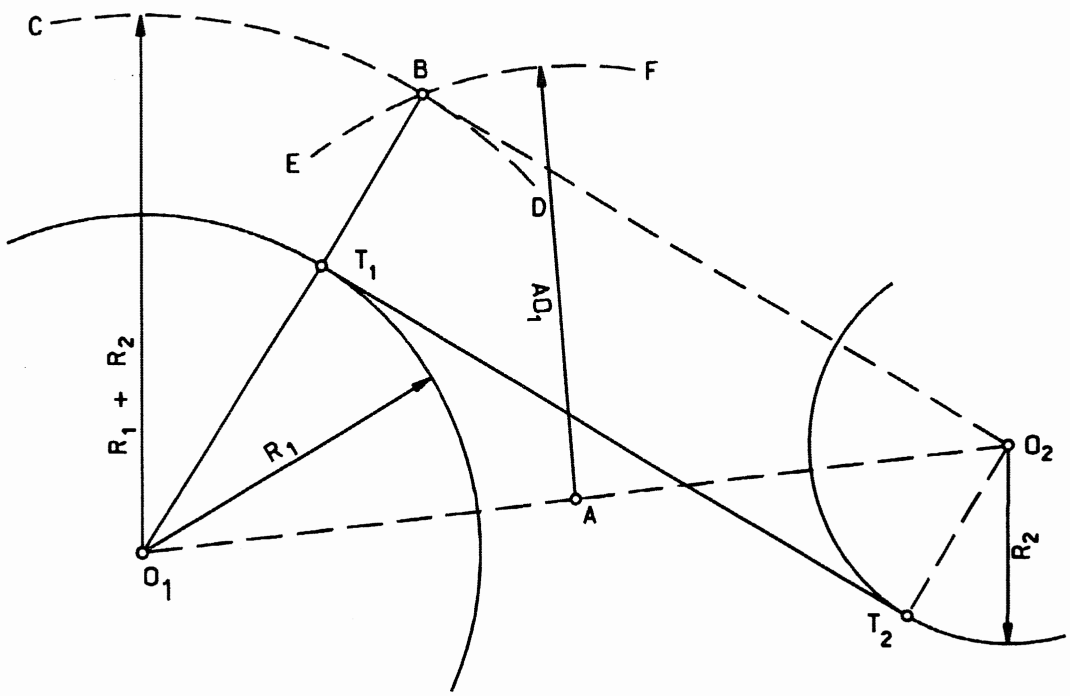
7. Construct a curve of R_2 radius which connects points **(3)**.

If the stations around the 3-centered curve are required, the curve formulas shown in Figure 6.6H are used. Knowing α (alpha) and the radius, the curve length can be readily computed. Note: $\theta = \Delta - 2\alpha$.

3-CENTERED CURVE EXAMPLE

Figure 6.6SS

Problem: To construct a line tangent to the inside of two curves of known center and radii.



Given:

Two curves with centers at O_1 and O_2 and radii of R_1 and R_2 respectively.

Solution:

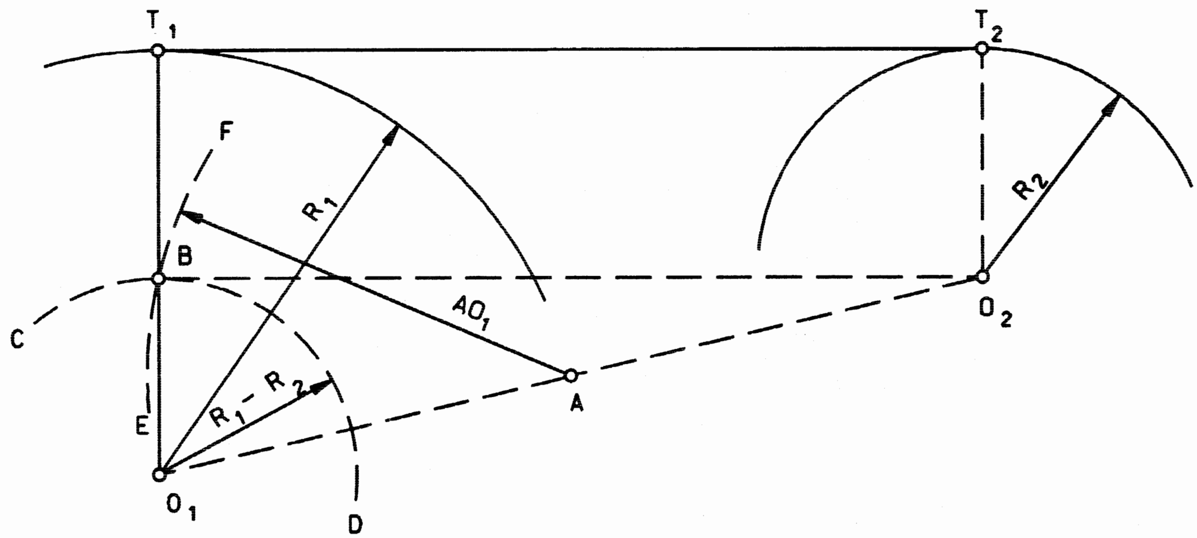
1. Join O_1 and O_2 and bisect this line to produce Point A. $AO_1 = AO_2$.
2. From Point O_1, swing an arc CD with a radius $R_1 + R_2$.
3. From Point A, swing an arc EF with a radius AO_1 until it intersects arc CD at Point B.
4. Join O_1 and B to produce point T_1.
5. $T_1 T_2$, the desired tangent line, is equal and parallel to BO_2; both of which are perpendicular to BT_1 and $O_2 T_2$ which are equal to R_2.

This geometric construction should delineate the ease of computing the necessary distances and angles by trigonometry.

COMPASS TANGENT CONSTRUCTION

Figure 6.6TT

Problem: To construct a line tangent to the inside of two curves of known center and radii.



Given:

Two curves with centers at O_1 and O_2 and radii of R_1 and R_2 respectively.

Solution:

1. Join O_1 and O_2 and bisect this line to produce Point A. $AO_1 = AO_2$.
2. From Point O_1, swing an arc CD with a radius R_1 - R_2.
3. From Point A, swing an arc EF with a radius AO_1 until it intersects arc CD at point B.
4. Join O_1 and B and extend this line to produce Point T_1.
5. T_1 T_2, the desired tangent line, is equal and parallel to BO_2; both of which are perpendicular to BT_1 and O_2 T_2 which are equal to R_2.

This geometric construction should delineate the ease of computing the necessary distances and angles by trigonometry.

COMPASS TANGENT CONSTRUCTION

Figure 6.6UU

6.7 REFERENCES

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

2. *Bureau of Engineering Manual - Part E*, City of Los Angeles, September 1970.

3. *Transition Curves for Highways*, Public Roads Administration, 1940.

Chapter Seven

Vertical Alignment

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(Continued)

Chapter Seven

VERTICAL ALIGNMENT

Chapter Seven presents ODOT criteria for the design of vertical alignment elements. This includes grades, vertical curves, vertical clearances and truck-climbing lanes.

7.1 GRADES

7.1.1 Terrain (Definitions)

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

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

   In general, rolling terrain generates steeper grades, causing trucks to reduce speeds below those of passenger cars.

3. Mountainous. Longitudinal and transverse changes in elevation are abrupt, and benching and side hill excavation are frequently required to provide the desirable highway alignment.

   Mountainous terrain aggravates the performance of trucks relative to passenger cars, resulting in some trucks operating at crawl speeds.

7.1.2 Maximum Grades

Chapters Twelve and Thirteen present ODOT criteria for maximum grades based on functional classification, urban/rural location, type of terrain, design speed and project scope of work. The maximum grades should be used only where absolutely necessary. Where practical, grades much flatter than maximum should be used.

7.1.3 Minimum Grades

1. Curbed Streets. The centerline profile on highways and streets with curb should have a minimum longitudinal gradient of 0.4%. Desirably, the minimum gradient will be 0.5%.

2. Uncurbed Roads. It is desirable to provide approximately a 0.5% longitudinal grade. This allows for the possibility that the original crown slope is subsequently altered as a result of swell, consolidation, maintenance operations or resurfacing. Level longitudinal gradients may be acceptable on pavements which are adequately crowned to drain laterally. If so, special ditches may be required.

7.1.4 Critical Length of Grade

Critical length of grade is the maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed. The highway gradient in combination with the length of grade will determine the truck speed reduction on upgrades. The following will apply for the critical length of grade:

1. Criteria. Figure 7.1A provides the critical length of grade for a given percent grade and acceptable truck speed reduction. The figure is based on an initial truck speed of 55 mph. For design purposes, the 10-mph speed reduction curve should be used. For entering speeds other than V = 55 mph, use Figures 7.5C or 7.5D.

2. Momentum Grades. Where an upgrade is preceded by a downgrade, trucks will often increase speed to make the climb. A speed increase of 5 mph on moderate downgrades (3-5%) and 10 mph on steeper downgrades (6-8%) of sufficient length are reasonable adjustments. These can be used in design to allow the use of a higher speed reduction curve in Figure 7.1A. However, the designer should consider that these speed increases may not be attainable if traffic volumes are high enough that a truck may be behind a passenger vehicle when descending the momentum grade.

3. Measurement. Vertical curves are part of the length of grade. Figure 7.1B illustrates how to measure the length of grade to determine the critical length of grade from Figure 7.1A.

4. Application. If the critical length of grade is exceeded, the designer should either flatten the grade, if practical, or should evaluate the need for a truck-climbing lane (see Section 7.5).

5. Highway Types. The critical length of grade criteria applies equally to two-lane or multilane highways and applies equally to urban and rural facilities.

6. Example Problems. Examples 7-1 and 7-2 illustrate the use of Figure 7.1A to determine the critical length of grade. Example 7-3 illustrates the use of both Figures 7.1A and 7.1B. In the examples, the use of subscripts 1, 2, etc., indicate the successive gradients and lengths of grade on the highway segment.

* * * * * * * * * *

Example 7-1

Given: Level Approach
$G = +4\%$
$L = 1200$ ft (length of grade)
Entering Speed = 55 mph

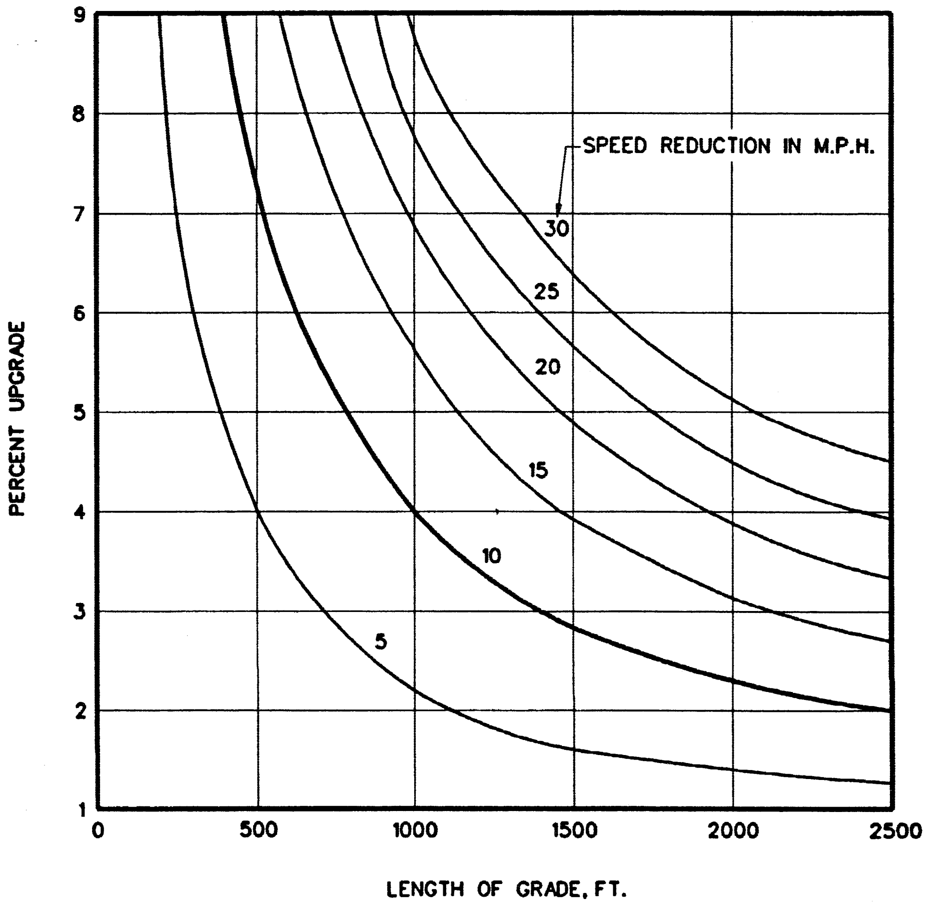
Problem: Determine if the critical length of grade is exceeded.

Solution: Figure 7.1A yields a critical length of grade of 1000 ft for a 10-mph speed reduction. The length of the grade (L) exceeds this value. Therefore, the designer should flatten the grade, if practical, or evaluate the need for a climbing lane.

Example 7-2

Given: Level Approach
$G_1 = +5\%$
$L_1 = 400$ ft
$G_2 = +2\%$
$L_2 = 600$ ft
Entering Speed = 55 mph

Problem: Determine if the critical length of grade is exceeded for the combination of grades G_1 and G_2.

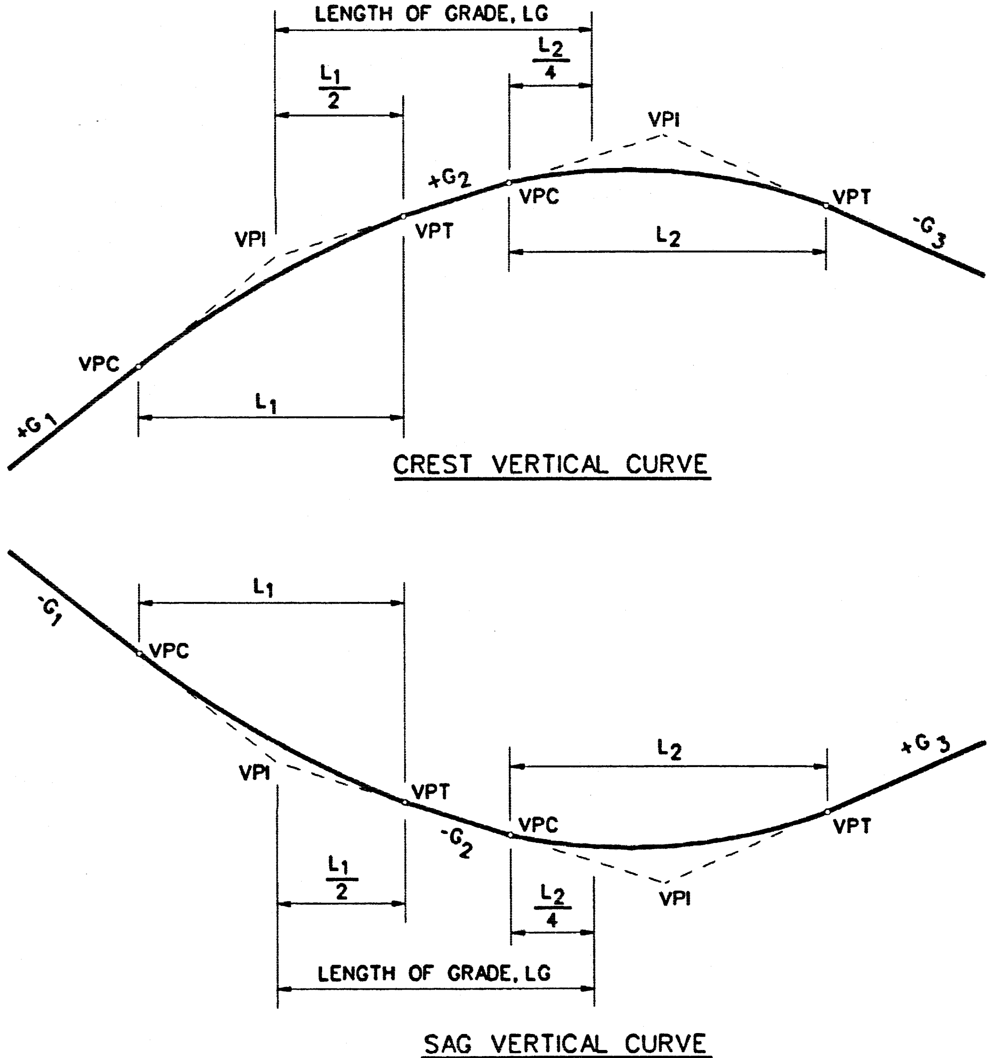


Source: (1)

Notes:
1. Typically, the 10-mph curve will be used.
2. See examples in Section 7.1 for use of figure.
3. Figure based on a 300-lb/hp truck with initial speed of 55 mph.

CRITICAL LENGTH OF GRADE

Figure 7.1A



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Notes:

1. For vertical curves where the two tangent grades are in the same direction (both grades + or both grades -), 50% of the curve length will be part of the length of grade.

2. For vertical curves where the two tangent grades are in opposite directions (one grade + and one grade -), 25% of the curve length will be part of the length of grade.

MEASUREMENT FOR LENGTH OF GRADE

Figure 7.1B

Solution: Using Figure 7.1A, G_1 yields a truck speed reduction of 5 mph. G_2 yields approximately 3 mph. The total of 8 mph is less than the allowable 10 mph. Therefore, the critical length of grade is not exceeded.

Example 7-3

Given: Figure 7.1C illustrates the vertical alignment on a low-volume, two-lane rural highway.

Problem: Determine if the critical length of grade is exceeded for G_2 or the combination upgrade G_3/G_4.

Solution: Figure 7.1B presents the criteria for determining the length of grade. These are calculated as follows for this example:

$$LG_2 = \frac{1000}{4} + 600 + \frac{850}{4} = 1063'$$

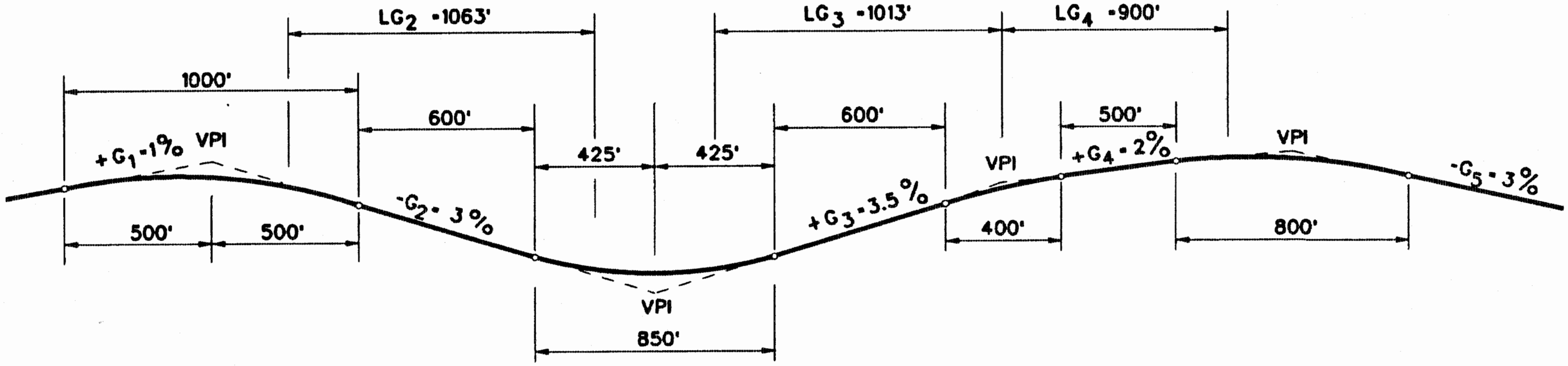
$$LG_3 = \frac{850}{4} + 600 + \frac{400}{2} = 1013'$$

$$LG_4 = \frac{400}{2} + 500 + \frac{800}{4} = 900'$$

Read into Figure 7.1A for G_2 (3%) and find a critical length of grade of 1400 ft. LG_2 is less than this value and, therefore, the critical length of grade is not exceeded.

Read into Figure 7.1A for G_3 (3.5%) and LG_3 = 1013′ and find a speed reduction of 9 mph. Read into Figure 7.1A for G_4 (2%) and LG_4 = 900′ and find a speed reduction of 4 mph. Therefore, the total speed reduction on the combination upgrade G_3/G_4 is 13 mph. However, for low-volume roads, the designer may assume a 5-mph increase in truck speed for the 3% "momentum" grade (G_2) which precedes G_3. Therefore, the speed reduction may be as high as 15 mph before the combination grade exceeds the critical length of grade. Assuming the benefits of the momentum grade leads to the conclusion that the critical length of grade is not exceeded.

* * * * * * * * * *



CRITICAL LENGTH OF GRADE CALCULATIONS
(Example 7-3)

Figure 7.1C

7.2 VERTICAL CURVES

7.2.1 Crest Vertical Curves

Crest vertical curves are in the shape of a parabola. The basic equations for determining minimum length of a crest vertical curve are:

$$L = KA \qquad \text{(Equation 7-1)}$$

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \qquad \text{(Equation 7-2)}$$

where:

- L = length of vertical curve, ft
- K = horizontal distance needed to produce a 1% change in gradient
- A = algebraic difference between the two tangent grades, percent
- S = sight distance, ft
- h_1 = height of eye above road surface, ft
- h_2 = height of object above road surface, ft

The length of the crest vertical curve will depend upon "A" for the specific curve and upon the selected sight distance, height of eye and height of object. Following sections discuss the selection of these values.

7.2.1.1 Stopping Sight Distance

The principal control in the design of crest vertical curves is to ensure that, at a minimum, stopping sight distance (SSD) is available throughout the curve. Table 7.2A presents K-values for various operational conditions. The following discusses the application of the K-values:

1. Passenger Cars (Level Grade). Table 7.2A presents minimum and desirable K-values for passenger cars on level grades. These are calculated by assuming h_1 = 3.5 ft, h_2 = 0.5 ft and S = $SSD_{minimum}$ or $SSD_{desirable}$ in the basic equation for crest vertical curves (Equation 7-2). See Section 5.7 for SSD values. The minimum values represent the lowest acceptable sight distance on a facility. However, the designer should make every reasonable effort to provide a design which meets or exceeds the desirable K-values.

   Figures 7.2A and 7.2B present graphical relationships between the algebraic difference (A), the K-value based on SSD and the length of curve (L). Figure 7.2A should be used to rough out the vertical alignment in the early stages of design. At the detailed design stage, the applicable K-value should be used to calculate the vertical curve length. The calculated length should be adjusted to fit actual field conditions.

2. Passenger Cars (Downgrade Adjustment). Table 7.2A presents minimum and desirable K-values for passenger cars adjusted for 3% - 9% downgrades. If the downgrade on the facility is 3% or steeper, the designer should consider providing the K-value adjusted for grade. This adjustment reflects the longer braking distances which may be needed on downgrades.

3. Trucks (Level Grade). Table 7.2B presents K-values for trucks on level grades. These are calculated by assuming h_1 = 8.0 ft, h_2 = 0.5 ft and S = SSD for trucks in the basic equation for crest vertical curves (Equation 7-2). See

Table 7.2A

K-VALUES FOR CREST VERTICAL CURVES
(Passenger Cars)

Design Speed (mph)	No Grade Adjustment ($K = S^2/1329$)		Downgrades (2) ($K = S^2/1329$)					
			3%		6%		9%	
	Desirable	Minimum (1)	Desirable	Minimum	Desirable	Minimum	Desirable	Minimum
20	10	10	12	12	13	13	14	14
25	20	20	20	20	20	20	22	22
30	30	30	34	34	37	37	40	40
35	50	40	53	44	59	49	68	57
40	80	60	90	66	101	75	118	90
45	120	80	136	93	156	109	181	130
50	160	110	192	140	224	167	267	204
55	220	150	262	181	309	220	--	--
60	310	190	369	249	435	304	--	--
65	400	230	464	280	550	348	--	--
70	540	290	637	364	768	464	--	--

Source: (1) Revised

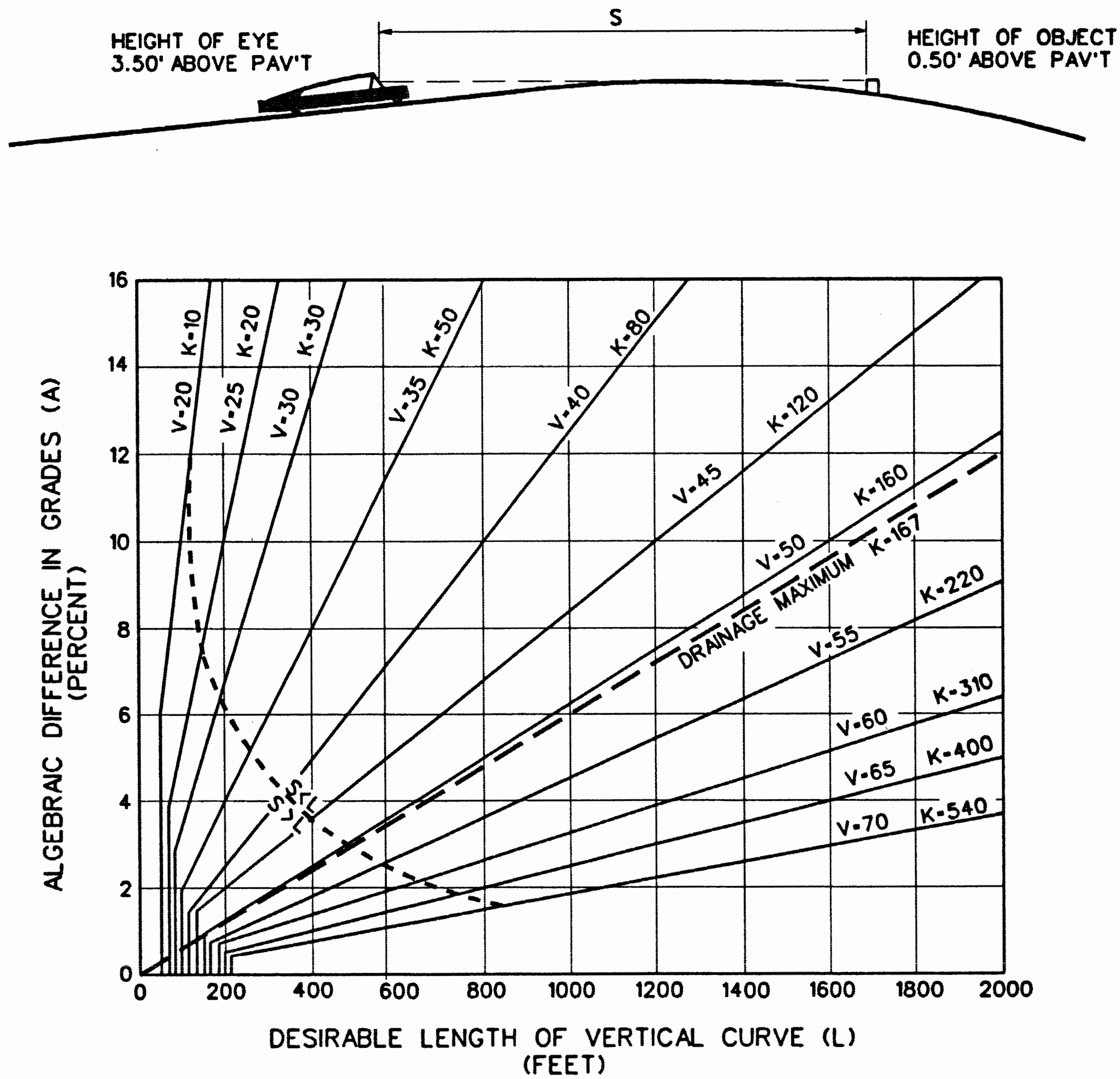
Notes:

1. The minimum K-values for passenger cars with no grade adjustment are the minimum acceptable values in design.

2. For downgrades intermediate between 3%, 6% and 9%, use a straight-line interpolation to calculate K.

3. Desirable values are based on design speed; minimum values are based on an assumed average running speed.

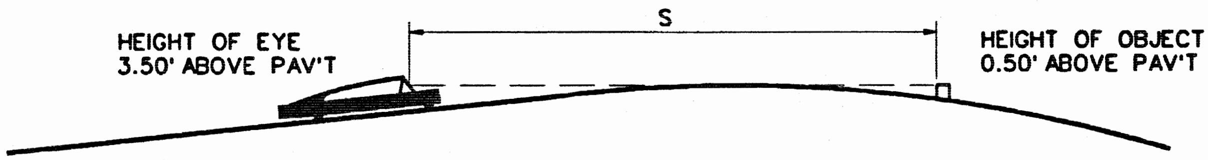
4. Assumed height of eye is 3.5 ft; height of object is 0.5 ft. For No Grade Adjustment, S is the Stopping Sight Distance (SSD) in Table 5.7A, "AASHTO Rounded for Design." For Downgrades, S is the SSD value in Table 5.7C.

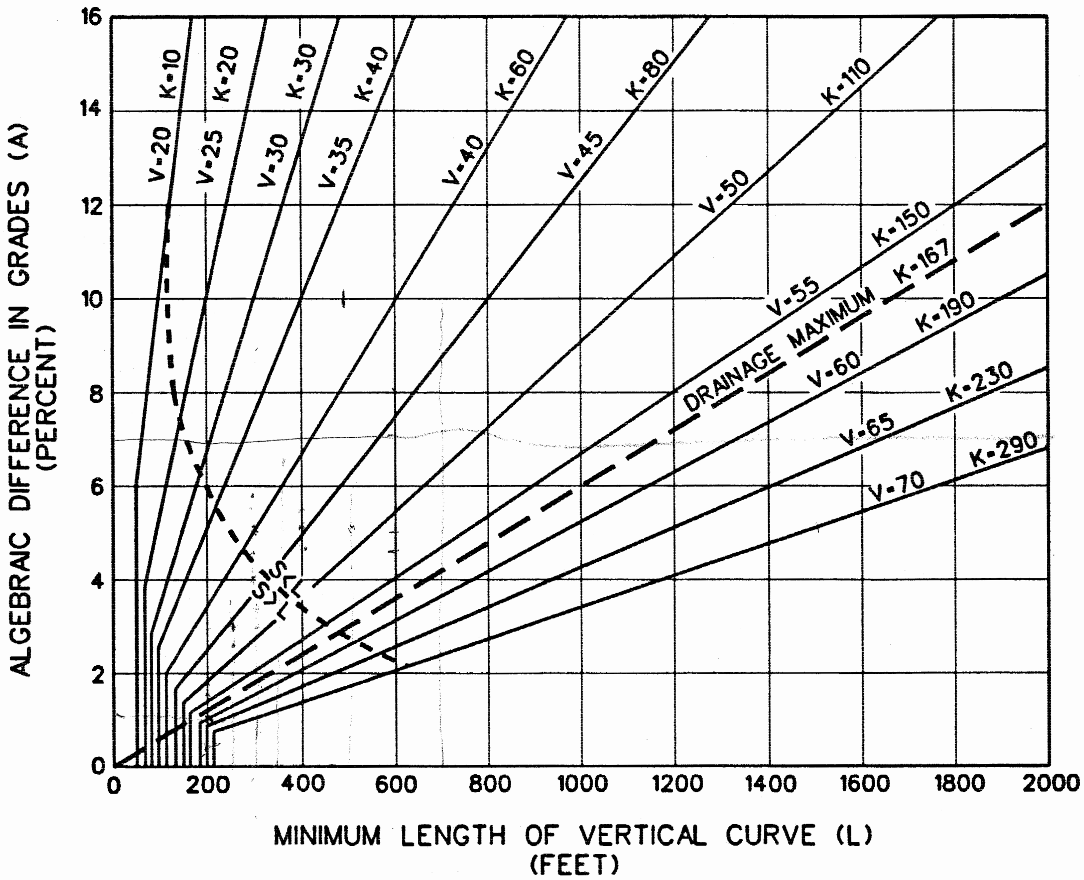


Source: (1) Revised

LENGTH OF CREST CURVE
(Desirable Stopping Sight Distance, Passenger Cars)

Figure 7.2A





Source: (1) Revised

LENGTH OF CREST CURVE
(Minimum Stopping Sight Distance, Passenger Cars)

Figure 7.2B

Table 7.2B

K-VALUES FOR CREST VERTICAL CURVES
(Trucks)

Design Speed (mph)	No Grade Adjustment ($K = S^2/2500$)	Downgrades ($K = S^2/2500$)		
		3%	6%	9%
20	10	12	15	22
25	21	27	38	64
30	36	46	67	114
35	62	84	127	235
40	97	132	201	375
45	147	204	318	612
50	208	289	450	859
55	285	397	618	--
60	372	517	793	--
65	500	699	1084	--
70	646	905	1405	--

Notes:

1. For downgrades intermediate between 3%, 6% and 9%, use a straight-line interpolation to calculate K.

2. Assumed height of eye is 8.0 ft; height of object is 0.5 ft. For No Grade Adjustment, S is the Stopping Sight Distance (SSD) in Table 5.7B for 2.5 seconds Perception/Reaction Time. For Downgrades, S is the SSD value in Table 5.7D for 2.5 seconds Perception/Reaction Time.

Section 5.7 for SSD values. The designer should consider providing the truck K-values at the following sites:

a. facilities with high truck volumes,

b. facilities with a high incidence of truck accidents,

c. railroad/highway grade crossings, and

d. special use facilities (e.g., truck weigh stations).

4. Trucks (Downgrade Adjustment). Table 7.2B presents K-values for trucks adjusted for 3% - 9% downgrades. The designer should consider using these K-values at the sites listed above and where the downgrade is 3% or steeper.

5. Minimum Length. The minimum length of a crest vertical curve should be 3V, where V is the design speed in mph. This applies regardless of the calculated length of vertical curve.

7.2.1.2 Decision Sight Distance

At some locations, it may be warranted to provide decision sight distance in the design of crest vertical curves. Section 5.7 discusses candidate sites and provides design values for decision sight distance. These "S" values should be used in the basic equation for crest vertical curves (Equation 7-2, in Section 7.2.1). In addition, the following will apply:

1. Height of Eye (h_1). For passenger cars, h_1 is 3.5 ft. For trucks, h_1 is 8.0 ft.

2. Height of Object (h_2). Decision sight distance, in many cases, is predicated upon the same principles as stopping sight distance; i.e., the driver needs sufficient distance to see a 6" object. Therefore, h_2 is 0.5 ft at many locations. However, candidate sites for decision sight distance may also be candidate sites for assuming that the object is the pavement surface (e.g., freeway exit gores). Therefore, the designer may decide to assume that h_2 = 0.0 ft for application.

7.2.1.3 Drainage

Drainage should be considered in the design of crest vertical curves where curbed sections are used. Drainage problems should not be experienced if the vertical curvature is sharp enough so that a minimum longitudinal grade of at least 0.3% is reached at a point about 50 ft from either side of the apex. To ensure that this objective is achieved, the length of the vertical curve should be based upon a K-value of 167 or less. For crest vertical curves on curbed sections where this K-value is exceeded, the drainage design should be more carefully evaluated near the apex.

For uncurbed sections of highway, drainage should not be a problem at crest vertical curves. However, it is desirable to provide a longitudinal gradient of at least 0.15% at points about 50 ft on either side of the high point (K = 333).

See Chapter Fifteen and the *ODOT Drainage Manual* for more information.

7.2.2 Sag Vertical Curves

Sag vertical curves are in the shape of a parabola. Typically, they are designed to allow the vehicle's headlights to illuminate the roadway surface (i.e., the height of object = 0.0 ft) for a given distance "S." These assumptions yield the following basic equations for determining the minimum length of sag vertical curves:

$$L = KA \qquad \text{(Equation 7-1)}$$

$$L = \frac{AS^2}{200h_1 + 3.5S} \qquad \text{(Equation 7-3)}$$

where:

L = length of vertical curve, ft

K = horizontal distance needed to produce a 1% change in gradient

A = algebraic difference between the two tangent grades, percent

S = sight distance, ft

h_1 = height of headlights above pavement surface, ft

The length of the sag vertical curve will depend upon "A" for the specific curve and upon the selected sight distance and headlight height. The following sections discuss the selection of these values.

7.2.2.1 Stopping Sight Distance

The principal control in the design of sag vertical curves is to ensure that, at a minimum, stopping sight distance (SSD) is available for headlight illumination throughout the curve. Table 7.2C presents K-values for various operational conditions. These are calculated for level conditions and they apply to all sites; i.e., the grade adjustment for SSD does not apply to sag vertical curves. The following discusses the application of the K-values:

1. Passenger Cars. Table 7.2C presents minimum and desirable K-values for passenger cars. These are calculated by assuming h_1 = 2.0 ft, S = $SSD_{minimum}$ or $SSD_{desirable}$ in the basic equation for sag vertical curves (Equation 7-3). See Section 5.7 for SSD values. The minimum values represent the lowest acceptable sight distance on a facility. However, the designer should make every reasonable effort to provide a design which meets or exceeds the desirable K-values.

   Figures 7.2C and 7.2D present graphical relationships between the algebraic difference (A), the K-value based on SSD and the length of curve (L). Figure 7.2C should be used to rough out the vertical alignment in the early stages of design. At the detailed design stage, the applicable K-value should be used to calculate the vertical curve length. The calculated length should be adjusted to fit actual field conditions.

2. Trucks. Table 7.2C presents K-values for trucks. These are calculated by assuming h_1 = 4.0 ft and S = SSD for trucks in the basic equation for sag vertical curves (Equation 7-3). See Section 5.7 for SSD values. The designer should consider providing the truck K-values at the following sites:

   a. facilities with high truck volumes,

   b. facilities with a high incidence of truck accidents,

   c. railroad/highway grade crossings, and

   d. special use facilities (e.g., truck weigh stations).

3. Minimum Length. The minimum length of a sag vertical curve should be 3V, where V is the design speed in mph. This applies regardless of the calculated length of vertical curve.

   One exception to this minimum length on sag vertical curves may apply. If the sag

Table 7.2C

K-VALUES FOR SAG VERTICAL CURVES

Design Speed (mph)	Passenger Cars ($K = S^2/(400 + 3.5S)$)		Trucks ($K = S^2/(800 + 3.5S)$)
	Desirable	Minimum (1)	
20	20	20	20
25	30	30	32
30	40	40	48
35	50	50	71
40	70	60	96
45	90	70	126
50	110	90	157
55	130	100	190
60	160	120	223
65	180	130	265
70	220	150	308

Source: (1) Revised

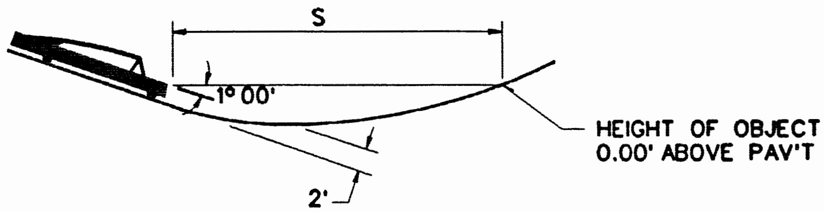
Notes:

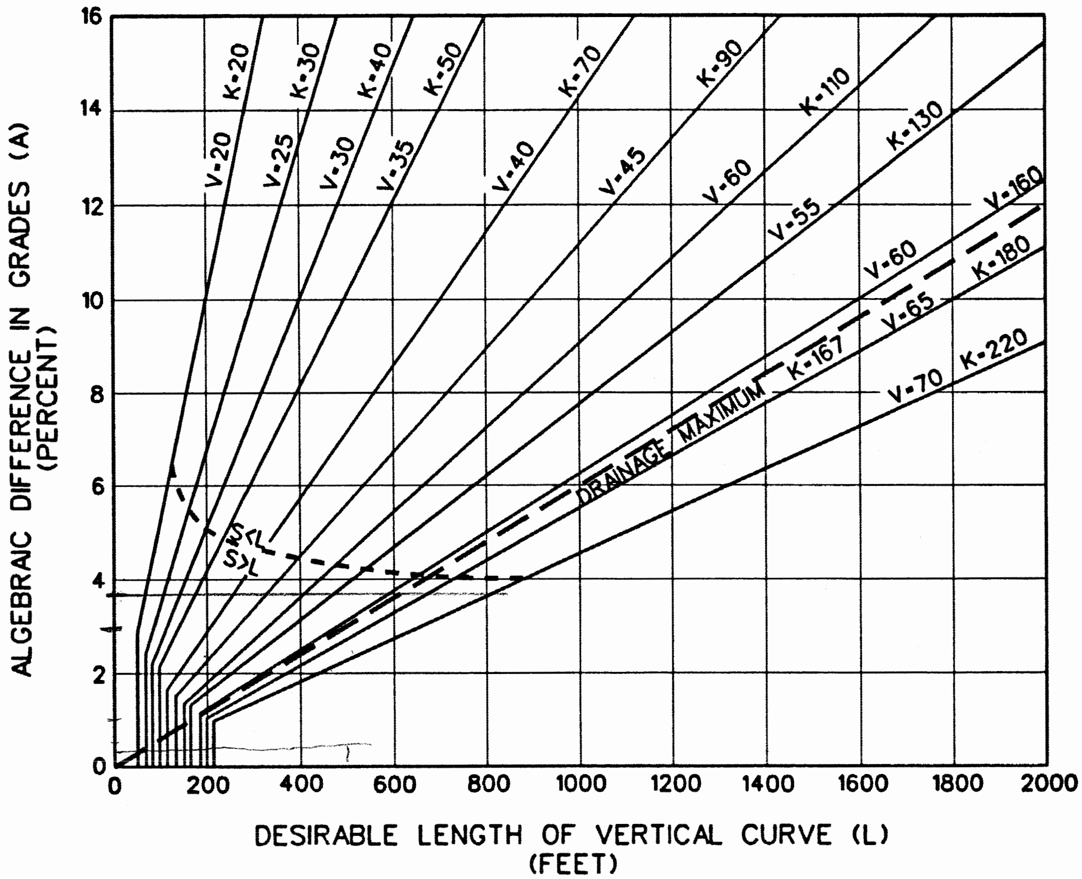
1. The minimum K-values for passenger cars are the minimum acceptable values in design.

2. Desirable values are based on design speed; minimum values are based on an assumed average running speed.

3. For passenger cars, assumed height of eye is 3.5 ft, height of object is 0.5 ft, and S is the Stopping Sight Distance (SSD) in Table 5.7A, "AASHTO Rounded for Design."

4. For trucks, assumed height of eye is 8.0 ft, height of object is 0.5 ft, and S is the Stopping Sight Distance (SSD) in Table 5.7B for 2.5 seconds Perception/Reaction Time.

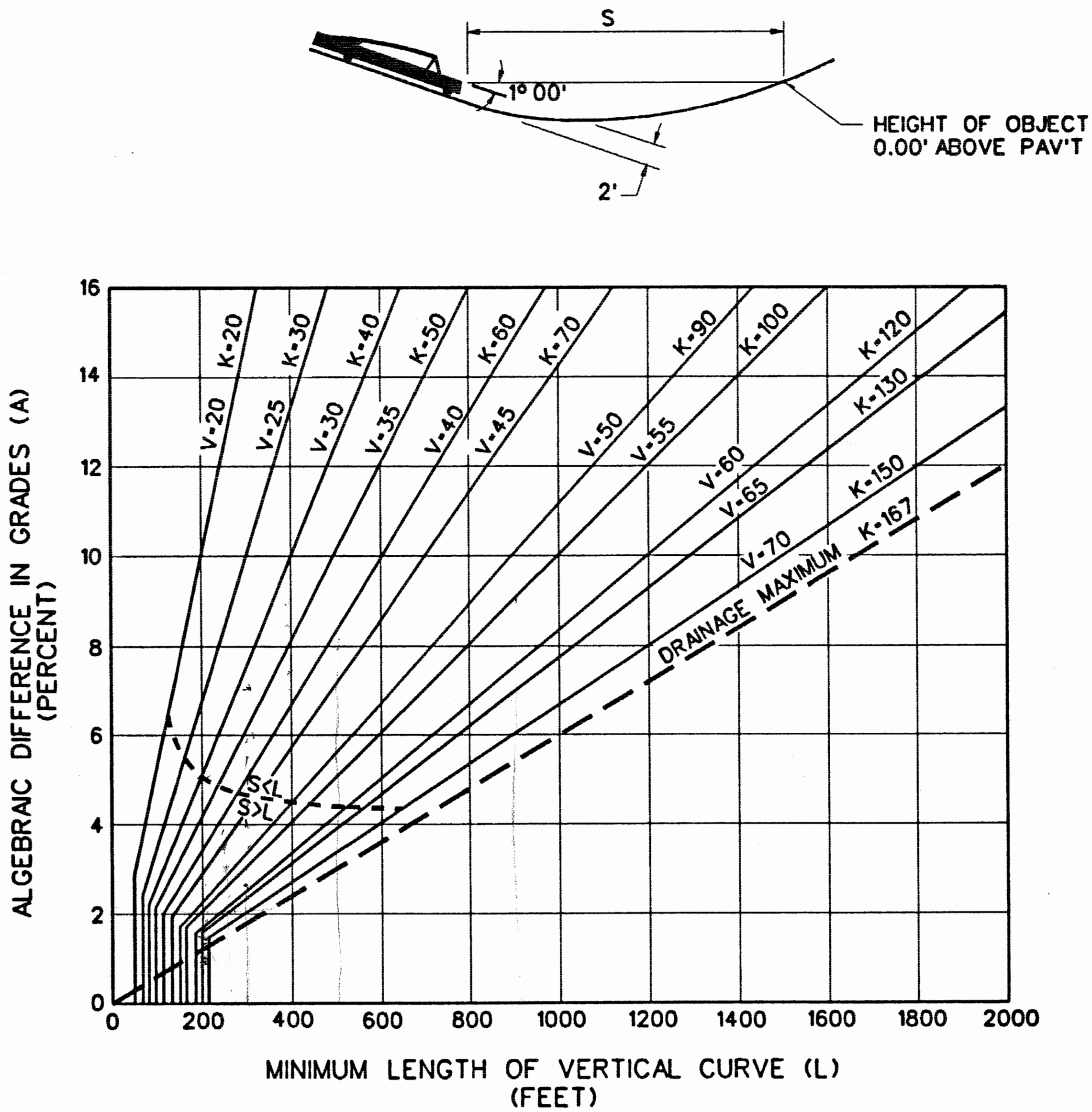




Source: (1) Revised

LENGTH OF SAG CURVE
(Desirable Stopping Sight Distance, Passenger Cars)

Figure 7.2C



Source: (1) Revised

LENGTH OF SAG CURVES
(Minimum Stopping Sight Distance, Passenger Cars)

Figure 7.2D

is in a "sump," the use of the minimum length criteria may produce longitudinal slopes too flat to drain the stormwater without exceeding the criteria for the limits of ponding on the travel lane. See Chapter Fifteen and the ODOT *Drainage Manual* for more discussion.

7.2.2.2 Comfort Criteria

On fully lighted continuous sections of highway and where it is impractical to provide stopping sight distance, it may be warranted to design a sag vertical curve to meet the comfort sag criteria. These criteria are based on the comfort effect of change in vertical direction in a sag vertical curve because of the combined gravitational and centrifugal forces. The general consensus is that riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 1 ft/sec^2. The length-of-curve equation for the comfort criteria is:

$$L = \frac{AV^2}{46.5} \qquad \text{(Equation 7-4)}$$

where:

L = length of vertical curve, ft

A = algebraic difference in tangent grades, percent

V = design speed, mph

Section 9.1 presents comfort criteria for sag vertical curves which apply specifically to at-grade intersections. These criteria yield sags which have sharper curvature than sags based on Equation 7-4. The criteria in Section 9.1 are not considered appropriate for sags on mainline away from intersections.

7.2.2.3 Underpasses

The designer should check sag vertical curves through underpasses to ensure that the underpass structure does not obstruct the driver's visibility. The following equation applies to underpasses:

$$L = \frac{AS^2}{800\ (C - 4.25)} \qquad \text{(Equation 7-5)}$$

where:

L = length of vertical curve, ft

A = algebraic difference between the two tangent grades, percent

S = sight distance, ft

C = vertical clearance of underpass, ft

The L calculated from Equation 7-5 for underpasses should be compared to the L calculated based on headlight illumination (Equation 7-3). The larger of the two lengths will govern.

7.2.2.4 Decision Sight Distance

At some locations, it may be warranted to provide decision sight distance in the design of sag vertical curves. Section 5.7 discusses candidate sites and provides design values for decision sight distance. These "S" values should be used in the basic equation for sag vertical curves (Equation 7-3). The height of headlights (h_1) is 2.0 ft.

7.2.2.5 Drainage

Drainage should be considered in the design of sag vertical curves where curbed sections are used. Drainage problems are minimized if the sag vertical curve is sharp enough so that both of the following criteria are met:

1. a minimum longitudinal grade of at least 0.3% is reached at a point about 50 ft from either side of the low point, and

2. there is at least a 4-inch elevation differential between the low point in the sag and the two points 50 ft to either side of the low point.

To ensure that the first objective is achieved, the length of the vertical curve should be based upon a K-value of 167 or less. For sag vertical curves on curbed sections where this K-value is exceeded, the drainage design should be more carefully evaluated near the low point. For example, it may be necessary to install flanking inlets on either side of the low point.

For uncurbed sections of highway, drainage should not be a problem at sag vertical curves. However, it is desirable to provide a longitudinal gradient of at least 0.15% at points about 50 ft on either side of the low point (K = 333).

See Chapter Fifteen and the *ODOT Drainage Manual* for more information.

7.2.3 Low-Speed Urban Streets

In general, the criteria presented in Section 7.2 applies to the use of vertical curves on low-speed urban streets ($V \leq 45$ mph). In addition, the following will apply:

1. Angular Breaks (Sags). On the mainline of low-speed urban streets, a maximum angular break (i.e., no vertical curve) of 0.8% is acceptable where the following conditions are met:

   a. The facility is curbed.

   b. The angular break is in a sag.

   c. Field conditions make the use of a vertical curve impractical.

   Note that a series of angular breaks in lieu of a vertical curve is not an acceptable design, even where the angular breaks do not exceed 0.8%.

2. Angular Breaks (Crests). These are not allowed; i.e., a crest vertical curve must be inserted.

7.3 DESIGN PRINCIPLES AND PROCEDURES

7.3.1 General Controls

As discussed elsewhere in Chapter Seven, the design of vertical alignment involves, to a large extent, complying with specific limiting criteria. These include maximum and minimum grades, sight distance at vertical curves and vertical clearances. In addition, the designer should adhere to certain general design principles and controls which will determine the overall safety of the facility and will enhance the aesthetic appearance of the highway. These design principles for vertical alignment include:

1. Consistency. Use a smooth grade line with gradual changes, consistent with the type of highway and character of terrain, rather than a line with numerous breaks and short lengths of tangent grades.

2. Roller Coaster. The "roller-coaster" type of profile should be avoided. They may be proposed in the interest of economy, but they are aesthetically undesirable and may be hazardous.

3. Broken-Back Curvature. Avoid "broken-back" grade lines (two crest or sag vertical curves separated by a short tangent). One long vertical curve is more desirable.

4. Long Grades. On a long ascending grade, it is preferable to place the steepest grade at the bottom and flatten the grade near the top.

5. Intersections. Maintain moderate grades through intersections to facilitate turning movements. See Chapter Nine for specific information on vertical alignment through intersections.

6. Sags. Sag vertical curves should be avoided in cuts unless adequate drainage can be provided.

7. Coordination with Natural/Man-Made Features. The vertical alignment should be properly coordinated with the natural topography, available right-of-way, utilities, roadside development and natural/man-made drainage patterns.

8. Environmental Impacts. Vertical alignment should be properly coordinated with environmental impacts (e.g., encroachment onto wetlands). The ODOT Planning Division is responsible for evaluating environmental impacts.

7.3.2 Coordination of Horizontal and Vertical Alignment

Horizontal and vertical alignment should not be designed separately, especially for projects on new alignment. Their importance demands that the designer carefully evaluate the interdependence of the two highway design features. This will enhance highway safety and improve the facility's operation. The following should be considered in the coordination of horizontal and vertical alignment:

1. Balance. Curvature and grades should be in proper balance. Maximum curvature with flat grades or flat curvature with maximum grades does not achieve the desired balance.

2. Coordination. Vertical curvature superimposed upon horizontal curvature (i.e., vertical and horizontal P.I.'s at approximately the same stations) generally results in a more pleasing appearance and reduces the number of sight distance restrictions. However, this must be

tempered somewhat by Comment #'s 3 and 4, as follows.

3. Crest Vertical Curves. Sharp horizontal curvature should not be introduced at or near the top of pronounced crest vertical curves. This may be hazardous because the driver cannot perceive the horizontal change in alignment, especially at night when headlight beams project straight ahead into space. The potential hazard can be avoided if the horizontal curvature leads the vertical curvature.

4. Sag Vertical Curves. Sharp horizontal curves should not be introduced at or near the low point of pronounced sag vertical curves or at the bottom of steep vertical grades. Night visibility is again one problem, and this alignment creates an undesirable, distorted appearance.

5. Passing Sight Distance. In some cases, the need to provide a higher percentage of passing sight distance may supersede the desirability of combining horizontal and vertical alignment.

6. Intersections. At intersections, horizontal and vertical alignment should be as flat as possible. See Chapter Nine.

7. Divided Facilities. On divided facilities with wide medians, it is frequently advantageous to provide independent alignments for the two one-way roadways.

7.3.3 Profile Grade Line

7.3.3.1 General

The profile grade line is perhaps the roadway geometric characteristic which has the greatest impact on a facility's costs, aesthetics, safety and operation. The profile grade is a series of tangent lines connected by vertical curves. It is typically placed along the roadway centerline of undivided facilities and at the two median edges on divided facilities.

The designer must carefully evaluate many factors when establishing the profile grade line. These include:

1. maximum and minimum gradients;
2. sight distance criteria;
3. earthwork balance;
4. bridges and other structures;
5. high water levels;
6. drainage considerations;
7. highway intersections and interchanges;
8. snow drifting;
9. railroad/highway crossings;
10. types of soil;
11. adjacent land use and values;
12. highway safety;
13. coordination with other geometric features (e.g., cross section, design speed);
14. topography/terrain;
15. truck performance;
16. right-of-way;
17. utilities;
18. urban/rural location;
19. aesthetics/landscaping; and
20. construction costs.

The following sections discuss the establishment of the profile grade line in more detail.

7.3.3.2 Earthwork Balance

Where practical and where consistent with other project objectives, the designer should design the profile grade line to provide a balance of earthwork. This should not be achieved, however, at the expense of smooth grade lines and sight distance requirements at vertical curves. Ultimately, a project-by-project assessment will determine whether a project will be borrow, waste or balanced.

The following should be considered in earthwork balance:

1. Basic Approach. The best approach to laying grade and balancing earthwork is to provide a significant length of roadway in embankment, to limit the number and amount of excavation areas, and to provide the maximum practical length of roadway between balance points (e.g., up to two miles). The designer should, as practical, avoid long lengths of roadway in excavation and avoid several short balance distances.

2. Urban/Rural. Earthwork balance is typically a practical objective only in rural areas. In urban areas, other project objectives (e.g., limiting right-of-way impacts) may have a higher priority than balancing earthwork.

3. Borrow Sites. The availability and quality of borrow sites in the vicinity of the project will impact the desirability of balancing the earthwork.

4. Daylighting. Where practical, daylighting at cut areas should be provided.

5. Mass Diagram. A mass diagram illustrates the accumulated algebraic sum of yardage between project limits. These diagrams are useful in balancing earthwork and calculating haul distances and quantities. The mass diagram may indicate:

   a. the most economical procedure for disposing of excavated material,

   b. whether material should be moved backward or forward, or

   c. whether borrowing or wasting is more economical than achieving earthwork balance.

   Depending upon the nature of the project, the designer may decide to construct a mass diagram. See Chapter Four for more information.

6. Balance Points. Where practical, balance points should be located at natural and man-made interruptions in the roadway (e.g., bridges, railroad crossings, intersections).

7. Earthwork Computations. Chapter Four discusses the proper methods to compute and record the project earthwork quantities.

7.3.3.3 Bridges

The design of profile grade lines must be carefully coordinated with any bridges within the project limits. The following will apply:

1. Vertical Clearances. The criteria in Chapters Twelve and Thirteen and Section 7.4 must be met. When laying the preliminary grade line, an important element in determining available vertical clearance is the assumed structure depth.

This will be based on the structure type, span lengths and depth/span ratio. For preliminary design, the designer should assume a 23 ft to 25 ft distance between the finished grade of the roadway and the finished grade of the bridge deck. For final design, the designer must coordinate with the Bridge Division to determine the roadway and bridge grade lines.

2. Bridges Over Water. Where the proposed facility will cross bodies of water, the bridge elevation must be consistent with the necessary waterway opening to meet ODOT hydraulic requirements. The designer must coordinate with the Bridge Division to determine the approach roadway elevation to meet the necessary bridge elevation.

3. Railroad Bridges. Any proposed facilities over railroads must meet the applicable criteria (e.g., vertical clearances, structure type and depth). The designer should coordinate with the Planning Division, Rail Planning Branch and Bridge Division to establish the proper grade line.

4. Highway Under Bridge. Where practical, the low point of a roadway sag vertical curve should not be within the shadow of the bridge. This will help minimize ice accumulations, and it will reduce the ponding of water which may weaken the earth foundation beneath the bridge. To achieve these objectives, the low point of a roadway sag should be at least 100 ft from the bridge.

5. High Embankments. The designer should consider the impacts of high embankments on structures. This will increase the span length thus increasing structure costs.

7.3.3.4 Drainage/Snow

The profile grade line should be compatible with the roadway drainage design and should minimize snow drift problems. The following will apply:

1. Culverts. The roadway elevation should meet the ODOT criteria for minimum cover at culverts and minimum freeboard above the head water level at culverts. See Chapter Fifteen and the *ODOT Drainage Manual* for more information.

2. Coordination with Geometrics. The profile grade line must reflect compatibility between drainage design and roadway geometrics. These include the design of sag and crest vertical curves, spacing of inlets on curbed facilities, impacts on adjacent properties, superelevated curves, intersection design elements and interchange design elements.

3. Snow Drifting. For some rural roadways, the designer should consider locating the profile grade line at least 3 ft above the natural ground level to prevent snow from drifting onto the roadway.

7.3.3.5 Soils

The type of earth material encountered often influences the grade line at certain locations. If rock is encountered, for example, it may be more economical to raise the grade and reduce the rock excavation. Soils which are unsatisfactory for embankment or cause a stability problem in cut areas may also be determining factors in establishing a grade line. The designer should coordinate the development of the profile grade with the Materials Division, which will conduct a pedological survey.

7.3.3.6 Erosion Control

Section 17.14 discusses erosion control in general. Specifically related to the profile grade line, the following will minimize erosion:

1. Minimize the number of cut and fill sections.
2. Conform to the contour and drainage patterns of the area.
3. Make use of natural land barriers and contours to divert runoff and confine erosion and sedimentation.
4. Minimize the amount of disturbance.
5. Make use of existing vegetation.
6. Reduce slope length and steepness and ensure that erosion is confined to the right-of-way and does not deposit sediment on or erode away adjacent land.
7. Avoid locations having high base erosion potential.
8. Avoid cut or fill sections in seepage areas.

7.3.3.7 Ties with Existing Highways

A smooth transition is needed between the proposed profile grade line of the project and the existing grade line of an adjacent highway section. Existing grade lines should be considered for a sufficient distance beyond the beginning and end 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 if the adjoining road section is reconstructed.

7.3.4 Design Procedures

In the design of vertical alignment, the designer should be aware of his responsibility to communicate properly with other ODOT personnel (e.g., drafting, field survey). Section 7.3.3 provides several examples of needed coordination in the development of the profile grade line. The following also applies:

1. Preparation of Plans. Chapter Four "Plan Development" discusses the content and format of the plan sheets, abbreviations, symbols, scales and the use of ODOT's CADD system. The designer must ensure that the presentation of the vertical alignment is consistent with ODOT practices.
2. Project Development. Chapter Two "Project Development Process" indicates the necessary timing for the coordination between vertical alignment design and other highway activities (e.g., bridge elevations).
3. Survey (General). The designer should ensure that the vertical alignment is consistent with ODOT field survey practices. See Section Four of the ODOT's *Manual of Policies and Procedures*.

7.3.5 Mathematical Computations

Section 7.6 presents numerous figures which provide the needed mathematical equations and techniques to make various computations for vertical curves.

7.4 VERTICAL CLEARANCES

Table 7.4A summarizes the minimum vertical clearances for various conditions for new construction/reconstruction projects. Chapter Twelve provides additional information. Chapter Thirteen provides vertical clearance information for existing highways.

Table 7.4A

MINIMUM VERTICAL CLEARANCES
(New Construction/Reconstruction)

Type	Minimum Clearance
Roadway under Other Roadway or Railroad (State or non-State)	16′-9″ (1)
Roadway under Pedestrian Bridge	17′-9″ (1)
Roadway under Traffic Signal	17′ ± 1″ (2)
Railroad under Roadway	23′-0″ typ. (3)
Roadway under Sign Truss	18′-0″ (1)

Notes:

1. *Value allows 9″ for future resurfacing.*
2. *Distance is measured from roadway surface to the bottom of signal at the bottom of the back plate or to the mast arm. See ODOT Traffic Standard Drawings.*
3. *Contact the Planning Division, Rail Planning Branch and Bridge Division to determine acceptable vertical clearance.*

7.5 TRUCK-CLIMBING LANES

7.5.1 Warrants

A truck-climbing lane may be warranted to allow a specific upgrade to operate at an acceptable level of service. For desirable warranting criteria see *Highway Capacity Manual* (HCM). The following minimum criteria will apply.

7.5.1.1 Two-Lane Highways

A truck climbing lane on a two-lane, two-way highway will generally be warranted if the following conditions are satisfied:

1. the critical length of grade is exceeded for the 10-mph speed reduction curve (see Figure 7.1A); and
2. one of the following conditions exists:
   a. the level of service (LOS) on the upgrade is E or F, or
   b. there is a reduction of two or more LOS when moving from the approach segment to the upgrade; and
3. the construction costs and the construction impacts (e.g., environmental, right-of-way) are considered reasonable.

Truck-climbing lanes may also be warranted where the above criteria are not met if, for example, there is an adverse accident experience on the upgrade related to slow-moving trucks.

7.5.1.2 Multilane Highways

A truck-climbing lane will generally be warranted on a multilane highway if the following conditions are satisfied:

1. the critical length of grade is exceeded for the 10-mph speed reduction curve (see Figure 7.1A); and
2. the directional service volume exceeds 1000 vph; and
3. one of the following conditions exists:
   a. the LOS on the upgrade is E or F, or
   b. there is a reduction of two or more LOS when moving from the approach segment to the upgrade; and
4. the construction costs and the construction impacts (e.g., environmental, right-of-way) are considered reasonable.

Truck-climbing lanes may also be warranted where the above criteria are not met if, for example, there is an adverse accident experience on the upgrade related to slow-moving trucks.

7.5.2 Capacity Procedures

7.5.2.1 Two-Lane Highways

The objective of the capacity analysis procedure is to determine if the warranting criteria in Section 7.5.1 are met for two-lane facilities. This is accomplished by calculating the service flow rate for each LOS level (A through D) and comparing this to the actual flow rate on the upgrade. Because a LOS worse than D warrants a truck-climbing lane, it is not necessary to calculate the service flow rate for LOS E.

The following capacity procedure, which is based on the 1985 *Highway Capacity Manual* (HCM), will allow the designer to perform these calculations. This procedure is the HCM capacity methodology for a specific grade on a two-lane, two-way facility. The

HCM also presents a separate capacity methodology for analyzing general terrain segments, which applies to highway segments of at least 2 miles in length.

The mathematical expression for the LOS calculation is:

$$SF_i = 2800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

where:

SF_i	=	total service flow rate in both directions for LOS_i for prevailing roadway and traffic conditions, vph
2800	=	capacity of a two-lane, two-way facility (both directions) under ideal conditions, passenger cars per hour (pcph)
$(v/c)_i$	=	ratio of vehicular flow rate (for passenger cars unaffected by grade) to ideal capacity (2800 pcph) for LOS_i
f_d	=	adjustment factor for directional distribution
f_w	=	adjustment factor for narrow lanes and restricted shoulder width
f_g	=	adjustment factor for operational effects of grades on passenger cars
f_{HV}	=	adjustment factor for the presence of heavy vehicles in the upgrade traffic stream

Based on the specific characteristics of the upgrade, the calculation of SF_i will yield the vph (both directions for a mixed traffic composition) at the lower limit of the range of traffic volumes for LOS_i. Therefore, if the actual vph (both directions for a mixed traffic composition) on the upgrade yields LOS E or F, the truck-climbing lane capacity warrant is satisfied. In addition, if the LOS on the upgrade is two or more levels worse than the LOS on the highway segment approaching the upgrade, the truck-climbing lane capacity warrant is satisfied.

The following step-by-step procedure is used to calculate SF_i:

1. Compile the following data for the analysis:

   a. average annual daily traffic (AADT) (mixed composition for year under design);

   b. the K factor (i.e., the proportion of the AADT occurring in the design hour);

   c. the directional distribution (D) during the design hour (DHV);

   d. the truck factor (T) during the DHV (i.e., the % of trucks, buses and recreational vehicles);

   e. peak-hour factor (PHF) from Table 7.5A (default values);

   f. the design speed;

   g. lane and shoulder widths (ft);

   h. percent grade;

   i. percent no-passing zones (based on the MUTCD criteria for striping of no-passing zones), see Section 5.7 and Reference 6; and

   j. length of grade (in miles).

2. Use Table 7.5B to determine the average upgrade speed for LOS.

Table 7.5A

PEAK-HOUR FACTORS
(Default Values)

Level of Service	Peak-Hour Factor
A	0.91
B	0.92
C	0.94
D	0.95
E	1.00

Source: (2)

Notes:

1. *Table is only applicable for the calculation of a service flow rate.*
2. *Table only applies to two-lane, two-way rural facilities.*

Table 7.5B

LEVEL-OF-SERVICE CRITERIA FOR SPECIFIC GRADES

Level of Service	Average Upgrade Speed (mph)
A	≥ 55
B	≥ 50
C	≥ 45
D	≥ 40
E	≥ 25-40[a]
F	< 25-40[a]

Source: (2)

a *The exact speed at which capacity occurs varies with the percentage and length of grade, traffic composition and volume.*

3. Use Table 7.5C to determine v/c for LOS_i for the percent grade and percent no-passing zones for the upgrade under design.
4. Use Table 7.5D to determine f_d for the directional distribution.
5. Use Table 7.5E to determine f_w for the effects of lane and shoulder width.
6. To determine f_g, use the following equation:

$$f_g = 1/[1 + (P_p I_p)]$$

where:

f_g = adjustment factor for the operation of passenger cars on grades

P_p = proportion of passenger cars in the upgrade traffic stream, expressed as a decimal

I_p = impedance factor for passenger cars, computed as:

$$I_p = 0.02\ (E - E_o)$$

E = base passenger-car equivalent for a given percent grade, length of grade and average upgrade speed (see Table 7.5F)

E_o = base passenger-car equivalent for 0% grade and a given speed (see Table 7.5F)

7. To determine f_{HV}, use the following equation:

$$f_{HV} = 1/[1 + P_{HV}\ (E_{HV} - 1)]$$

Table 7.5C

VALUES OF v/c RATIO[a] vs. SPEED, PERCENT GRADE, AND PERCENT NO-PASSING ZONES FOR SPECIFIC GRADES

PERCENT GRADE	AVERAGE UPGRADE SPEED (MPH)	PERCENT NO-PASSING ZONES					
		0	20	40	60	80	100
3	55	0.27	0.23	0.19	0.17	0.14	0.12
	52.5	0.42	0.38	0.33	0.31	0.29	0.27
	50	0.64	0.59	0.55	0.52	0.49	0.47
	45	1.00	0.95	0.91	0.88	0.86	0.84
	42.5	1.00	0.98	0.97	0.96	0.95	0.94
	40	1.00	1.00	1.00	1.00	1.00	1.00
4	55	0.25	0.21	0.18	0.16	0.13	0.11
	52.5	0.40	0.36	0.31	0.29	0.27	0.25
	50	0.61	0.56	0.52	0.49	0.47	0.45
	45	0.97	0.92	0.88	0.85	0.83	0.81
	42.5	0.99	0.96	0.95	0.94	0.93	0.92
	40	1.00	1.00	1.00	1.00	1.00	1.00
5	55	0.21	0.17	0.14	0.12	0.10	0.08
	52.5	0.36	0.31	0.27	0.24	0.22	0.20
	50	0.57	0.49	0.45	0.41	0.39	0.37
	45	0.93	0.84	0.79	0.75	0.72	0.70
	42.5	0.97	0.90	0.87	0.85	0.83	0.82
	40	0.98	0.96	0.95	0.94	0.93	0.92
	35	1.00	1.00	1.00	1.00	1.00	1.00
6	55	0.12	0.10	0.08	0.06	0.05	0.04
	52.5	0.27	0.22	0.18	0.16	0.14	0.13
	50	0.49	0.40	0.35	0.31	0.28	0.26
	45	0.85	0.76	0.68	0.63	0.59	0.55
	42.5	0.93	0.84	0.78	0.74	0.70	0.67
	40	0.97	0.91	0.87	0.83	0.81	0.78
	35	1.00	0.96	0.95	0.93	0.91	0.90
	30	1.00	0.99	0.99	0.98	0.98	0.98
7	55	0.00	0.00	0.00	0.00	0.00	0.00
	52.5	0.13	0.10	0.08	0.07	0.05	0.04
	50	0.34	0.27	0.22	0.18	0.15	0.12
	45	0.77	0.65	0.55	0.46	0.40	0.35
	42.5	0.86	0.75	0.67	0.60	0.54	0.48
	40	0.93	0.82	0.75	0.69	0.64	0.59
	35	1.00	0.91	0.87	0.82	0.79	0.76
	30	1.00	0.95	0.92	0.90	0.88	0.86

Source: (2)

[a] Ratio of flow to ideal capacity of 2,800 pcph, assuming passenger-car operation is unaffected by grade.

NOTE: Interpolate for intermediate values of "Percent No-Passing Zones"; round "Percent Grade" to the next higher whole number.

Table 7.5D

ADJUSTMENT FACTOR FOR DIRECTIONAL DISTRIBUTION ON SPECIFIC GRADES, f_d

Percent of Traffic On Upgrade	Adjustment Factor
100	0.58
90	0.64
80	0.70
70	0.78
60	0.87
50	1.00
40	1.20
≤30	1.50

Source: (2)

Table 7.5E

ADJUSTMENT FACTORS FOR THE COMBINED EFFECT OF NARROW LANES AND RESTRICTED SHOULDER WIDTH, f_w

Usable Shoulder Width[a] (ft)	12-ft Lanes[b]		11-ft Lanes[b]		10-ft Lanes[b]		9-ft Lanes[b]	
	LOS A-D	LOS E	LOS A-D	LOS E	LOS A-D	LOS E	LOS A-D	LOS E
≥6	1.00	1.00	0.93	0.94	0.84	0.87	0.70	0.76
4	0.92	0.97	0.85	0.92	0.77	0.85	0.65	0.74
2	0.81	0.93	0.75	0.88	0.68	0.81	0.57	0.70
0	0.70	0.88	0.65	0.82	0.58	0.75	0.49	0.66

Source: (2)

[a] Where shoulder width is different on each side of the roadway, use the average shoulder width.
[b] For analysis of specific grades, use LOS E factors for all speeds less than 45 mph.

where:

f_{HV} = adjustment factor for the presence of heavy vehicles in the upgrade traffic stream

P_{HV} = total proportion of heavy vehicles (trucks + RV's + buses) in the upgrade traffic stream, expressed as a decimal

E_{HV} = passenger-car equivalent for specific mix of heavy vehicles present in the upgrade traffic stream, computed as:

$$E_{HV} = 1 + (0.25 + P_{T/HV})\ (E - 1)$$

$P_{T/HV}$ = proportion of trucks among heavy vehicles; i.e., the proportion of trucks in the traffic stream divided by the total proportion of heavy vehicles in the traffic stream, expressed as a decimal

E = base passenger-car equivalent for a given percent grade, length of grade and speed (see Table 7.5F)

8. From the data in Step #'s 1-7, calculate SF_i for each LOS_i.

9. Calculate the V=DHV (mixed vph for both directions during the design hour):

$$DHV = AADT\ x\ K$$

where terms are defined under Step #1.

10. Convert the DHV to an hourly flow rate using the PHF:

$$v = V/PHF$$

where:

v = the equivalent hourly flow rate during a given time interval less than one hour (usually 15 minutes), vph

V = the total number of vehicles for the DHV, vph

PHF = peak-hour factor

11. Compare SF_D (for LOS D) from Step #8 to v from Step #10. If $SF_D \geq v$, the upgrade will operate better than LOS E and, therefore, the climbing-lane capacity warrant is not satisfied. If $SF_D < v$, the upgrade will operate at LOS E or F and, therefore, the climbing-lane capacity warrant is satisfied.

12. Compare the service flow rates from Step #8 to v from Step #10 to determine the actual LOS on the upgrade. If the upgrade LOS is two or more levels worse than the LOS on the highway segment approaching the upgrade, then the climbing-lane capacity warrant is satisfied.

Figure 7.5A provides a worksheet which allows for a convenient documentation of the data and calculations in the capacity analysis.

* * * * * * * * * *

<u>Example 7-4</u>

<u>Given</u>: An upgrade on a rural two-lane highway is described as follows:

AADT = 4000
K = 12.5%
PHF = 0.85
Percent trucks = 10%
Percent RV's = 2%
Percent buses = 1%
Directional distribution during DHV = 60/40
(with 60% in upgrade direction)

Table 7.5F

PASSENGER-CAR EQUIVALENTS FOR SPECIFIC GRADES ON TWO-LANE RURAL HIGHWAYS, E AND E_o

GRADE (%)	LENGTH OF GRADE (mi)	AVERAGE UPGRADE SPEED (mph)					
		55.0	52.5	50.0	45.0	40.0	30.0
0	All	2.1	1.8	1.6	1.4	1.3	1.3
3	¼	2.9	2.3	2.0	1.7	1.6	1.5
	½	3.7	2.9	2.4	2.0	1.8	1.7
	¾	4.8	3.6	2.9	2.3	2.0	1.9
	1	6.5	4.6	3.5	2.6	2.3	2.1
	1½	11.2	6.6	5.1	3.4	2.9	2.5
	2	19.8	9.3	6.7	4.6	3.7	2.9
	3	71.0	21.0	10.8	7.3	5.6	3.8
	4	[a]	48.0	20.5	11.3	7.7	4.9
4	¼	3.2	2.5	2.2	1.8	1.7	1.6
	½	4.4	3.4	2.8	2.2	2.0	1.9
	¾	6.3	4.4	3.5	2.7	2.3	2.1
	1	9.6	6.3	4.5	3.2	2.7	2.4
	1½	19.5	10.3	7.4	4.7	3.8	3.1
	2	43.0	16.1	10.8	6.9	5.3	3.8
	3	[a]	48.0	20.0	12.5	9.0	5.5
	4	[a]	[a]	51.0	22.8	13.8	7.4
5	¼	3.6	2.8	2.3	2.0	1.8	1.7
	½	5.4	3.9	3.2	2.5	2.2	2.0
	¾	8.3	5.7	4.3	3.1	2.7	2.4
	1	14.1	8.4	5.9	4.0	3.3	2.8
	1½	34.0	16.0	10.8	6.3	4.9	3.8
	2	91.0	28.3	17.4	10.2	7.5	4.8
	3	[a]	[a]	37.0	22.0	14.6	7.8
	4	[a]	[a]	[a]	55.0	25.0	11.5
6	¼	4.0	3.1	2.5	2.1	1.9	1.8
	½	6.5	4.8	3.7	2.8	2.4	2.2
	¾	11.0	7.2	5.2	3.7	3.1	2.7
	1	20.4	11.7	7.8	4.9	4.0	3.3
	1½	60.0	25.2	16.0	8.5	6.4	4.7
	2	[a]	50.0	28.2	15.3	10.7	6.3
	3	[a]	[a]	70.0	38.0	23.9	11.3
	4	[a]	[a]	[a]	90.0	45.0	18.1
7	¼	4.5	3.4	2.7	2.2	2.0	1.9
	½	7.9	5.7	4.2	3.2	2.7	2.4
	¾	14.5	9.1	6.3	4.3	3.6	3.0
	1	31.4	16.0	10.0	6.1	4.8	3.8
	1½	[a]	39.5	23.5	11.5	8.4	5.8
	2	[a]	88.0	46.0	22.8	15.4	8.2
	3	[a]	[a]	[a]	66.0	38.5	16.1
	4	[a]	[a]	[a]	[a]	[a]	28.0

Source: (2)

[a] Speed not attainable on grade specified.

NOTE: Round "Percent Grade" to next higher whole number.

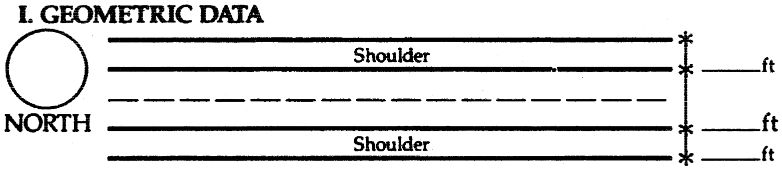
WORKSHEET FOR SPECIFIC GRADES

Page 1

Site Identification:____________ Date:________ Time:________

Name:____________ Checked by:____________

I. GEOMETRIC DATA



Design Speed:________mph
Grade:________%,________mi
% No Passing Zones:________

II. TRAFFIC DATA

Total Volume, Both Dir.____________vph
Flow Rate = Volume ÷ PHF
______ = ______ ÷ ______

Directional Distribution:____________
Traffic Composition:______ %T,______ %RV,______ %B
PHF:____________

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1 / [1 + P_p I_p]$
$I_p = 0.02\,(E - E_o)$

$f_{HV} = 1 / [1 + P_{HV}(E_{HV} - 1)]$
$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$

Speed (mph)	P_p	I_p	E	E_o	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E	f_{HV}
55										
52.5										
50										
45										
40										
30										

IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800 ×	v/c ×	f_d ×	f_w ×	f_g ×	f_{HV}
55 (LOS A)		2,800					
52.5		2,800					
50 (LOS B)		2,800					
45 (LOS C)		2,800					
40 (LOS D)		2,800					
30		2,800					

Upgrade = 4%
Percent no-passing zones = 50%
Length of upgrade = 1¼ miles
Lane width = 12 ft
Shoulder width = 8 ft
Design speed = 60 mph

Note: Determine if:

a. the critical length of grade is exceeded, based on a 10-mph truck-speed reduction, and

b. the highway segment approaching the upgrade has LOS A.

Problem:

Determine if a truck-climbing lane is warranted based on the capacity analysis in Section 7.5.

Solution:

The following presents the calculations specifically to determine SF_D. The completed worksheet at the end of the Example presents the service flow calculations for LOS A, B and C. As indicated in the procedure, it is not necessary to calculate LOS E to determine if the truck-climbing lane capacity warrant is satisfied.

The step-by-step procedure in Section 7.5.2 yields the following solution for SF_D:

1. The "Given" data has been compiled.

2. Table 7.5B yields an average upgrade speed of 40 mph for the lowest part of the LOS D range.

3. Using the given data of a 4% upgrade, 50% no-passing zones and V = 40 mph, Table 7.5C yields a v/c ratio of 1.00.

4. With a 60/40 directional distribution, Table 7.5D yields f_d = 0.87.

5. With 12-ft lanes and 8-ft shoulders, Table 7.5E yields f_w = 1.00.

6. For the operation of passenger cars on upgrades:

$$f_g = 1/[1 + (P_p I_p)]$$
$$f_g = 1/[1 + (0.87)(0.039)]$$
$$f_g = 0.967$$

where:

P_p = 1.00 - 0.13 = 0.87

E = 3.25 (interpolated from Table 7.5F with a 4% upgrade, V = 40 mph and a length of 1¼ miles)

E_o = 1.3 (from Table 7.5F with a 0% upgrade and V = 40 mph)

I_p = 0.02 (E - E_o) = 0.02 (3.25 - 1.3) = 0.039

7. For the operation of heavy vehicles on upgrades:

$$f_{HV} = 1/[1 + P_{HV} (E_{HV} - 1)]$$
$$f_{HV} = 1/[1 + (0.13)(3.293 - 1)]$$
$$f_{HV} = 0.770$$

where:

P_{HV} = .10 + .02 + .01 = 0.13

E = 3.25 (from Step #6)

$$P_{T/HV} = \frac{.10}{.10 + .02 + .01} = \frac{.10}{.13} = 0.769$$

$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$

$E_{HV} = 1 + (0.25 + 0.769)(3.25 - 1) = 3.29$

8. From the data in Step #'s 1-7:

   $SF_D = 2800 \times (v/c)_D \times f_d \times f_w \times f_g \times f_{HV}$
   $SF_D = (2800)(1.00)(0.87)(1.00)(0.967)(0.770)$
   $SF_D = 1814\ vph$

9. The DHV is calculated:

   $DHV = AADT \times K = 4000 \times .125$
   $DHV = 500\ vph$

10. With a DHV = 500 and PHF = 0.85, the equivalent hourly flow rate is:

    $v = V/PHF = 500/0.85$
    $v = 588\ vph$

11. From Step #'s 8 and 9, SF_D is greater than the actual flow rate. Therefore, the LOS on the grade is better than LOS E; therefore, a truck-climbing lane is not warranted based on this criteria.

12. Based on the calculations for LOS A through LOS D, the actual LOS on the upgrade is LOS B. This is one LOS worse than the LOS on the approaching highway segment. Therefore, a truck-climbing lane is not warranted based on this criteria.

Figure 7.5B presents the completed worksheet for the Example.

* * * * * * * * * *

7.5.2.2 Multilane Highways

Climbing lanes on multilane highways are not as easily justified as those on two-lane facilities because of the operational advantage of multilane highways; i.e., a passenger car can pass a slow-moving truck without occupying an opposing lane of travel. As indicated in Section 7.5.1, ODOT has adopted criteria to warrant a truck-climbing lane on multilane highways. These are based on the critical length of grade and on the LOS on the upgrade.

The calculation of LOS for an upgrade on multilane highways is similar to that on two-lane highways (see Section 7.5.2.1). However, the various adjustment factors to calculate the service flow rate differ. This reflects the operational differences between multilane and two-lane facilities. The designer should reference the 1985 *Highway Capacity Manual* for the detailed capacity methodology for specific upgrades on multilane highways.

7.5.3 Design

Table 7.5G summarizes the design details for a truck-climbing lane. One design element is the determination of the beginning and ending of the full-width lane. As indicated in Table 7.5G, this should be determined by the acceleration and deceleration characteristics of the truck from Figures 7.5C and 7.5D. The following example illustrates how to use the figure.

* * * * * * * * * *

Example 7-5

Given:

Level Approach
G_1 = +3% for 700 ft (VPI to VPI)
G_2 = +5% for 4000 ft (VPI to VPI)
G_3 = -2% beyond the composite upgrade (G_1/G_2)
V = 60 mph (design speed)

WORKSHEET FOR SPECIFIC GRADES

Page 1

Site Identification: Example 7-4 Date: ______ Time: ______

Name: ______ Checked by: ______

I. GEOMETRIC DATA

NORTH

Shoulder 8 ft

24 ft

Shoulder 8 ft

Design Speed: 60 mph
Grade: 4 %, 1/4 mi
% No Passing Zones: 50 %

II. TRAFFIC DATA

Total Volume, Both Dir. 500 vph
Flow Rate = Volume ÷ PHF
588 = 500 ÷ 0.85

Directional Distribution: 60/40
Traffic Composition: 10 %T, 2 %RV, 1 %B
PHF: 0.85

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1 / [1 + P_p I_p]$
$I_p = 0.02 (E - E_o)$

$f_{HV} = 1 / [1 + P_{HV} (E_{HV} - 1)]$
$E_{HV} = 1 + (0.25 + P_{T/HV}) (E - 1)$

Speed (mph)	P_p	I_p	E	E_o	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E	f_{HV}
55	0.87	.249	14.55	2.1	.822	.13	14.8	.769	14.55	.358
52.5		.130	8.30	1.8	.898		8.4		8.30	.510
50		.087	5.95	1.6	.930		6.0		5.95	.606
45		.051	3.95	1.4	.958		4.0		3.95	.719
40		.039	3.25	1.3	.967		3.3		3.25	.770
30		.029	2.75	1.3	.975		2.8		2.75	.810

IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800 ×	v/c ×	f_d ×	f_w ×	f_g ×	f_{HV}
55 (LOS A)	122	2,800	0.170	0.87	1.00	.822	.358
52.5	335	2,800	0.300			.898	.510
50 (LOS B)	693	2,800	0.505			.930	.606
45 (LOS C)	1451	2,800	0.865			.958	.719
40 (LOS D)	1814	2,800	1.000			.967	.770
30	1924	2,800	1.000			.975	.810

Source: (2)

Figure 7.5B

Table 7.5G

DESIGN CRITERIA FOR TRUCK-CLIMBING LANES

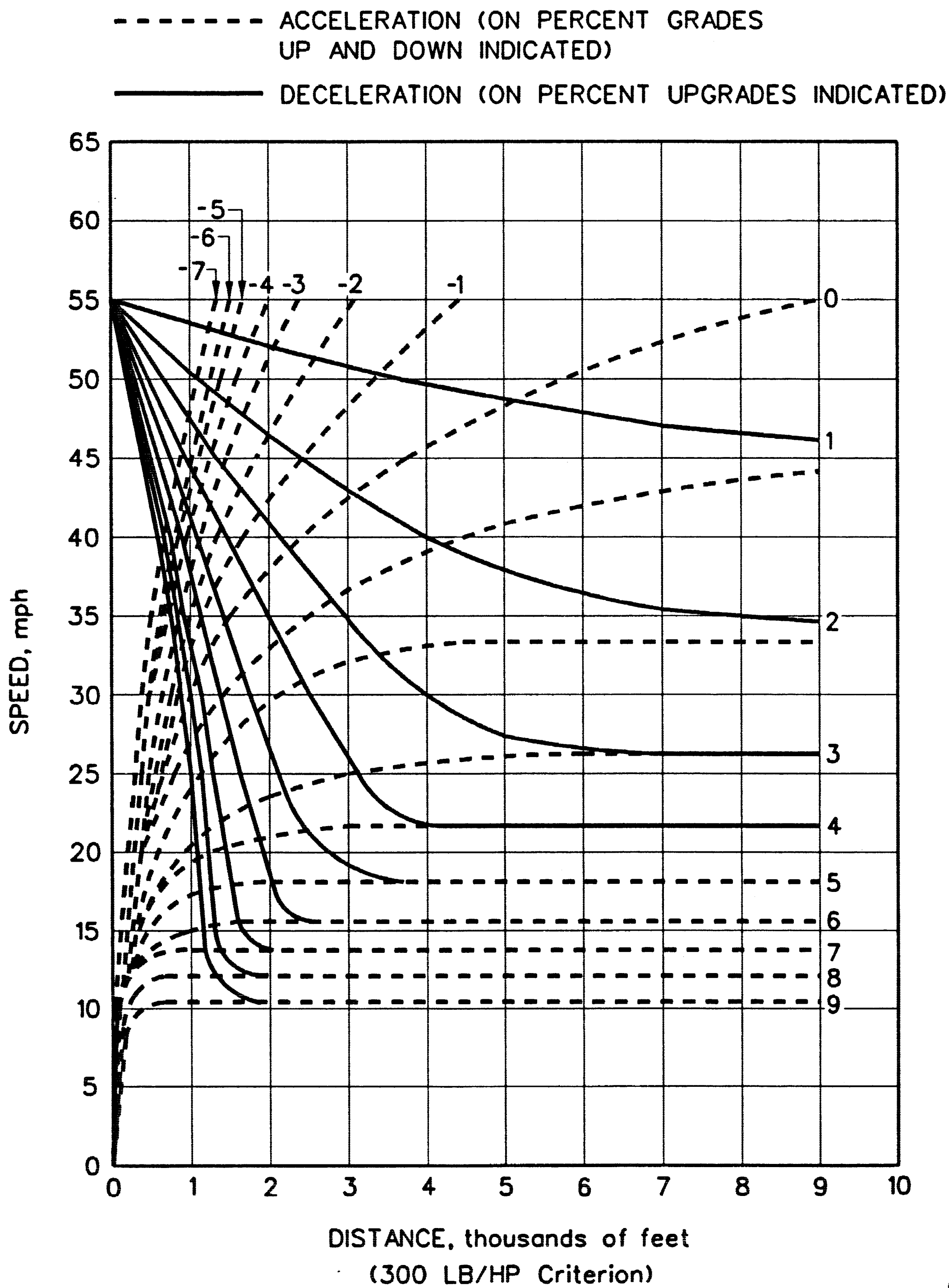
DESIGN ELEMENT	DESIRABLE	MINIMUM
Lane Width	Same as approach roadway.	Same as approach roadway.
Shoulder Width	Same as approach roadway.	Freeway: 6′ paved Non-Freeway: 4′ paved
Cross Slope on Tangent	Same as adjacent travel lane.	Same as adjacent travel lane.
Superelevation	(1)	(1)
Beginning of Full-Width Lane	Near the VPT of the grade.	To where truck speed is 10 mph below highway design speed or 45 mph, whichever is less. (2)
End of Full-Width Lane (3)	To where truck has reached highway design speed or 55 mph, whichever is less.	To where truck has reached 10 mph below highway design speed or 45 mph, whichever is less. (2)
Entering Taper	25:1	150′
Exiting Taper	50:1	200′
Minimum Full-Width Length	N/A	1000′

Notes:

(1) For horizontal curves on truck-climbing lanes, the designer will determine the proper superelevation of the climbing lane on the high side by reading into the applicable e_{max} table for V=40 mph or the design speed, whichever is less. See Chapter Six. This reflects the slower operating speeds of the truck-climbing lane. The maximum difference in cross slope between the travel lane and truck-climbing lane is 4%.

(2) Use Figures 7.5C and 7.5D to determine truck deceleration and acceleration rates.

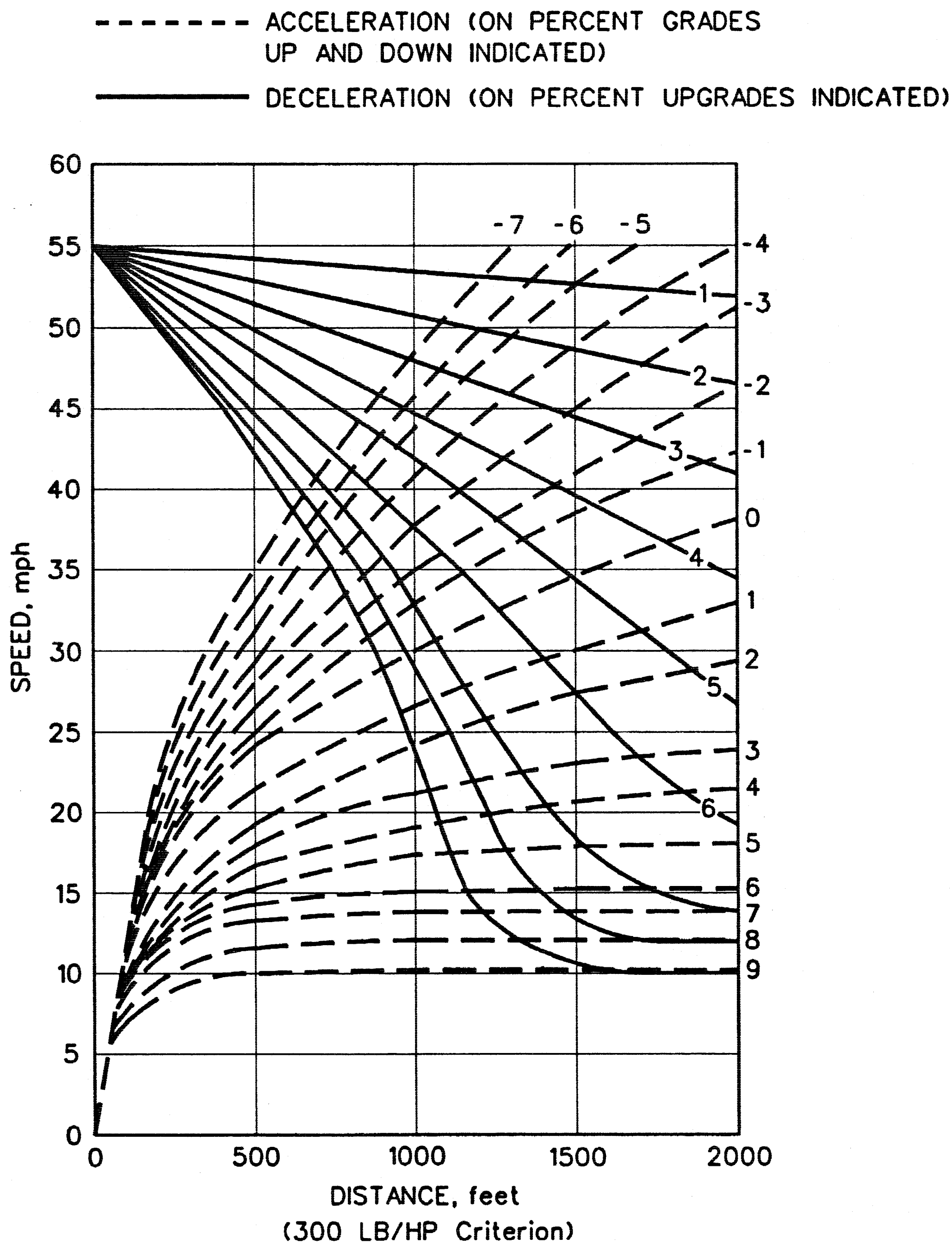
(3) The designer should also consider the available sight distance to the point where the truck will merge back into the through travel lane. At a minimum, this will be stopping sight distance. The driver should have decision sight distance available to the merge point to safely complete the maneuver, especially where the merge is on a horizontal curve and/or on an upgrade.



Source: (1)

Note: For design speeds above 55 mph, use an initial speed of 55 mph. For design speeds 55 mph and below, use the design speed as the initial speed.

PERFORMANCE CURVES FOR HEAVY TRUCKS
(0-9000 Feet)
Figure 7.5C



Source: (1) Revised

Note: For design speeds above 55 mph, use an initial speed of 55 mph. For design speeds 55 mph and below, use the design speed as the initial speed.

PERFORMANCE CURVES FOR HEAVY TRUCKS
(0-2000 Feet)

Figure 7.5D

Problem:

Using the minimum criteria from Table 7.5G, determine the beginning and ending points of the full-width climbing lane.

Solution:

The beginning of the lane will be determined by the point at which the truck has decelerated to a speed of 45 mph. Using Figure 7.5D, the truck will decelerate to a speed of 50 mph on the 3% upgrade over a distance of 700 ft. Using Figure 7.5D again, determine the distance along the +5% line from 50 mph to 45 mph. This occurs from approximately 400 to 750 on the horizontal axis. Therefore, the truck will decelerate from 50 mph to 45 mph on the +5% upgrade in a distance of 350 ft (750-400) beyond the VPI between the 3% and 5% upgrades. This is the point where the full-width truck-climbing lane should begin. Further note that, from Figure 7.5C, the grade is long enough that the truck will reach a final speed of 18 mph (i.e., the terminal speed for a 5% upgrade).

The end of the lane will be determined by the point at which the truck has accelerated to a speed of 45 mph. The truck will have a speed of 18 mph as it enters the 2% downgrade at the VPI. Read into Figure 7.5D at the 18-mph point on the vertical axis over to the dashed -2% line. This is at approximately 200 ft along the horizontal axis. The -2% line is followed up to 45 mph, which is approximately 1800 ft along the horizontal axis. Therefore, the truck will require 1600 ft (1800-200) from the VPI to reach 45 mph. The truck will require approximately an additional 1200 ft to reach 55 mph (the desirable criteria).

* * * * * * * * * * *

7.5.4 Downgrades

Truck lanes on downgrades are not typically considered. However, steep downhill grades may also have a detrimental effect on the capacity and safety of facilities with high traffic volumes and numerous heavy trucks. Although specific criteria have not been established for these conditions, trucks descending steep downgrades in low gear may produce nearly as great an effect on operations as an equivalent upgrade. Therefore, consideration should be given to providing a truck lane for downhill traffic on a site-by-site basis.

7.6 MATHEMATICAL DETAILS FOR VERTICAL CURVES

This Section presents mathematical details used by ODOT for various applications to the design of vertical curves. Table 7.6A summarizes the figures in Section 7.6.

Table 7.6B presents definitions for the basic geometric elements of vertical curves.

Table 7.6A

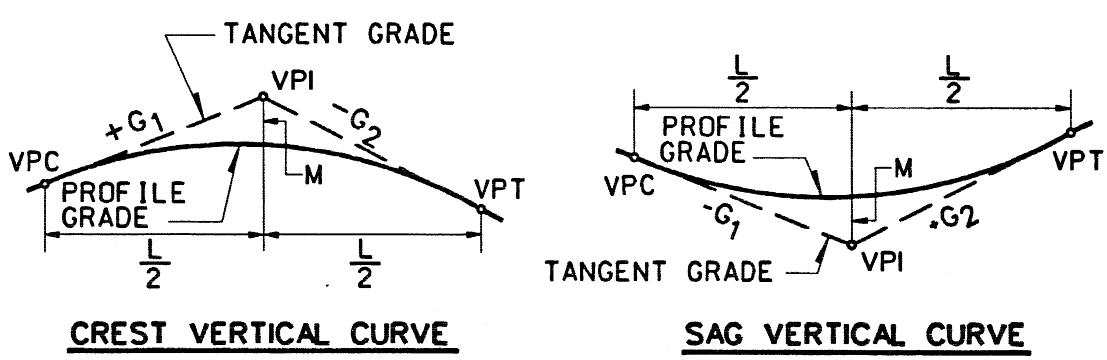
MATHEMATICAL DETAILS FOR VERTICAL CURVES

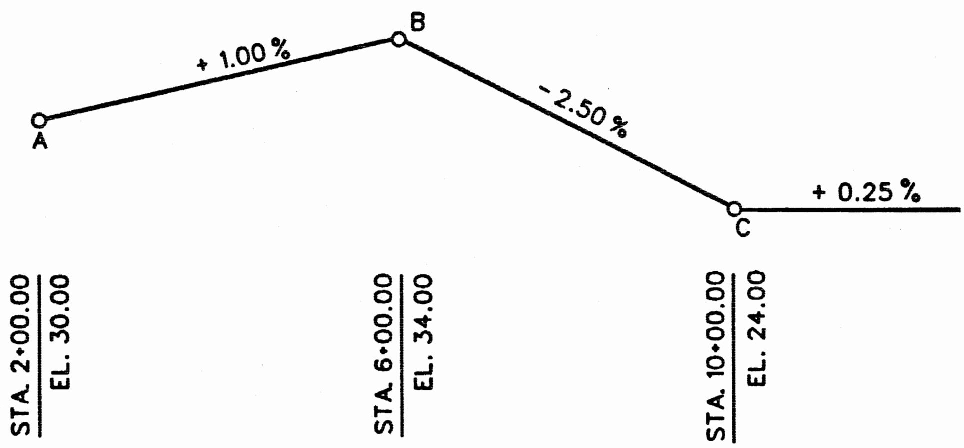
Figure Number	Figure Title
Figure 7.6A	Running Grade
Figure 7.6B	Vertical Curve Offsets
Figure 7.6C	Vertical Curve Computations
Figure 7.6D	Vertical Curve Computations
Figure 7.6E	Vertical Curve Computations (Distance to Point on Curve)
Figure 7.6F	Vertical Curve Computations (Gradient to Point on Curve)
Figure 7.6G	Vertical Curve Extension
Figure 7.6H	Unsymmetrical Vertical Curve

Table 7.6B

VERTICAL CURVE DEFINITIONS

ELEMENT	ABBREVIATION	DEFINITION
Vertical Point of Curvature	VPC	The point at which a tangent grade ends and the vertical curve begins.
Vertical Point of Tangency	VPT	The point at which the vertical curve ends and the tangent grade begins.
Vertical Point of Intersection	VPI	The point where the extension of two tangent grades intersect.
Grade	G_1, G_2	The rate of slope between two adjacent VPI's expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in ft for each 100 ft of horizontal distance. Upgrades in the direction of stationing are identified as plus (+). Downgrades are identified as minus (-).
External Distance	M	The vertical distance (offset) between the VPI and the roadway surface along the vertical curve.
Algebraic Difference in Grade	A	The value of A is determined by the deflection in percent between two tangent grades.
Length of Vertical Curve	L	The horizontal distance in ft from the VPC to the VPT.





FORMULA:

$$\frac{\text{DIFFERENCE IN ELEV. BETWEEN ANY KNOWN STATIONS ON TANGENT}}{\text{DISTANCE BETWEEN THOSE STATIONS}} = \%\ \text{GRADE}$$

EXAMPLE:

$$\text{GRADE A TO C: } \frac{\text{ELEV. AT C - ELEV. AT A}}{\text{DISTANCE A TO C}} = \frac{34.00-30.00}{4.0} = 1.0\%$$

RUNNING GRADE

Figure 7.6A

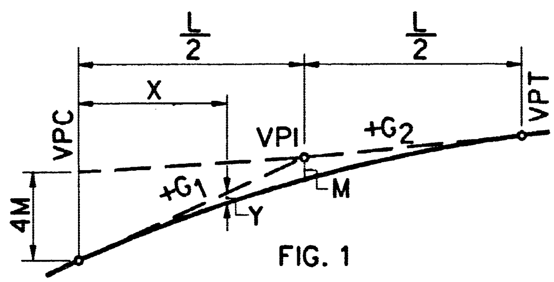


FIG. 1

CREST VERTICAL CURVE

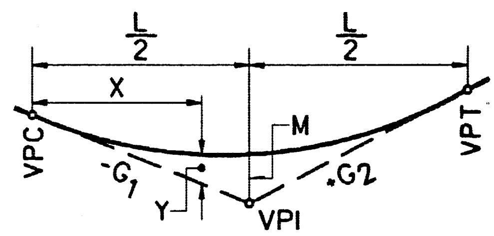


FIG. 2

SAG VERTICAL CURVE

Formulas:

$$M = \frac{A}{8}(L)$$

$$Y = \frac{4M}{L^2}(X^2) \text{ or } \frac{A}{2L}(X^2)$$

Where:

- M = Maximum offset, ft
- A = Algebraic diff. of grades (G_1 and G_2), %
- L = Length of curve in stations
- Y = Offset at distance X from VPC or VPT, ft

Given:

In Figure 1, G_1 = +4.45% and G_2 = +1.15%. The length of curve L = 6 Sta. The distance X = 1.74 Sta.

Required: M and offset Y.

$$M = \frac{3.3}{8}(6) = 2.475'$$

$$Y = \frac{4 \times 2.48}{6^2}(1.74)^2 = 0.835'$$

Given:

In Figure 2, G_1 = -4.55% and G_2 = +3.00%. The length of curve L = 5 Sta. The distance X = 1.75 Sta.

Required: M and offset Y.

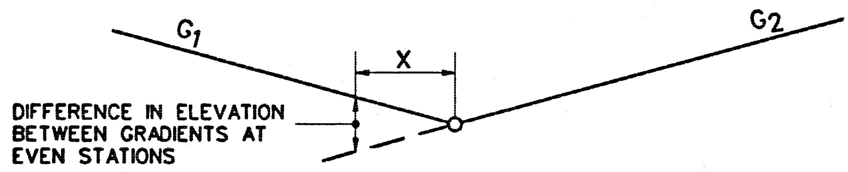
$$M = \frac{7.55}{8}(5) = 4.719'$$

$$Y = \frac{4 \times 4.72}{5^2}(1.75)^2 = 2.31'$$

VERTICAL CURVE OFFSETS

Figure 7.6B

COMPUTATIONS FOR ODD PI



The distance X from any even Station to an odd PI is equal to:

$$\frac{\textit{Diff. in Elev. at Even Sta.}}{\textit{Algebraic Diff. of Gradients}} \; x \; 100$$

Given: G_1 = -2.0% and G_2 = +3.0%. Difference in Elevation of 2.5 ft between gradients at Sta. 100+00.00.

Required: Distance X

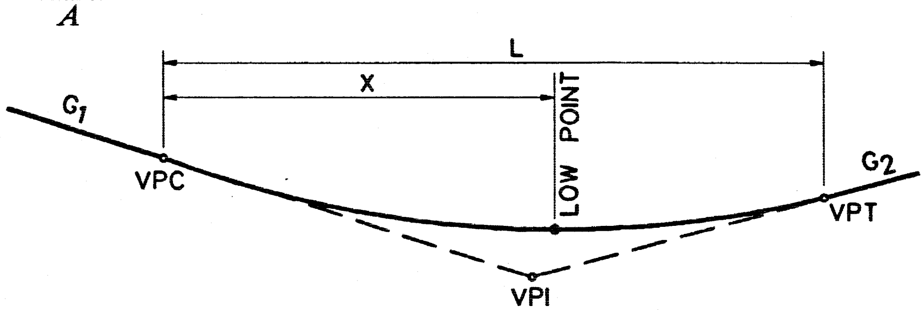
$$X = \frac{2.5}{5}\,(100) = 50' \qquad \text{VPI is at Sta. } 100 + 50.00$$

COMPUTATION OF LOWEST OR HIGHEST POINT ON VERTICAL CURVE

X = Distance to lowest or highest point from VPC
G_1 = % of grade back of VPI
L = Length of vertical curve in stations
A = Algebraic difference in grades

Then:

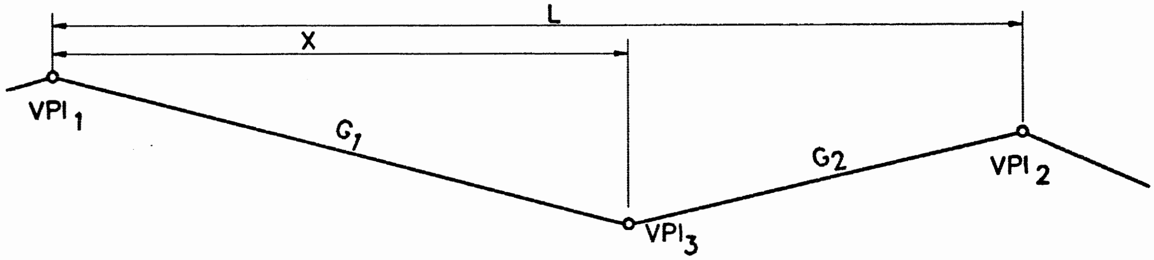
$$X = \frac{G_1\,L}{A}$$



VERTICAL CURVE COMPUTATIONS

Figure 7.6C

TO FIND STATION OF VPI WHERE TWO KNOWN GRADES INTERSECT



Given: Station and Elevation at VPI_1
Station and Elevation at VPI_2
G_1, G_2 (in %)

Find: Station and Elevation at VPI_3

Solution: Let L = Distance between VPI_1 and VPI_2. Then:

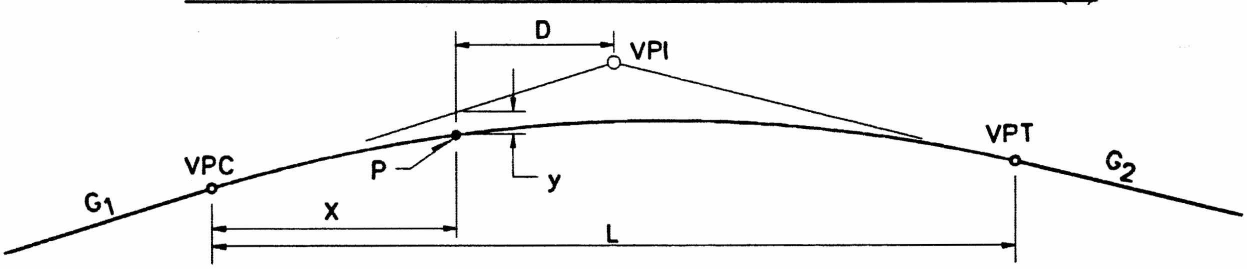
$$X = \frac{(El.\ VPI_2 - El.\ VPI_1) - G_2 L}{G_1 - G_2}$$

Let X = Distance between VPI_1 + VPI_3. Then:

$$El.\ at\ VPI_3 = G_1 X + El.\ VPI_1$$

All distances in Stations.

TO PASS A VERTICAL CURVE THROUGH A GIVEN POINT (P)



A = Algebraic difference in grades
y = Vertical curve correction at point "P"
X = Distance from "P" to VPC
D = Distance from "P" to VPI
L = Length of vertical curve

Given: A, y, X

Find: L in Stations

Solution:

1. $AX^2 - 4yX - 4yD = 0$
2. Substitute knowns into above equation and solve for X:
$$X = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$
3. L = (2) (D + X)
4. Substitute X into the above equation and solve for L

Note: All distances in Stations (except distance y).

VERTICAL CURVE COMPUTATIONS

Figure 7.6D

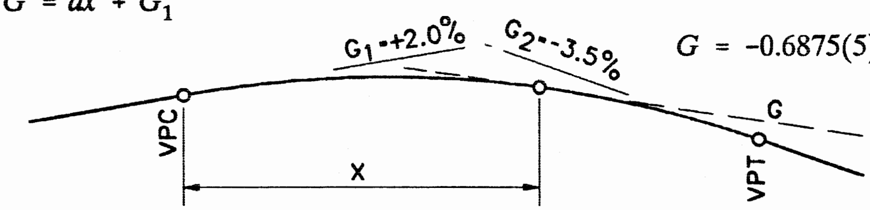
TO FIND % OF GRADE AT ANY POINT ON A VERTICAL CURVE

L = Length of VC in Stations
x = Distance from VPC in Stations

$$a = \frac{G_2 - G_1}{L}$$

Gradient at a point on curve x distance from VPC.

$$G = ax + G_1$$



Example: Find gradient at a point 500 ft from VPC for an 800-ft vertical curve.

$G_1 = +2.0\%$ $L = 8$
$G_2 = -3.5\%$ $x = 5$

$$a = \frac{-3.5 - 2.0}{8} = -0.6875$$

$$G = -0.6875(5) + 2.0 = -1.4375\%$$

TO FIND A POINT ON CURVE WHERE A GIVEN % OF GRADE OCCURS

Distance x from VPC to point of selected gradient.

$$x = \frac{G_1 - G}{a}$$

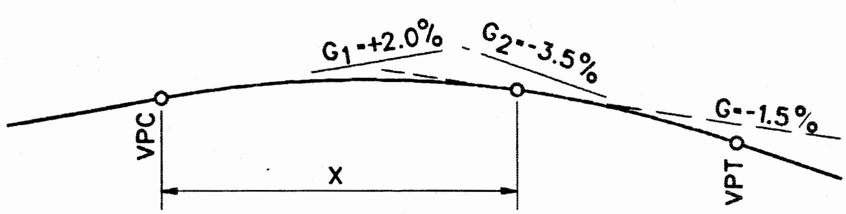
Example: Find point on curve where gradient is -1.5%.

$G_1 = +2.0\%$ $L = 8$
$G_2 = -3.5\%$ $G = -1.5\%$

$$a = \frac{-3.5 - 2.0}{8} = -0.6875$$

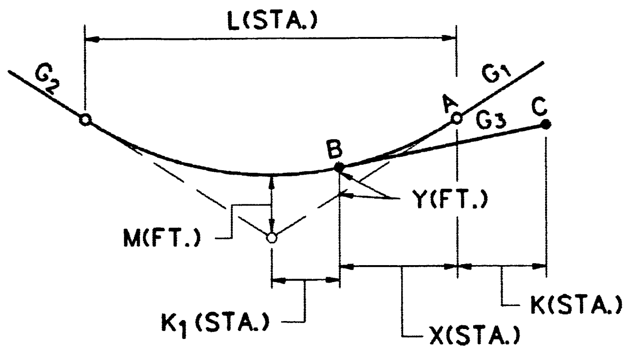
$$x = \frac{2.0 + 1.5}{0.6875}$$

$$x = 5.0909 \textit{ Stations}$$



VERTICAL CURVE COMPUTATIONS
(Distance to Point on Curve)

Figure 7.6E



PROBLEM 1:

Given: L, G_1, G_2, M, K, El @A & El @C

Find: X, Y, G_3 and El @B

Let:

$$m = G_1$$
$$n = G_1 - G_2$$
$$D = 4M - Ln$$
$$E = -LnK$$
$$F = L^2mK + L^2(El\,@A - El\,@C)$$

Solution:

1. $DX^2 + EX + F = 0$

$$\therefore X = \frac{-E \pm \sqrt{E^2 - 4DF}}{2D}$$

2. $G_3 = \dfrac{Lm - nX}{L}$

3. $Y = \dfrac{4MX^2}{L^2}$

4. $El\,@B = El\,@A + Y - G_1X$

PROBLEM 2:

Given: G_1, G_2, K_1, Y and El @B

Find: L, M

Let:

$$n = G_1 - G_2$$

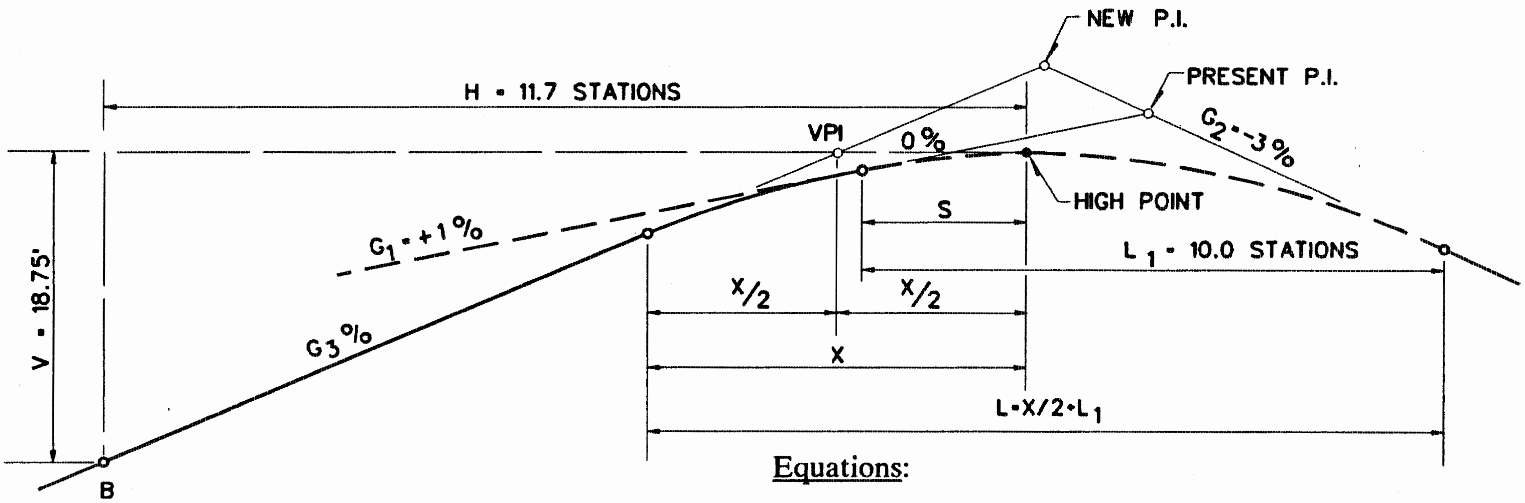
Solution:

1. $L = 2K_1 + \dfrac{4Y}{n} + (\dfrac{4}{n}\sqrt{Y^2 + YK_1n})$

2. $M = \dfrac{Ln}{8}$

VERTICAL CURVE COMPUTATIONS
(Gradient to Point on Curve)

Figure 7.6F



Given:

A = Algebraic difference in grades (G_1 & G_2), %
L_1 = Length of existing vertical curve in Stations

High Point:

$$S = \frac{G_1 L_1}{A}$$

B = Any known point on line G_3
H = Horizontal distance in stations
V = Vertical Height in ft

Find:

X (distance in stations)
G_3 of grade

Equations:

$$G_3 = \frac{AX}{L_1}$$

$$V = G_3 \left(H - \frac{X}{2}\right)$$

$$X = H - \frac{\sqrt{(AH/L_1)^2 - 2(AV/L_1)}}{(A/L_1)}$$

Example:

$V = 18.75\ ft$, $H = 11.7\ Sta.$, $L_1 = 10.0\ Sta.$
$A/L_1 = 0.4$, $G_1 = +1.0\%$, $G_2 = -3.0\%$

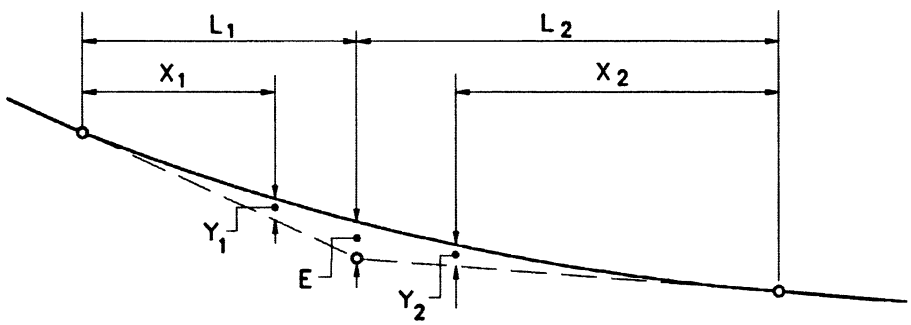
$$X = 11.70 - \frac{\sqrt{(0.4 \times 11.7)^2 - 2(0.4 \times 18.75)}}{0.4}$$

$X = 5.1319\ Sta.$

$G_3 = 5.1319 \times 0.4 = 2.05276\%$

VERTICAL CURVE EXTENSION

Figure 7.6G



A = Algebraic difference in grades
E = Middle ordinate
Y_1 = Offset at any point X_1
Y_2 = Offset at any point X_2

$$E = \frac{L_1 \times L_2 \times A}{2(L_1 + L_2)}$$

$$Y_1 = E\left(\frac{X_1}{L_1}\right)^2$$

$$Y_2 = E\left(\frac{X_2}{L_2}\right)^2$$

UNSYMMETRICAL VERTICAL CURVE

Figure 7.6H

7.7 REFERENCES

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

2. *Highway Capacity Manual*, Transportation Research Board, 1986.

3. *Methods for Predicting Truck Speed Loss on Grades*, Federal Highway Administration, Report No. FHWA/RD-86/059, October 1986.

4. *New Methods for Determining Requirements for Truck-Climbing Lanes*, Federal Highway Administration, Publication No. FHWA-IP-89-022, September 1989.

5. *Policies and Procedures*, Volumes 1 and 2, Office of Design, Oklahoma Department of Highways, 1969.

6. *Manual on Uniform Traffic Control Devices*, Federal Highway Administration, 1988.

Chapter Eight

Cross Section Elements

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Chapter Eight

CROSS SECTION ELEMENTS

Chapter Twelve "Geometric Design Tables (New Construction/Reconstruction)," provides numerical criteria for various cross section elements on new construction and reconstruction projects. Chapter Thirteen provides criteria for cross section elements on existing highways. Chapter Eight will provide additional guidance which should be considered in the design of these cross section elements.

8.1 ROADWAY SECTION

Figure 8.1A presents schematics of the basic elements of the roadway section. These elements are discussed in the following sections.

8.1.1 Travel Lanes

8.1.1.1 Width

Travel lane widths can vary between 9 and 12 ft, depending upon the functional classification, traffic volumes, design speed, rural/urban location and project scope of work. The tables in Chapters Twelve and Thirteen provide specific criteria for travel lane widths.

8.1.1.2 Cross Slopes (State Highways)

Surface cross slopes are required for proper drainage of through travel lanes on tangent sections. This reduces the hazards of wet pavements by quickly removing water from the surface, and it reduces the likelihood of ponding. On State highways, the following will apply:

1. Typical. The travel lane cross slope can range from 1.5% to 3%, depending on surface width and drainage considerations, with 2% typically used.

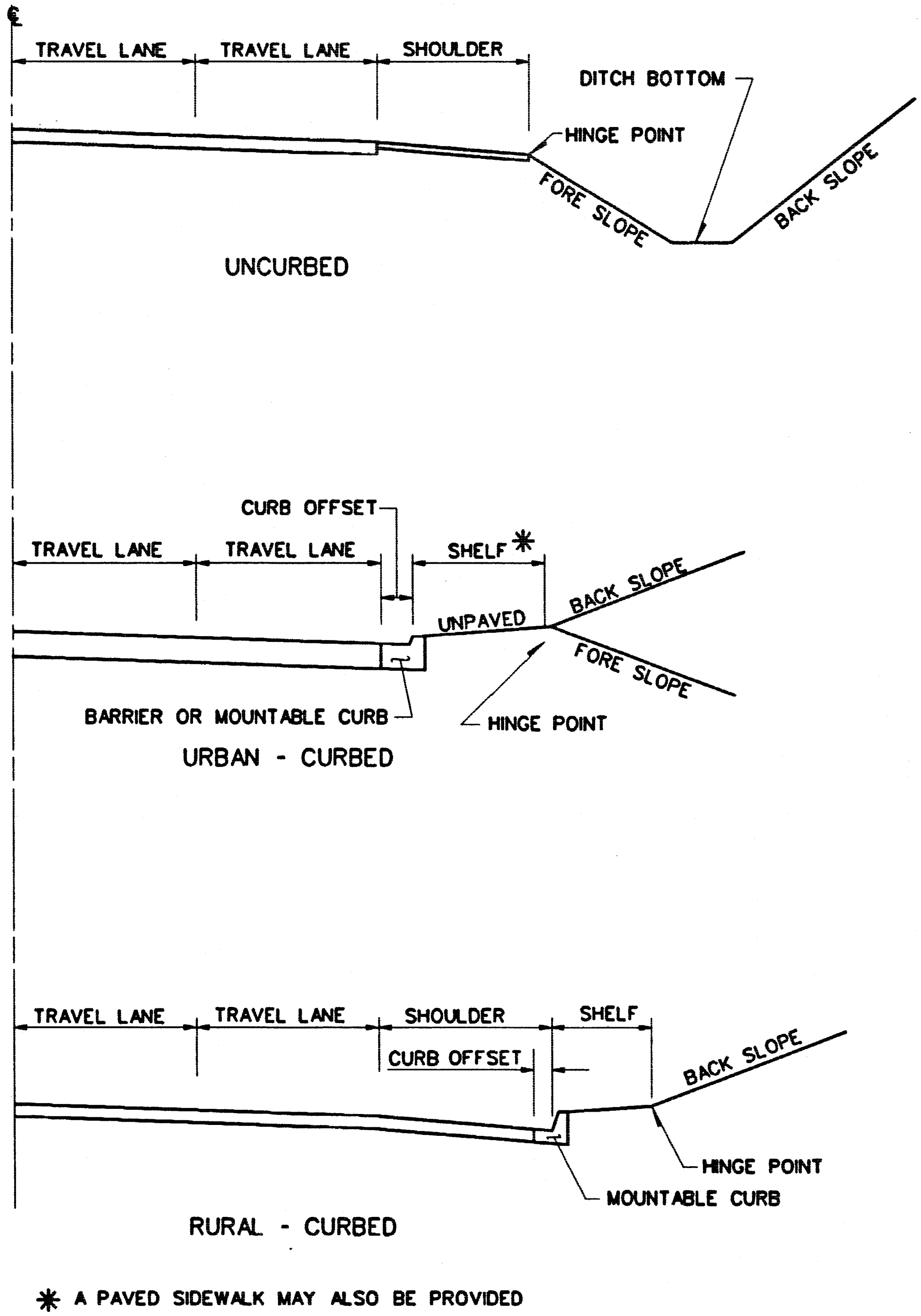
2. Open-Graded. Where an open-graded friction course is used, the cross slope should be a minimum of 2%.

3. Multilane/Curbed. On multilane facilities with curbs, the typical cross slope is 3% for any travel lanes adjacent to the curb.

4. Divided Facilities. For divided facilities with two lanes in each direction, a uniform cross slope sloping away from the median is typically used (see Figure 8.8A). With three lanes in each direction, either a uniform cross slope or a crowned slope may be used. The crown point may be at either edge of an interior travel lane.

8.1.1.3 Cross Slopes (Non-State Highways)

For non-State highways, the travel lane cross slopes will vary depending upon the pavement surface and local practices. For paved surfaces (including chip seal), the typical cross slope is the same as for State highways (i.e., 2%). For unpaved surfaces, the cross slopes



ROADWAY SECTION
(Definitions)

Figure 8.1A

can range from 2% to 8%. Gravel surfaces generally range from 2% to 4% with 3% typically used. Dirt surfaces typically range from 4% to 6% with a maximum acceptable cross slope of 8%.

8.1.2 Shoulders/Curb Offsets

8.1.2.1 Definitions

The following definitions apply to the term "shoulder":

1. Shoulder. The portion of the roadway contiguous to the travel lane. For curbed facilities, the term "shoulder" will apply when this width is 4 ft or more.

2. Graded Shoulder Width. The width of the shoulder measured from the edge of travelway to the intersection of the shoulder slope and fill slope planes. See Figure 8.1B.

3. Usable Shoulder Width. The width of the shoulder that can be used by a driver for emergency parking or stopping. Figure 8.1B illustrates the definition for a usable shoulder width for various side slope conditions.

4. Curb Offset. On curbed facilities, the portion of the roadway section from the edge of travel lane to the gutter line when that distance is less than 4 ft.

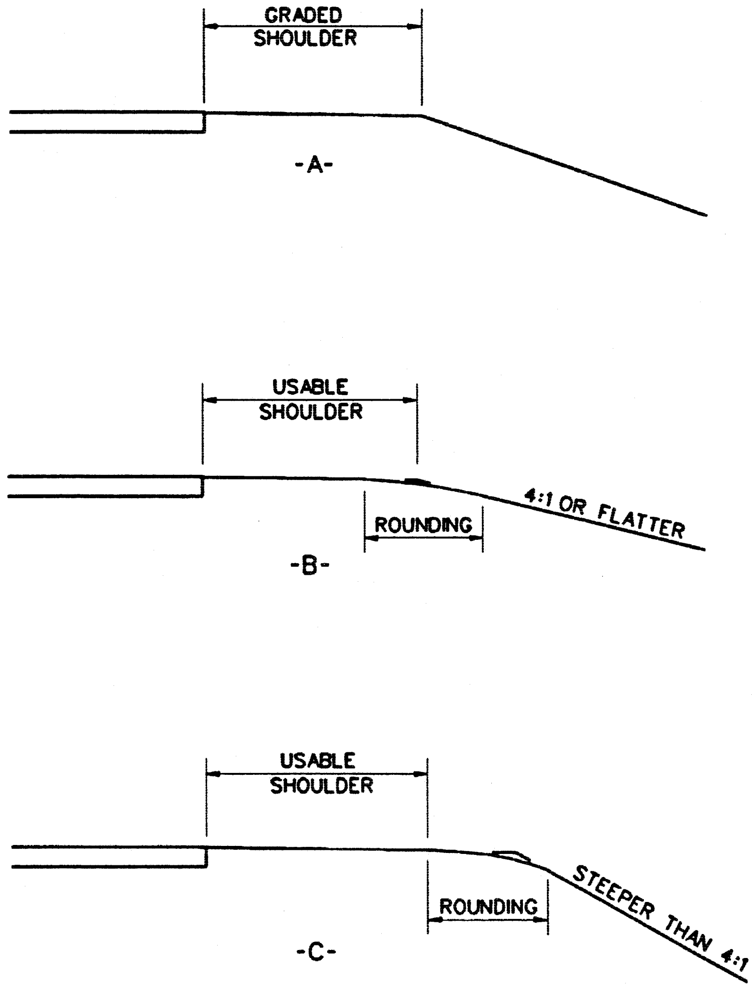
8.1.2.2 Functions

Shoulders serve many functions. The wider the shoulder, the greater the benefits, including:

1. providing structural support for the travelway which prevents, for example, pavement edge dropoffs;
2. increasing highway capacity;
3. encouraging uniform travel speeds;
4. providing space for emergency and discretionary stops;
5. improving roadside safety by providing more recovery area for run-off-the-road vehicles;
6. providing a sense of openness;
7. improving sight distance around horizontal curves;
8. enhancing highway aesthetics;
9. facilitating maintenance operations (e.g., snow storage);
10. providing additional lateral clearance to roadside appurtenances (e.g., guardrail, traffic signals);
11. facilitating pavement drainage;
12. providing space for pedestrian and bicycle use; and
13. providing space for bus stops.

8.1.2.3 Widths

Shoulder widths will vary according to functional classification, traffic volumes, urban/rural location, curbed/uncurbed facilities and the project scope of work. The tables in Chapters Twelve and Thirteen present the shoulder width criteria for the various conditions. To provide additional offset, all shoulder widths should desirably be increased by 2 ft when a barrier is present.



Source: (1)

GRADED AND USABLE SHOULDERS
(Definitions)

Figure 8.1B

8.1.2.4 Surface Type

Chapters Twelve and Thirteen indicate the shoulder surface type for the various highway conditions. The following summarizes general ODOT and local practices:

1. Multilane Divided. Both left and right shoulders will be paved on all multilane divided facilities.

2. Rural Principal Arterials. Shoulders will be paved.

3. Rural Other Arterial Highways. Shoulders will typically be paved where the DHV > 200 vph. Where the DHV < 200, 2 ft may typically be paved and the remainder sodded.

4. Rural Collectors (State Highways). Where the DHV > 200 vph, 4 ft will typically be paved and the remainder sodded. Where the DHV < 200, 2 ft may typically be paved and the remainder sodded.

5. Rural Non-State Roads (Collector/Local). Shoulders will typically be either gravel or sod.

6. Urban/Suburban Curbed Facilities. Curb and gutter (curb offset) is typically used. See Figure 8.1A.

7. Urban/Suburban Uncurbed Arterials. Shoulders will typically be paved.

8. Urban/Suburban Uncurbed Collectors. Desirably, 2 ft will typically be paved and the remainder sodded. Fully sodded shoulders are acceptable.

9. Urban/Suburban Uncurbed Local Streets. Shoulders will typically be either chip seal, gravel or sod.

Where a barrier is placed on a shoulder which is not paved, the full shoulder width (plus the desirable 2-ft additional width) in the barrier area should be paved.

8.1.2.5 Cross Slopes

The normal cross slope of the shoulder depends on the type and width of shoulder. The tables in Chapters Twelve and Thirteen provide the cross slope used for each classification. The following summarizes ODOT and local practices:

1. Full-width paved shoulders are typically 3% - 4% on freeways and 2% - 4% on all other facilities.

2. Combination paved/sodded shoulders are typically sloped at a uniform rate of 2% - 4%.

3. Gravel shoulders typically range from 4% to 6% cross slopes.

4. Sodded shoulders typically range from 4% to 8% cross slopes.

5. The cross slope of a curb offset will be the same as that of the adjacent travel lane.

8.1.3 Auxiliary Lanes

Auxiliary lanes are any lanes beyond the basic through travel lanes which are intended for use by vehicular traffic for specific functions. Auxiliary lanes include:

1. left- and right-turn lanes at intersections,

2. truck-climbing lanes,

3. acceleration/deceleration lanes at interchanges,

4. weaving lanes within an interchange,

5. continuous auxiliary lanes between two closely spaced interchanges, and

6. two-way, left-turn lanes.

Desirably, auxiliary lanes will be the same width as the adjacent through lanes, although in many cases a lesser width may be appropriate. The tables in Chapters Twelve and Thirteen present specific criteria for auxiliary lanes. The tables also provide criteria for shoulder widths adjacent to auxiliary lanes.

The cross slope for an auxiliary lane will typically be the same as the adjacent through lane.

8.1.4 Parking Lanes

Depending upon the highway conditions, on-street parking may be allowed. Section 17.1 discusses those factors which should be considered for the introduction or retention of on-street parking, and it discusses specific design criteria (e.g., for angle parking). Chapters Twelve and Thirteen present widths for parking lanes.

8.1.5 Curbs and Curbed Sections

Curbs are often used on urban facilities to control drainage, delineate the pavement edge, channelize vehicular movements, control access, limit right-of-way needs, provide separation between vehicles and pedestrians and present an attractive appearance. In urban areas, curbs have a major benefit in containing the drainage within the pavement area and in channelizing or controlling traffic into and out of adjacent properties.

8.1.5.1 Warrants (Curbed Section)

Selecting a curbed section or uncurbed section depends upon many variables, including vehicular speeds, urban/rural location, drainage and construction costs. The following discusses those factors which will determine whether or not a curbed section is warranted:

1. Urban Location. Because of restricted right-of-way and other constraints, curbed sections are typically used in urban areas.

2. Suburban Location. Where design speeds are 50 mph or more, uncurbed sections are typically used. The exceptions listed under #3 for rural locations also apply to high-speed suburban facilities. Where design speeds are less than 50 mph, the use of a curbed or uncurbed section will be made on a project-by-project basis considering right-of-way constraints, drainage, pedestrian activity, channelization needs, driveway access control, etc.

3. Rural Location. The use of curbs on rural highways is usually limited to conditions such as the following:

   a. where a raised median is present;

   b. where there is sufficient development along the highway and there is a need to channelize traffic into and out of properties;

   c. where it is absolutely necessary to control drainage;

   d. where restricted right-of-way provides no room for roadside ditches; and/or

   e. at other sites (e.g., interchanges, intersections) as determined by the Geometric Design Branch.

8.1.5.2 Curb Types

There are two basic types of curbs — mountable and barrier. By definition, mountable curbs have a height of 6 inches or less with a face no steeper than 1 horizontal to 3 vertical. Barrier curbs may range in height between 6 inches and 12 inches with a face steeper than 1 horizontal to 3 vertical.

Figure 8.1C presents the basic curb types used by ODOT. The *ODOT Standard Drawings* provide additional information on the design details and placement for the different curb types. This includes details on driveway radii, construction items, etc.

8.1.5.3 Curb Type Selection

The following discusses those factors which should be considered when selecting a curb type:

1. Material. Concrete curbs are typically used.

2. Outside Curb. Barrier curbs are typically used on the outside of the roadway.

3. Raised Medians. Mountable curbs are typically used with raised medians.

4. Speeds. Barrier curbs may be used where the design speed is 45 mph or less. Mountable curbs may be used at any design speed.

5. Vehicular Encroachment. Where sidewalks, roadside appurtenances, etc., are present, it is desirable to restrict vehicular encroachment beyond the curb. Although no curb type will prohibit encroachments, barrier curbs are superior to mountable curbs. Where vehicular encroachment is permissible or even desirable, mountable curbs should be used.

6. Sidewalks. Where sidewalks are present, barrier curbs are typically used.

7. Local Practices. Where local practices differ from ODOT practices, ODOT criteria should prevail on State highways. On non-State highways, local practices will normally govern.

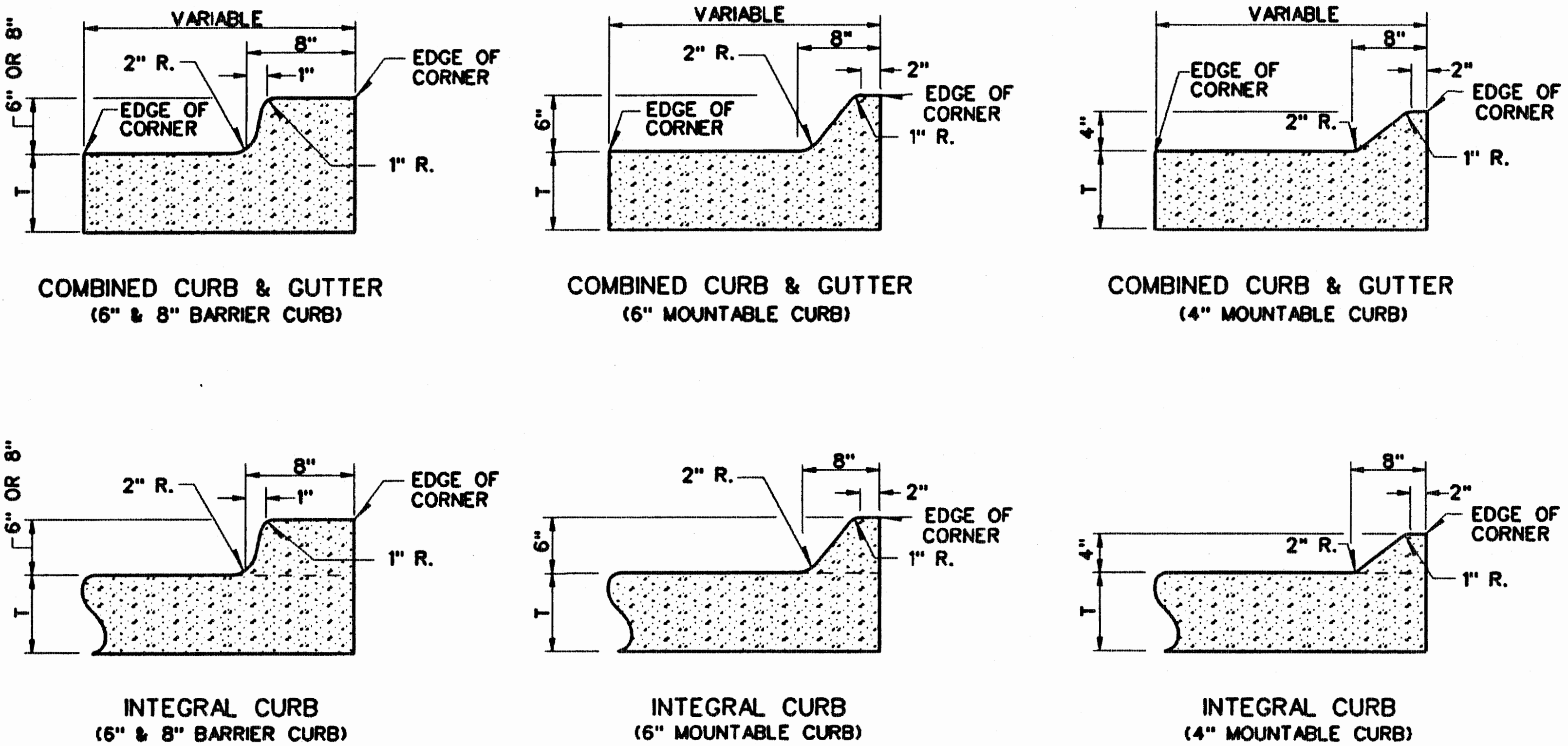
8.1.5.4 Design Considerations

The use of a curbed section requires the consideration and implementation of many design elements. The following discusses these design considerations:

1. Drainage. ODOT practices limit the allowable amount of water ponding on the roadway. A closed drainage system is typically used with curbed sections. The hydraulic analysis will, among other factors, depend on several curb characteristics. These include type of material (concrete or asphalt), cross slopes leading up to the curb, and shape of the curb face. In addition, it may be desirable or necessary to prevent the gutter flow from overtopping the curb. This will affect the selected curb height. See the *ODOT Drainage Manual* for specific criteria and procedures for drainage analysis.

2. Cross Slopes. Where an integral curb and gutter section is used, the cross slope of the gutter is the same as the adjacent pavement surface. Where a separate curb and gutter section is used, the gutter pan may have a steeper cross slope than the adjacent pavement surface.

3. Roadside Safety. The clear zone distance is based on whether a curbed or uncurbed section is used. In addition, the placement of barriers behind curbs must meet certain criteria. Chapter Eleven



Notes:

1. T-dimension equals the thickness shown on typical section.
2. Combined curb and gutter used with asphalt surface. Integral curb used with concrete surface.

Source: (2)

TYPICAL CURB TYPES

Figure 8.1C

discusses roadside safety criteria relative to curbs.

4. Transitions. Figure 8.1D presents design details for transitions from a curbed to an uncurbed section. See the *ODOT Standard Drawings* for additional design details.

5. Future Resurfacing. The designer should consider the likelihood and depth of a future resurfacing course when determining the initial curb height. For example, the curb height may be determined by the sum of the water overtopping depth (based on a drainage analysis) and the future resurfacing depth.

6. Side Slopes. Side slope shape and dimensions in cuts and fills are based on the use of a curbed or uncurbed section. See Section 8.3 for specific criteria.

7. Driveways. The *ODOT Standard Drawings* and *ODOT Policy on Driveway Regulations for Oklahoma Highways* present design details for the use of curbs at driveways.

8. Handicapped Accessibility. Curbs should be designed with curb ramps at all pedestrian crosswalks to provide adequate access for the safe and convenient movement of physically handicapped individuals. Section 17.4 and the ODOT Standard Drawings provide details on the design and location of curb ramps.

8.1.6 Sidewalks

8.1.6.1 Warrants

The following guidance will determine the warrants for sidewalks in the project design:

1. Sidewalks Currently Exist (Roadway and Bridge). Where sidewalks currently exist along a roadway, the sidewalk will normally be reconstructed in kind. If a bridge with an existing sidewalk is replaced or rehabilitated, the sidewalk will normally be retained.

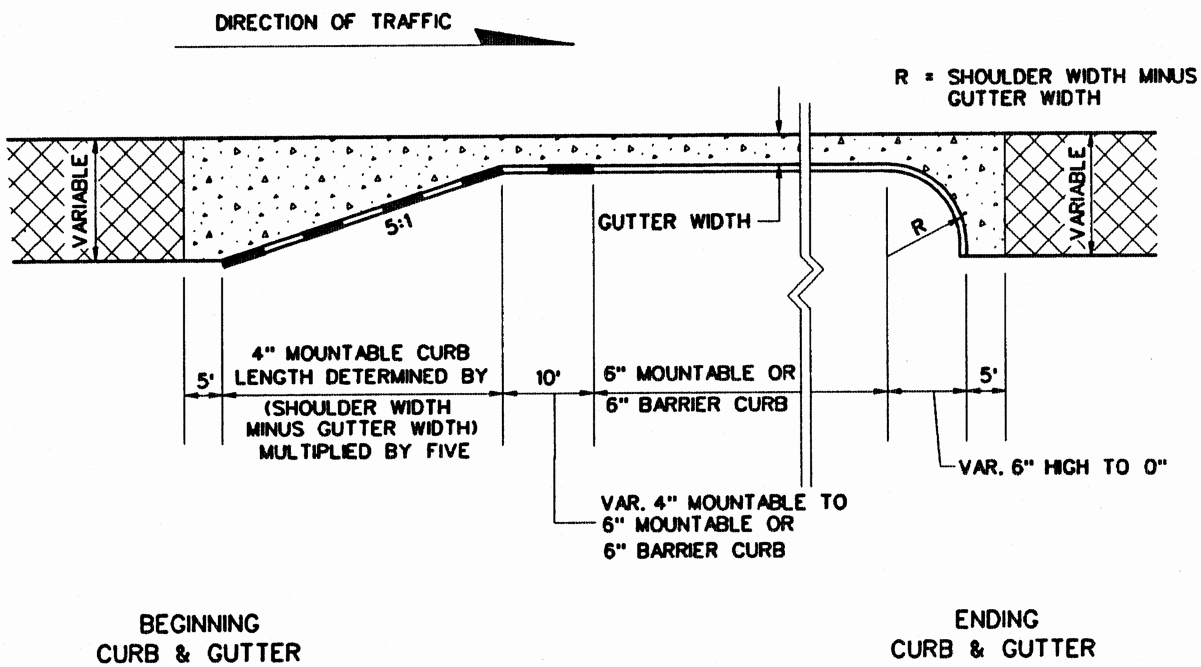
   Existing sidewalks may not be replaced if currently there is little or no pedestrian activity. Therefore, the designer should evaluate the cost effectiveness of replacing existing sidewalks.

2. Bridge Without Sidewalk/Roadway With Sidewalk. If a bridge without a sidewalk will be replaced or rehabilitated and if existing sidewalks approach the bridge, a sidewalk will normally be placed on the replaced or rehabilitated bridge.

3. One Side vs. Two Sides. If a sidewalk only exists on one side of the road or bridge, the designer will review the criteria in Comment No. 4 to determine if the construction of a new sidewalk on the other side is warranted.

4. Sidewalks Currently Do Not Exist. The need for sidewalks will be determined on a case-by-case basis. In general, the designer should consider providing sidewalks along any roadway or on a bridge where pedestrians normally move or would be expected to move if they had a sidewalk available (i.e., a latent demand exists). In rural and suburban areas, sidewalks may be justified at points of community development such as schools, local businesses and industrial plants that result in pedestrian concentrations along the highway. The typical urban locations for sidewalks are in residential or commercial areas.

   The justification for the construction of sidewalks depends in part on the vehicle/



Source: (2)

TRANSITION FROM CURBED TO UNCURBED
(Plan View)

Figure 8.1D

pedestrian hazard (which is governed by the volume of pedestrian and vehicular traffic) and the speed of vehicular traffic. Table 8.1A provides general guidelines on pedestrian traffic volumes which may justify the construction of one sidewalk and/or two sidewalks. The ODOT Planning Division will provide a forecast of pedestrian counts. In conducting the evaluation, the designer should consider the costs of adding a sidewalk (curbs, grading, right-of-way, drainage, utility relocations, etc.) and any other mitigating factors (e.g., environmental impacts, structural impacts on bridges, etc.).

On bridges, intersection sight distance (ISD) may be a special consideration if intersecting roads or driveways are near the end(s) of the bridge. The presence of a sidewalk will improve the ISD for vehicles entering the main road. See Section 9.2 for specific ISD criteria.

If the designer determines that sidewalks are not currently warranted, the designer should consider providing the necessary grading adjacent to the roadway to ease the construction of a sidewalk in the future. Desirably, the graded width will be 8 ft and will have a cross slope of 2%-4%.

8.1.6.2 Sidewalk Design Criteria

In determining the sidewalk design, the designer should consider the following:

1. Widths. Sidewalk widths may vary from 4 ft to 8 ft. A typical sidewalk is 4-ft wide with a 2-ft buffer area between the roadway and sidewalk. If there is no buffer area provided, the sidewalk should be 6-ft wide to accommodate any appurtenances which may be included in the sidewalk (see Comment No. 4).

   High pedestrian volumes may warrant greater widths in, for example, commercial areas and school zones. In these cases, the designer may conduct a detailed capacity analysis to determine the sidewalk width. Chapter Thirteen of the *Highway Capacity Manual* should be used for this analysis.

2. Bicycles. If a sidewalk is frequently used by both pedestrians and bicyclists, an additional 4 ft should be added to the sidewalk width. See Section 17.3 for more discussion on bicyclists use of sidewalks.

3. CBD Areas. The entire area between the curb and building is often fully used as a paved sidewalk.

4. Appurtenances. The designer should also consider the impacts of roadside appurtenances within the sidewalk (e.g., fire hydrants, parking meters, utility poles). These elements will reduce the effective width because they interfere with pedestrian activity. Preferably, these appurtenances should be placed behind the sidewalk. If they are placed within the sidewalk, the sidewalk should have an effective width of 4 ft desirable and 3 ft minimum. The effective width will be measured from the edge of the appurtenance to the edge of the sidewalk. The 3-ft minimum is necessary to meet the handicapped accessibility requirements (see Section 17.4).

5. Cross Slope. The typical cross slope on the sidewalk is 2%. If the sidewalk is on an accessible route, then the maximum cross slope will be 2%. See Section 17.4.

6. Buffer Areas. If the available right-of-way is sufficient, buffer areas between the curb and sidewalk are desirable. These areas provide greater separation between vehicle and pedestrian. The buffer area

Table 8.1A

PEDESTRIAN AND VEHICLE VOLUMES FOR WHICH THE CONSTRUCTION OF SIDEWALKS MAY BE CONSIDERED

Vehicular Traffic (DHV)	Pedestrians per day suggested for construction of sidewalks when design speed, mph, is:	
	30 to 50	60 to 70
Sidewalk, one side:		
30 to 100	150	100
More than 100	100	50
*Sidewalk, both sides:**		
50 to 100	500	300
More than 100	300	200

* Smaller pedestrian traffic volume may justify two sidewalks to avoid a considerable amount of cross traffic.

Source: (3)

should be at least 2-ft wide to be effective and, if practical, should be 8 to 12 ft in width.

7. Pedestrian Rails (on Bridges). Chapter Eleven provides criteria for when a pedestrian rail will be required on a bridge to separate vehicular and pedestrian flows.

8.2 MEDIANS

8.2.1 Functions

A median is defined as the portion of a divided highway separating the traveled way for traffic in opposing directions. Medians are desirable on many multilane highways. The principal functions of a median are:

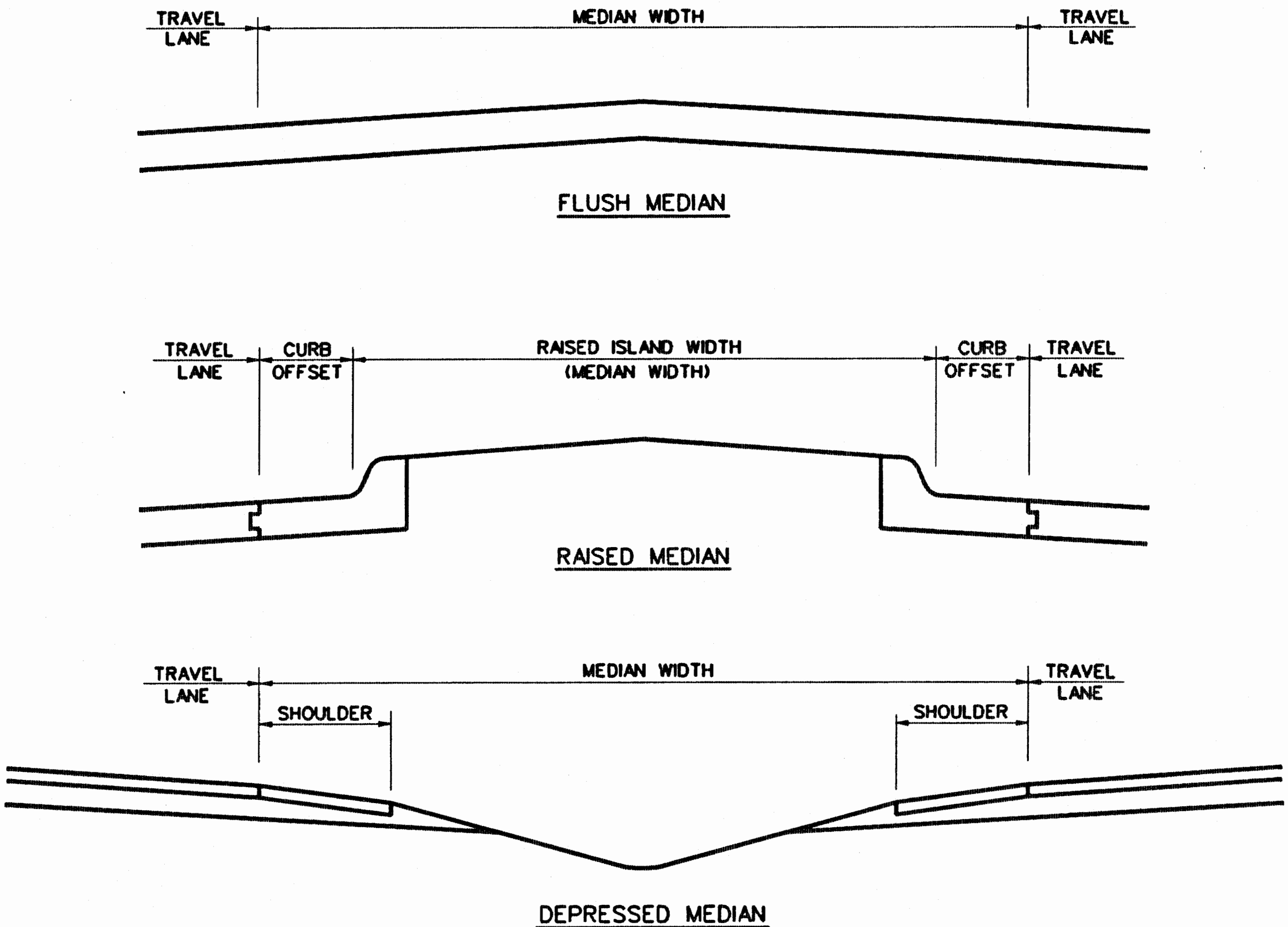
1. to provide separation from opposing traffic,
2. to prevent undesirable turning movements,
3. to provide an area for deceleration and storage of left-turning vehicles,
4. to provide an area for storage of vehicles crossing the mainline at intersections,
5. to facilitate drainage collection,
6. to provide an area for pedestrian refuge, and
7. to provide width for future lanes.

8.2.2 Median Widths

8.2.2.1 General

In general, the median should be as wide as can be used advantageously. The median width is measured from the edge of the two inside travel lanes. Figure 8.2A presents median schematics which define the median width for the three basic median types (see Section 8.2.3). The design width will depend on the functional class, type of median, availability of right-of-way, construction costs, maintenance, acceptable median slopes, the anticipated ultimate development of the facility, operations at crossing intersections and field conditions. Several factors will determine the appropriate median width:

1. Left Turns. The need for left-turn bays should be considered when selecting a median width.
2. Crossing Vehicles. A median preferably should be a minimum of 25-ft wide to safely allow a crossing passenger vehicle to stop between the two roadways.
3. Freeways. Median widths of 40 ft or more are considered widely separated. In general, median widths of 20 ft or more may not require glare screens.
4. Signalization. At signalized intersections, wide medians may lead to increased crossing times and less efficient traffic operations.
5. Median Barriers. With narrow medians, a median barrier may be warranted. See Section 11.6.
6. Intersections. Several vehicular maneuvers at intersections are partially dependent on the median width. These include U-turns and turning maneuvers at median openings. The designer should evaluate the likely maneuvers at intersections and provide a median width that will accommodate the selected design vehicle. See Chapter Nine.
7. Uniformity. In general, a uniform median width is desirable. However, variable-width medians may be advantageous where right-of-way is restricted, at-grade intersections are widely spaced (0.5 mile or more), or an independent alignment is practical.
8. Sight Distance. Where a median barrier is present at a horizontal curve, the median width may be a factor in whether



MEDIAN WIDTH DEFINITIONS

Figure 8.2A

or not adequate sight distance is available at the curve. See Section 6.5.

8.2.2.2 Criteria

Chapter Twelve presents specific numerical criteria for median widths on new construction/reconstruction projects on major highways. These are summarized in Table 8.2A. If glare screens, light poles or other appurtenances are placed on a median barrier, the desirable median width is 26 ft. This provides sufficient room for maintenance vehicles without blocking a travel lane. Section 8.2.3 presents criteria for median width based on the type of median (flush, raised, depressed).

8.2.3 Median Types

Section 8.8 provides typical sections for various median types.

8.2.3.1 Flush Medians

A median is defined as a flush median when its vertical elevation above the surface of the adjacent roadway pavement is 1 inch or less. Flush medians are often used on urban highways and streets. A flush median should have adequate cross slope to properly control drainage in the median area.

The typical width for a flush median ranges from 2 ft to 16 ft; at least 2 ft is required to provide any special treatments (e.g., cross hatching, rumble strips). The ODOT Standard Drawings present striping details for flush medians.

Two-way left-turn lanes (TWLTL) are also considered flush medians. Desirably, the roadway cross section with a flush median will allow ultimate development for a TWLTL in urban/suburban areas. The tables in Chapters Twelve and Thirteen provide criteria for TWLTL widths. Section 9.4 provides information on design details for TWLTL at intersections. Section 9.4 also discusses the ODOT rural transition section, which is a variation of the TWLTL.

8.2.3.2 Raised Medians

A median is defined as a raised median if it contains a raised portion (greater than 1 inch) within its limits. Raised medians are often used on urban and suburban highways and streets to control access and left turns. When compared to flush medians, raised medians offer several advantages:

1. Mid-block left turns are controlled.
2. Left-turn channelization can be more effectively delineated.
3. A distinct location is available for traffic signs, signals and pedestrian refuge.
4. Limited physical separation is available.

The disadvantages of raised medians when compared to flush medians are:

5. They are more expensive to construct and more difficult to maintain.
6. They may need greater widths to serve the same function (e.g., left-turn lanes at intersections) because of the raised island and offset between curb and travel lane.
7. Curbs may result in adverse vehicular behavior upon impact.
8. Prohibiting mid-block left turns may overload street intersections and may increase the number of U-turns.

Table 8.2A

MEDIAN WIDTHS

Concrete Median Barrier	No. of Lanes	Median Width
Yes	4	Desirable: 14′ Minimum: 10′
	6	Truck DDHV <250: 22′ Truck DDHV >250: 26′
No	All	Desirable: 64′ Minimum: 46′*

* *Existing median widths of 40′ may be retained. A 40′ median width may be used on new alignment when right-of-way is severely restricted.*

9. Access for emergency vehicles (e.g., fire, ambulance) may be more difficult.

If a raised median will be used, the designer should consider the following in the design of the median:

10. Curb Type. Mountable curbs are typically used for raised medians. These allow vehicles to exit the travel lanes where needed for emergency reasons. Barrier curbs should only be used where mountable curbs cannot properly control access.

11. Speed. Barrier curbs may be used where the design speed is 45 mph or less. Mountable curbs may be used at any design speed.

12. Curbs at Turn Bays. If mountable curbs are used where the design speed is 45 mph or less, either barrier or mountable curbs may be used adjacent to channelized left- or right-turn lanes.

13. Curb Type with Appurtenances. If the median width is less than 12 ft and appurtenances will be located in the median (e.g., traffic signal poles, light poles, signs, bridge piers), a 6-inch barrier curb may be warranted on low-speed facilities.

14. Desirable Width. If practical, the width of a raised median should be sufficient to allow for the development of a channelized left-turn lane. For example, an 16-ft median width would allow the following:

    a. a 12-ft turn lane,

    b. a zero curb offset between the turn lane and raised island, and

    c. a minimum 4-ft raised island.

15. Minimum Width. The minimum width of a raised island should be 4 ft. Curb offsets would be in addition to the raised

island width. See Chapters Twelve and Thirteen.

16. Raised Island (Paved). For raised islands up to 10 ft, the island will be paved according to the detail for the concrete dividing strip in the ASCD and CSCD sheets in the *ODOT Standard Drawings*. The 4-inch thickness is typically used; the 6-inch thickness is used on low-speed facilities at the noses of raised islands where vehicles frequently ride over the nose.

17. Raised Island (Sodded). For raised islands greater than 10 ft, the area between the curbs is usually backfilled and sodded. However, where there are numerous signs, guardrail, bridge piers, etc., in the island, it may be more economical to pave the raised island to eliminate excessive hand mowing.

8.2.3.3 Depressed Medians

A depressed median is typically used where practical on freeways and other divided arterials. Depressed medians have better drainage characteristics and, therefore, are preferred on major highways. Depressed medians should be as wide as practical to allow for the addition of future travel lanes on the inside while maintaining a sufficient future median width.

Because the water is flowing into a depressed median, the designer needs to consider drainage when determining the appropriate depth of depressed median. The median ditch should be designed so that the depth of flow during the design-year discharge (e.g., Q_{10}) will be at least 6 inches below the subgrade. The *ODOT Drainage Manual* and Chapter Fifteen provide additional details on drainage design.

Desirably, the center longitudinal slope of a depressed median should be at least 0.5%, with 0.4% as a minimum.

8.3 ROADSIDE ELEMENTS

8.3.1 Fill Slopes

Fill slopes are the slopes extending outward and downward from the edge of the shoulder to intersect the natural ground line. The slope criteria depend upon the functional classification, fill height, urban/rural location and the presence of curbs. ODOT's criteria on fill slopes are presented in Tables 8.3A (uncurbed) and 8.3B (curbed). Note that, for uncurbed facilities, several of the criteria are based on the clear zone (as measured from the edge of the travel lane) which applies to the facility. The application of these criteria are as follows: The indicated slope from the shoulder edge is used to the clear zone (if practical) or to the intersection with the subgrade, whichever is the greater distance. See the accompanying figure in Table 8.3A. If the slope from the shoulder edge does not intersect the natural ground line, then the slopes in Table 8.3A, based on the fill height, should be used to the toe of the slope. Desirably, the top and bottom of the slopes will be rounded.

Although Tables 8.3A and B provide specific criteria for fill slopes, consideration must be given to right-of-way restrictions, utility considerations and roadside development in determining the appropriate fill slope for the site conditions. If practical, flatter fill slopes than indicated should be used.

As indicated in Tables 8.3A and 8.3B, the maximum fill slope should in general be 3:1. A 3:1 slope is a practical maximum when considering maintenance operations (e.g., mowing), erosion control and roadside safety. Slopes steeper than 3:1 may be used with mitigating treatments (e.g., slope paving). A slope steeper than 3:1 will normally require a roadside barrier (see Chapter Eleven).

Depending upon the soils condition and the height of fill, it may be warranted to provide benching on an existing slope before adding additional fill material. Figure 8.3A illustrates the benching detail which is typically used.

8.3.2 Cut Slopes

8.3.2.1 Typical Slope Rates (Earth Cuts)

In earth cuts on facilities without curbs, roadside ditches are provided to control drainage. As indicated in Tables 8.3C and 8.3D, the ditch section includes the fore slopes, ditch width and back slope as appropriate for the facility type. On facilities with curbs, a shelf (6-ft typical) is provided with the back slope beyond the shelf. Applicable criteria are provided in Table 8.3D. The following sections provide additional information for earth cuts and rock cuts.

8.3.2.2 Rock Cuts (Back Slopes)

The back slope is variable depending on field conditions. See Figure 8.3B. At a maximum, the back slope should not exceed ¼:1. For large cuts, benching of the back slope may be required. The Materials Division will assist in the analysis and design of rock cut sections.

8.3.2.3 Erosion Control

The designer must ensure that permanent erosion control is considered in the design of ditches in cut slopes. The designer must review the existing soil conditions to determine if additional measures may be required to control erosion (e.g., additional topsoil, special plantings, paving). As a general guide, longitudinal ditch slopes of 2% or greater will usually require some type of lining. For more information on the design of ditch linings, the designer should review the *ODOT Drainage Manual* and HEC #15 *Design of Stable Channels with Flexible Linings*. The Roadside Development Branch, Rural Design will assist in the final determination on the erosion control measures for cut slopes.

8.3.2.4 Roadside Safety

To safely accommodate a run-off-the-road vehicle, the slopes of the ditches should be as flat as practical. Chapter Eleven presents specific criteria to determine desirable fore slope and back slope combinations.

If practical, utility poles, non-breakaway signs, etc., should be placed outside of the clear zone. See Chapter Eleven.

8.3.2.5 Hydraulic Design

In general, the depth of a roadside ditch should ensure that the flow line for the design discharge will be at least 6 inches below the subgrade intercept with the fore slope. The longitudinal slope of the ditch will desirably be at least 0.5%; the minimum slope should be 0.4%. The *ODOT Drainage Manual* further discusses the hydraulic design of roadside ditches.

8.3.3 Geotechnical Features

The designer must ensure that the topography and geology of the site is compatible with the proposed fill and cut slope sections. The project geotechnical report must be reviewed and data analyzed to ensure the stability of cut and fill slopes which are 3:1 or steeper.

In addition, major or unusual geotechnical features within a project require special consideration (see the following sections). These projects should be reviewed by the Materials Division and the FHWA. A meeting to discuss the geotechnical features may also be warranted.

8.3.3.1 Major Geotechnical Features

These include:

1. Earthwork. Soil or rock cuts or fills where (1) the maximum height of cut or fill exceeds 50 ft, or (2) the cuts or fills are located in topography and/or geologic units with known stability problems.

2. Soil and Rock Instability Corrections. Cut, fill or natural slopes which are presently or potentially unstable.

3. Retaining Walls. Where maximum height at any point along the length exceeds 30 ft. Geotechnical aspects include bearing capacity, settlement, overturning and sliding. The Bridge Division is responsible for retaining wall design and review.

8.3.3.2 Unusual Geotechnical Features

These include:

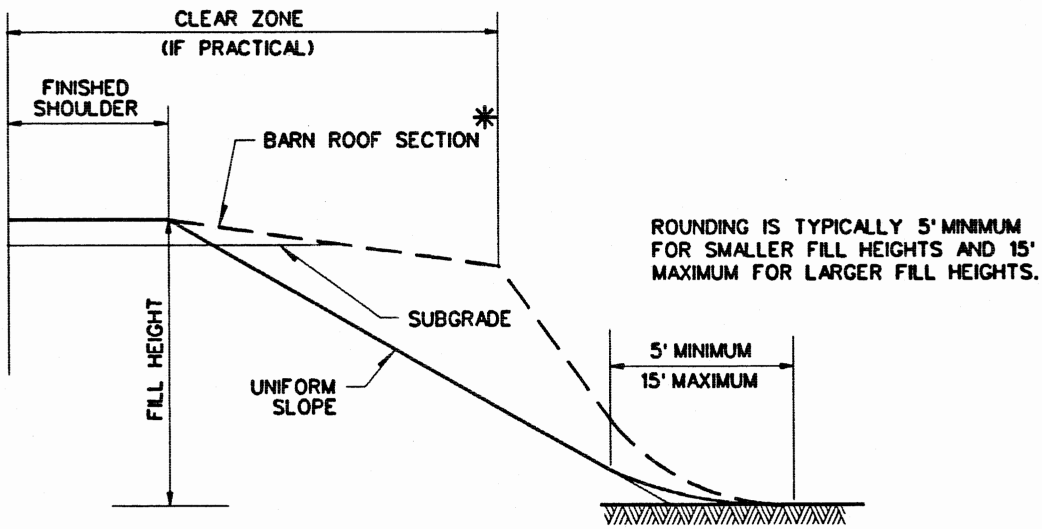
1. Difficult or Unusual Geotechnical Problems. Such conditions as embankment construction on a weak and compressible foundation material (difficult) or fills constructed using degradable shales (unusual).

2. New or Complex Designs. Geotextile soil reinforcement, permanent ground anchors, wick drains, stone columns, etc.

3. Unusual Design Methods. Experimental retaining wall systems or pile foundations where dense soils exist.

Table 8.3A

TYPICAL FILL SLOPES
(Uncurbed Facilities)



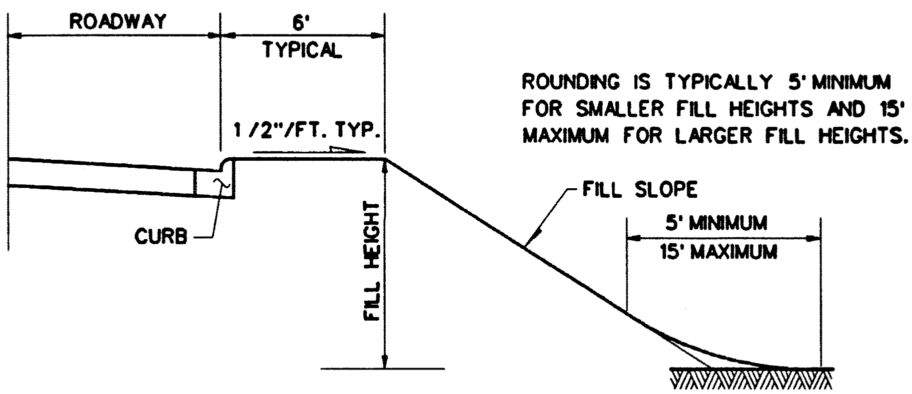
* The slope break on barn roof sections should extend to the clear zone.

FACILITY	FILL HEIGHT	FILL SLOPE
Freeways/Principal Arterials	0-4′ 4′-10′ >10′**	6:1 6:1 to clear zone; 4:1 to toe 6:1 to clear zone; 3:1 to toe
Other Arterials	0-4′ 4′-10′ >10′**	6:1 desirable; 4:1 maximum 4:1 4:1 to clear zone; 3:1 to toe
Collectors (State Highways)	0-4′ 4′-10′ >10′**	4:1 4:1 desirable; 3:1 maximum 3:1
Non-State Highways (Collectors/Local)	0-4′ 4′-10′ >10′**	4:1 desirable; 3:1 maximum 3:1 3:1

** Check Geotechnical Report for steeper slopes, if needed.

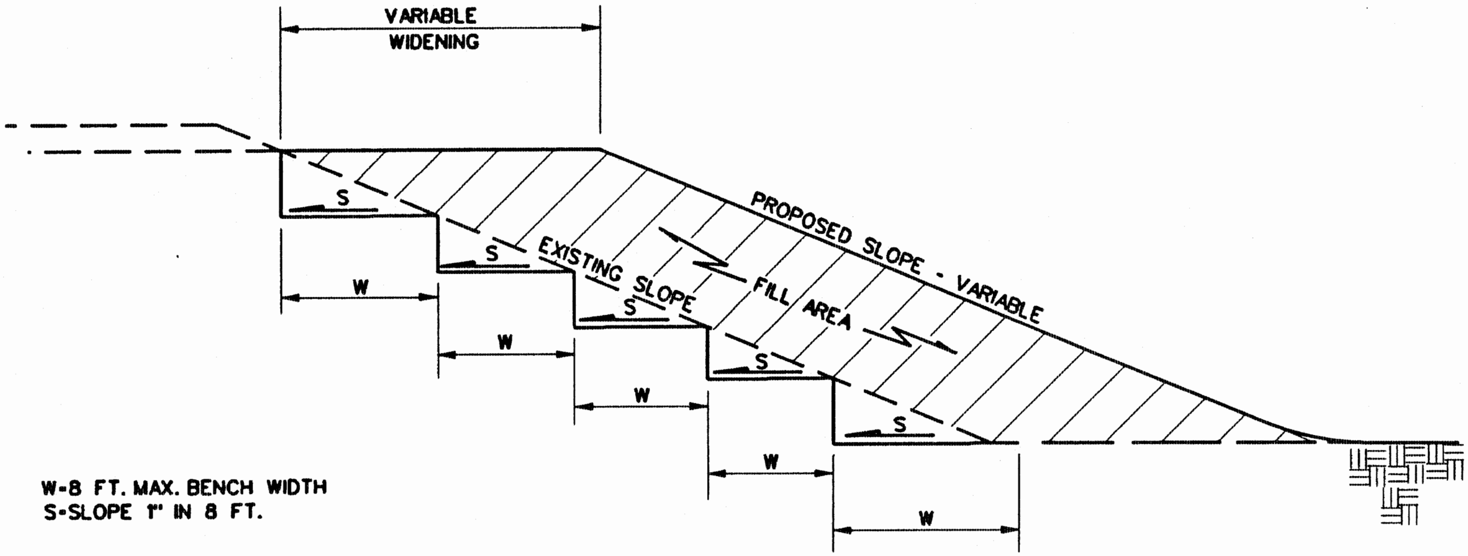
Table 8.3B

TYPICAL FILL SLOPES
(Curbed Facilities)



FACILITY	FILL HEIGHT	FILL SLOPE
Freeways	0-4′ 4′-10′ >10′*	6:1 6:1 to clear zone; 4:1 to toe 6:1 to clear zone; 3:1 to toe
Other Facilities	All*	6:1 desirable; 3:1 maximum

* Check Geotechnical Report for steeper slopes, if needed.



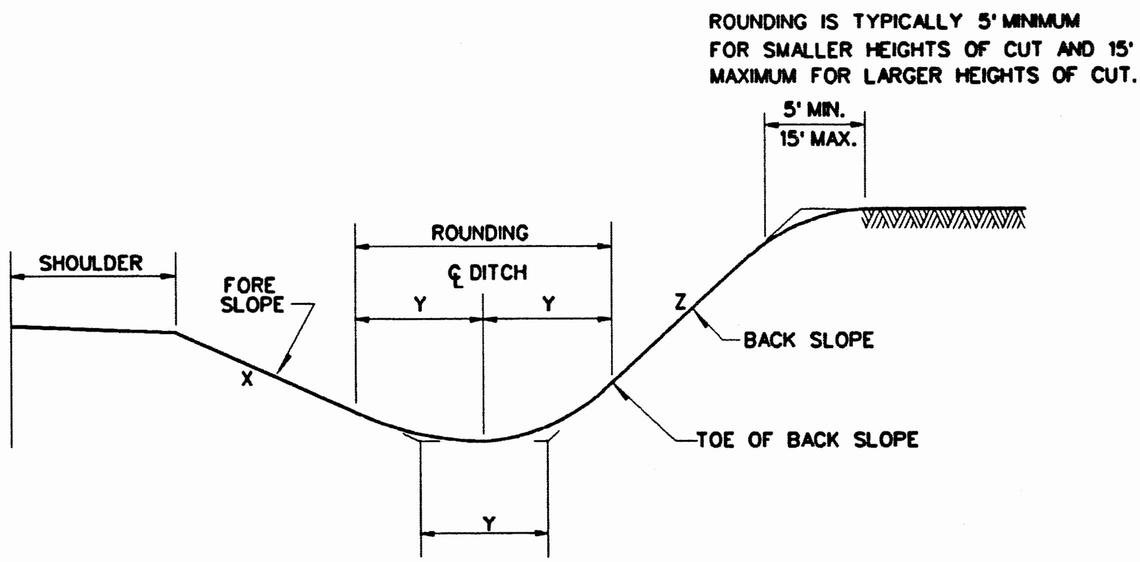
Note: Where required (in Fill Areas) the existing slopes shall be continuously benched, beginning at the lower limits of the slope. Benching is typically measured and paid for as Unclassified Excavation.

BENCHING DETAIL

Figure 8.3A

Table 8.3C

TYPICAL EARTH CUT SLOPES
(Uncurbed Facilities)

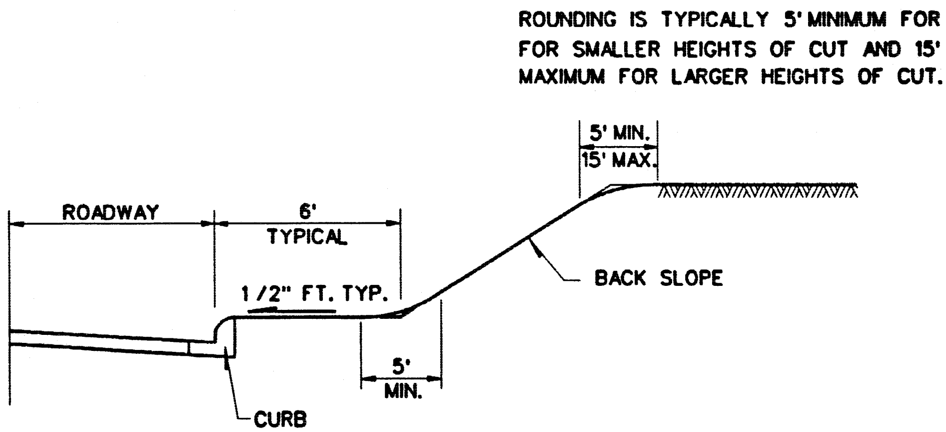


Rural Facilities	x	y	z
Freeways/Principal Arterials	6:1	8′	4:1
Other Arterials	6:1, V>50 4:1, V≤50	8′	4:1, V>50 3:1, V≤50
Collectors (State Highways)	6:1 desirable 4:1 maximum	8′ desirable 4′ minimum	3:1
Collectors (Non-State Highways)	4:1 desirable 3:1 maximum	4′ desirable 2′ minimum	3:1 desirable 2:1 maximum
Locals	3:1	As required for drainage	3:1 desirable 2:1 maximum

Notes:
1. See Figure 8.3B for dimensions in rock cuts.
2. See Chapter Twelve for criteria on suburban and urban facilities.
3. See Chapter Eleven for information on traversability of roadside ditches.
4. Refer to *ODOT Drainage Manual* for hydraulic design of roadside ditches.
5. Check Geotechnical Report to determine stability on all slopes 3:1 or steeper.

Table 8.3D

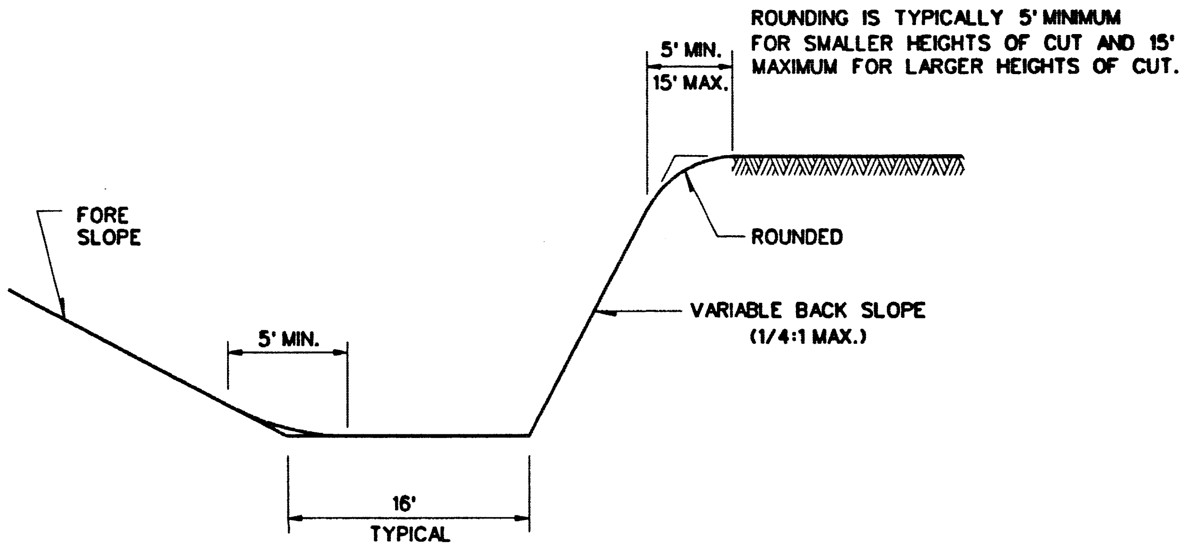
TYPICAL EARTH CUT SLOPES
(Curbed Facilities)



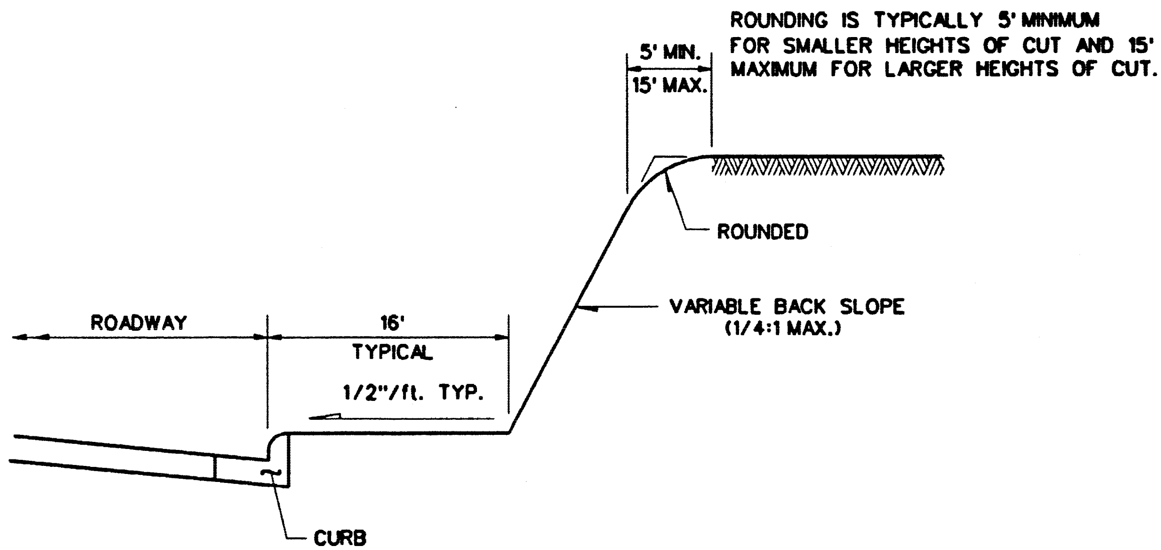
Facility	Back Slope
Freeways	6:1
Arterials	3:1
Collectors/Locals	2:1

Notes: 1. See Figure 8.3B for dimensions in rock cuts.

2. Check Geotechnical Report to determine stability for all slopes 3:1 and steeper.



UNCURBED FACILITIES



CURBED FACILITIES

Notes: 1. Back slope in rock cut may or may not require benching.

2. Check Geotechnical Report for difficult and/or unusual conditions.

TYPICAL ROCK CUT SLOPES

Figure 8.3B

8.4 BRIDGE AND UNDERPASS CROSS SECTIONS

The highway cross section must be carried over and under bridges, which often requires special considerations because of the confining nature of bridges and their high unit costs. The bridge or underpass section will depend upon the cross section of the approaching roadway, the highway functional classification and the project scope of work.

8.4.1 Bridges

8.4.1.1 New and Reconstructed Bridges

For freeways and arterials, the full approach roadway width, including shoulders, will be carried across the structure. The approach width will be determined by the lane and shoulder width criteria in Chapter Twelve.

For collectors and local roads and streets, the tables in Chapter Twelve provide the minimum roadway widths on bridges for these classifications. The bridge width will be based on traffic volumes.

8.4.1.2 Bridges To Remain In Place

An "existing bridge to remain in place" refers to any bridge work which does not require the total replacement of both the substructure and superstructure.

If an existing bridge within the limits of a project is structurally sound, if it meets ODOT's design loading structural capacity, and if the location has not had a significant accident history, it is unlikely to be cost effective to improve the geometrics of the bridge. The tables in Chapters Twelve and Thirteen provide the minimum widths for bridges to remain in place. If an existing bridge width is less than these criteria, it may be appropriate to widen the bridge as part of the project.

8.4.2 Underpasses

The approach roadway cross section, including clear zones, should be carried through the underpass. Clear zones are functions of design speed, traffic volumes, highway alignment and side slopes (see Chapter Eleven). If an auxiliary lane passes through the underpass adjacent to the mainline, the clear zone distance should be measured from the edge of the mainline or the auxiliary lane, whichever yields the greater distance (see Section 11.2). The lateral clearance for any collector-distributor roads should be treated separately from the mainline, with its clear zone based on its own design speed, side slope and traffic characteristics.

When determining the cross section width of a highway underpass, the designer should also consider the likelihood of future roadway widening. Widening an existing underpass in the future can be extremely expensive and it may be warranted, if some flexibility is available, to allow for possible future roadway expansion. Therefore, the designer should evaluate the potential for further development in the vicinity of the underpass which would significantly increase traffic volumes. If appropriate, a reasonable allowance for future widening may be to provide sufficient lateral clearance for one additional lane in each direction. As discussed in Section 5.3, an estimate for a 50-year expansion may be appropriate.

This consideration for future expansion greatly increases the value of providing the roadside clear zone through the underpass. If widening becomes necessary, space is available without a major disruption to the existing structure.

8.4.3 Travelway Width Reductions

When approaching a narrow bridge or underpass, the travelway width may need to be reduced to allow the roadway to pass over or under a bridge. These travelway reduction transitions should be designed using the following formulas:

1. For highways with design speeds greater than 45 mph, $L = WS$.

2. For highways with design speeds of 45 mph or less, $L = WS^2/60$.

For both formulas, L equals the taper length in ft, W the offset distance in ft and S the design speed in mph.

Where the travelway width will be reduced, the designer should consider the need for special traffic engineering treatments (e.g., delineation). See Chapter Fourteen.

8.5 TRANSITIONS

Careful consideration must be given to the design of transitions from multilane facilities to two-lane facilities. These are complex decision-making areas for a driver, who may not be expecting the lane reduction. Therefore, the designer should use the safest criteria practical, whether these connections are permanent or temporary.

The horizontal alignment for permanent and temporary transitions should follow the criteria presented in Chapter Six. All temporary connections should be designed as new facilities. This includes, but is not limited to, superelevation, transition lengths, reverse curves and the tangent length between curves.

Decision sight distance should be provided to and throughout the transition area. To achieve this objective, the project termini may need to be adjusted.

The following figures illustrate ODOT criteria for various transitions:

1. Figure 8.5A provides the details for a left transition from a 4-lane to a 2-lane facility on a curve.

2. Figure 8.5B provides the details for a right transition from a 2-lane to a 4-lane facility on a curve.

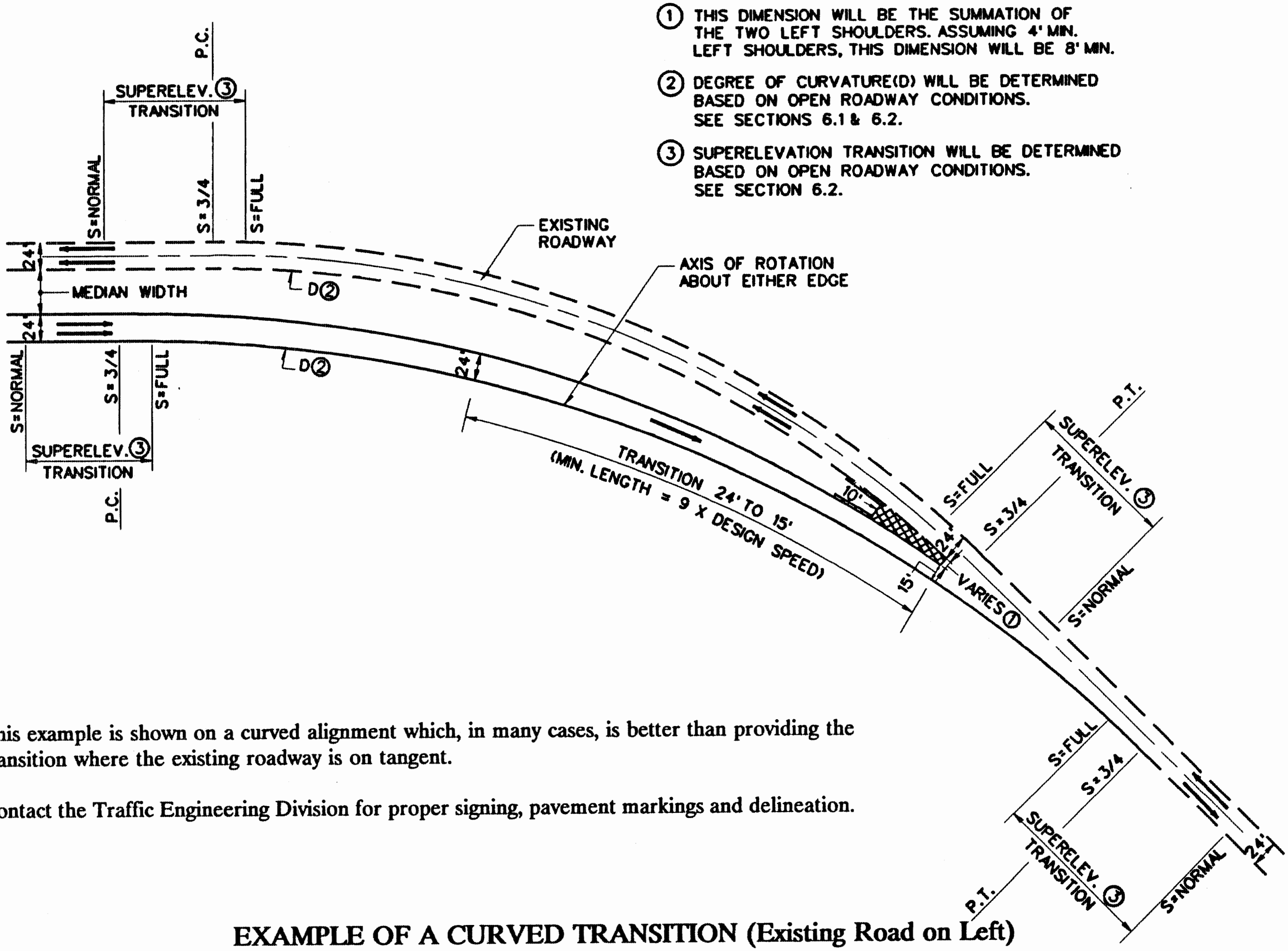
3. Figure 8.5C provides the details for a left transition on tangent.

4. Figure 8.5D provides the details for a right transition on tangent.

5. Figure 8.5E provides the details for a split transition from a 4-lane to a 2-lane facility on a tangent section.

6. Figure 8.5F provides the details for a split transition from a 4-lane undivided to a 4-lane divided facility on a tangent section.

7. Figure 8.5G provides the details for a split transition from a 4-lane undivided to a 5-lane TWLTL section on a tangent.



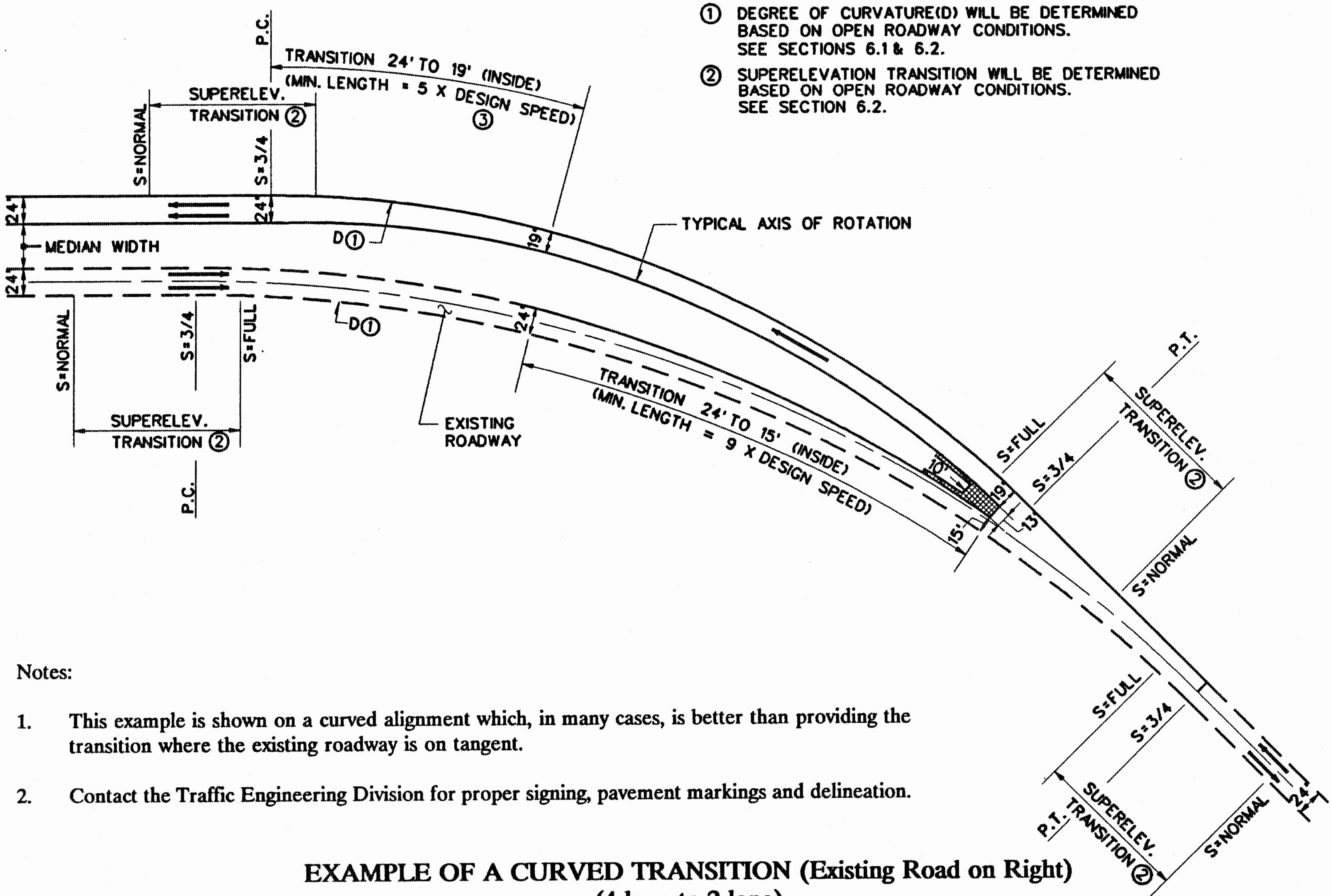
Notes:

1. This example is shown on a curved alignment which, in many cases, is better than providing the transition where the existing roadway is on tangent.

2. Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.

EXAMPLE OF A CURVED TRANSITION (Existing Road on Left)
(4-lane to 2-lane)

Figure 8.5A



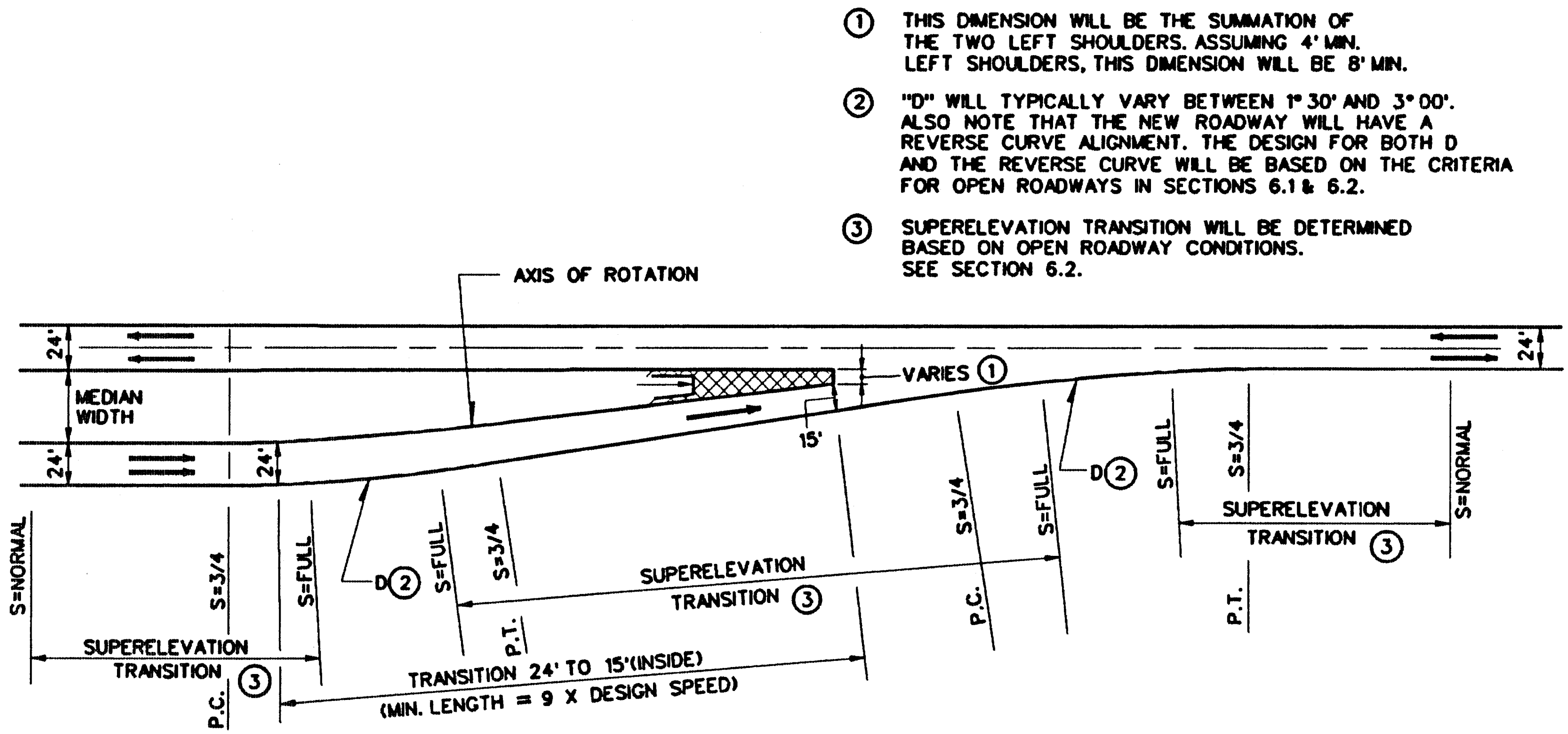
Notes:

1. This example is shown on a curved alignment which, in many cases, is better than providing the transition where the existing roadway is on tangent.

2. Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.

EXAMPLE OF A CURVED TRANSITION (Existing Road on Right)
(4-lane to 2-lane)

Figure 8.5B



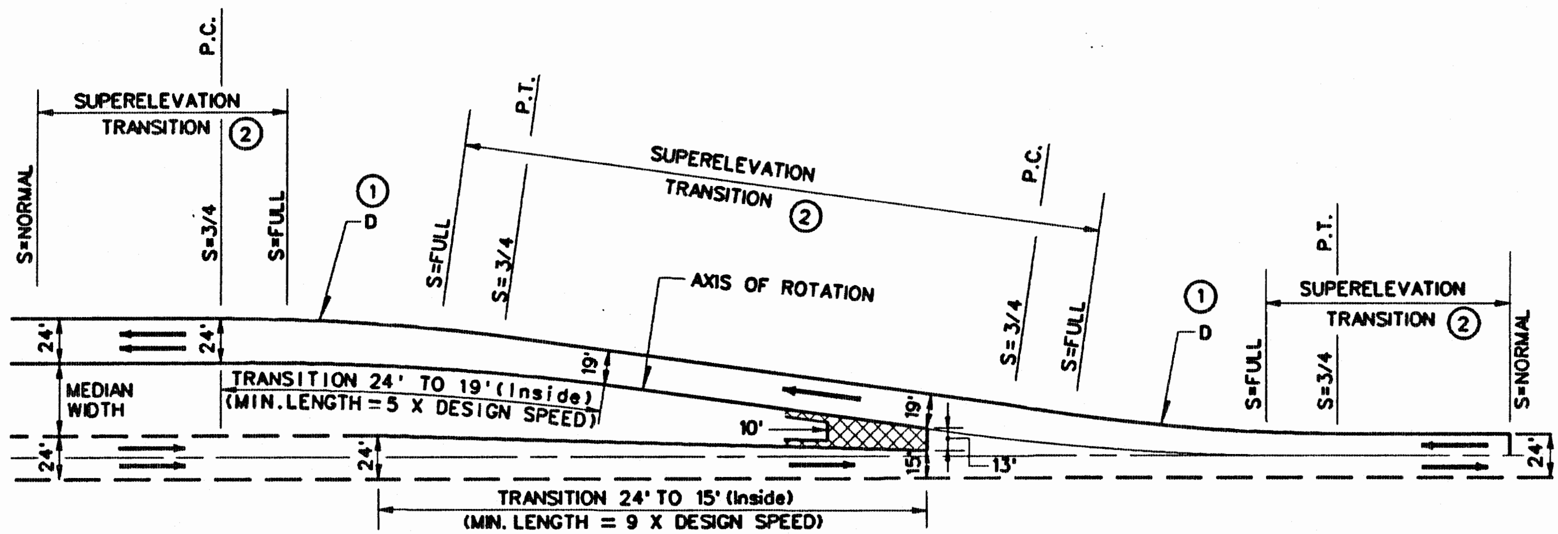
Note: Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.

EXAMPLE OF A TANGENT TRANSITION (Existing Road on Left)
(4-Lane to 2-Lane)

Figure 8.5C

① "D" WILL TYPICALLY VARY BETWEEN 1° 30' AND 3° 00'. ALSO NOTE THAT THE NEW ROADWAY WILL HAVE A REVERSE CURVE ALIGNMENT. THE DESIGN FOR BOTH D AND THE REVERSE CURVE WILL BE BASED ON THE CRITERIA FOR OPEN ROADWAYS IN SECTIONS 6.1 & 6.2.

② SUPERELEVATION TRANSITION WILL BE DETERMINED BASED ON OPEN ROADWAY CONDITIONS. SEE SECTION 6.2.

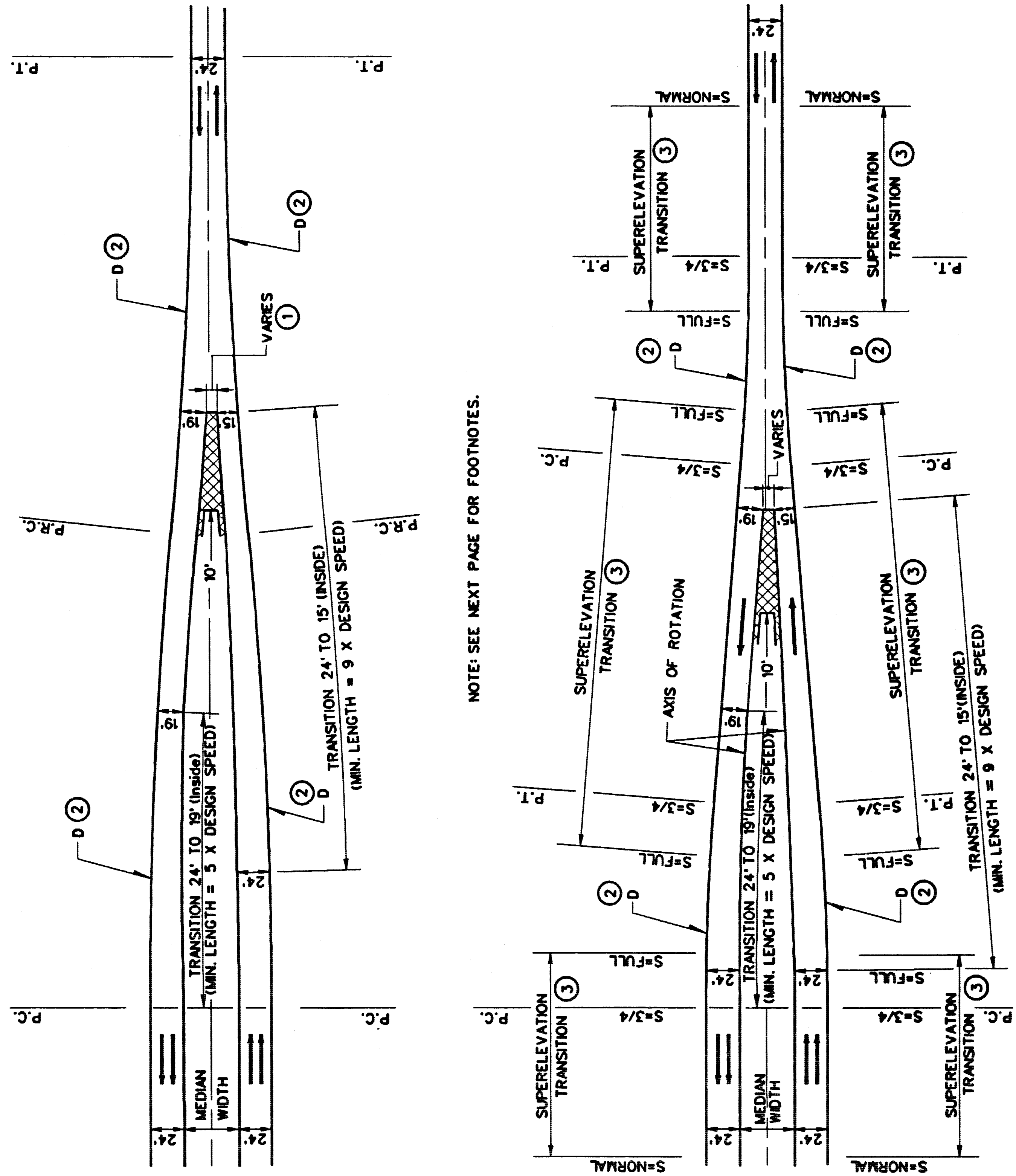


Notes: 1. The right transition on a tangent is less desirable than the left transition on a tangent.

2. Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.

EXAMPLE OF A TANGENT TRANSITION (Existing Road on Right)
(4-Lane to 2-Lane)

Figure 8.5D



SPLIT TRANSITION
(2-lane to 4-lane)

Figure 8.5E

Footnotes for Figure 8.5E

① THIS DIMENSION WILL BE THE SUMMATION OF THE TWO LEFT SHOULDERS. ASSUMING 4' MIN. LEFT SHOULDERS, THIS DIMENSION WILL BE 8' MIN.

② DEGREE OF CURVATURE(D) WILL BE DETERMINED BASED ON OPEN ROADWAY CONDITIONS. SEE SECTIONS 6.1 & 6.2.

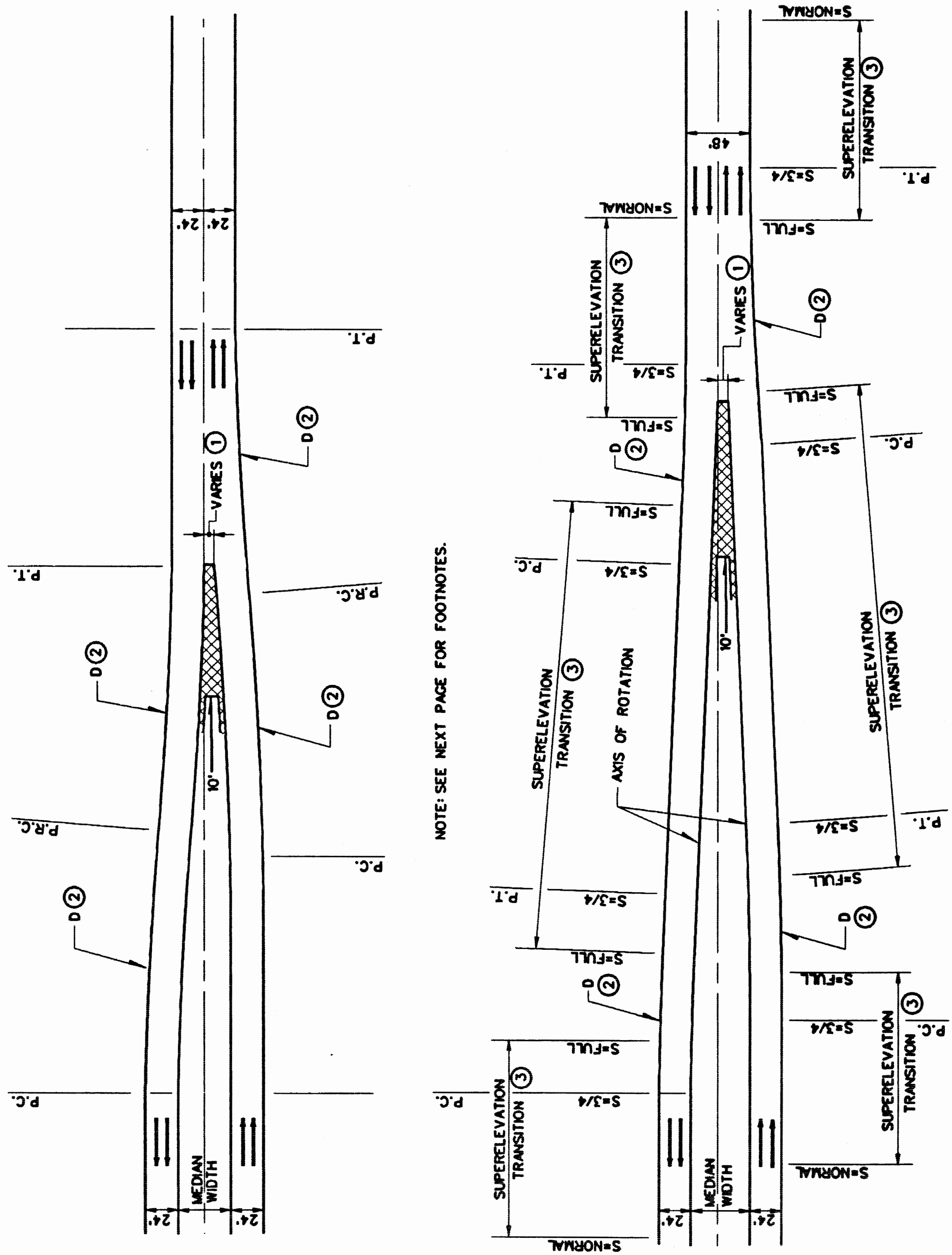
③ SUPERELEVATION TRANSITION WILL BE DETERMINED BASED ON OPEN ROADWAY CONDITIONS. SEE SECTION 6.2.

Notes:

A. The top figure assumes the normal crown section through the transition area. In this case, "D" will be limited to the maximum degree of curvature without superelevation based on open roadway conditions. See Section 6.2 for specific criteria based on design speed and e_{max}.

B. The bottom figure assumes that the two transition curves are superelevated. "D" and the superelevation transition will be based on open roadway conditions.

C. Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.



SPLIT TRANSITION
(4-Lane Undivided to 4-Lane Divided)

Figure 8.5F

Footnotes for Figure 8.5F

① THIS DIMENSION WILL BE THE SUMMATION OF THE TWO LEFT SHOULDERS. ASSUMING 4' MIN. LEFT SHOULDERS, THIS DIMENSION WILL BE 8' MIN.

② DEGREE OF CURVATURE(D) WILL BE DETERMINED BASED ON OPEN ROADWAY CONDITIONS. SEE SECTIONS 6.1 & 6.2.

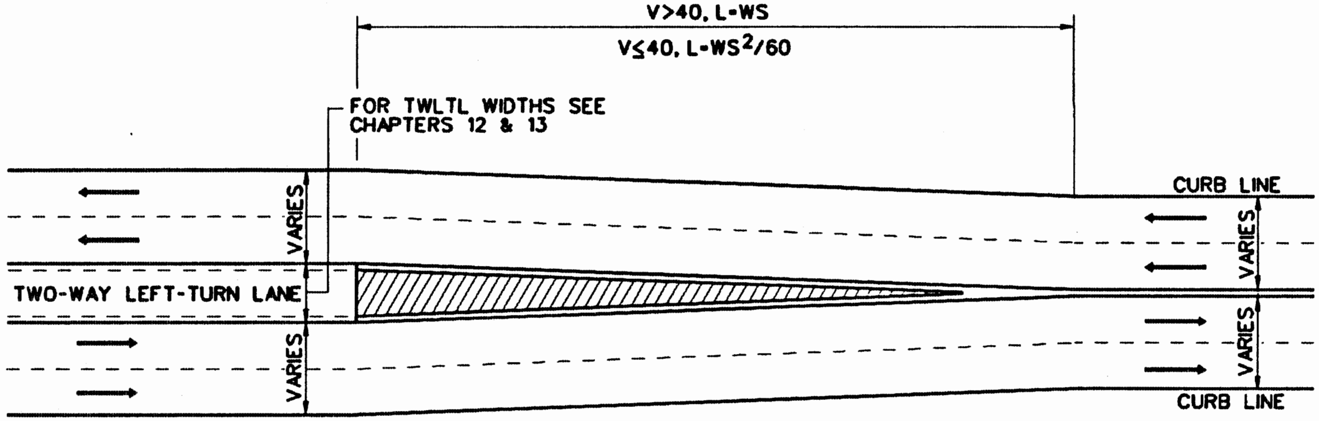
③ SUPERELEVATION TRANSITION WILL BE DETERMINED BASED ON OPEN ROADWAY CONDITIONS. SEE SECTION 6.2.

Notes:

A. The top figure assumes the normal crown section through the transition area. In this case, "D" will be limited to the maximum degree of curvature without superelevation based on open roadway conditions. See Section 6.2 for specific criteria based on design speed and e_{max}.

B. The bottom figure assumes that the two transition curves are superelevated. "D" and the superelevation transition will be based on open roadway conditions.

C. Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.



Note: Contact the Traffic Engineering Division for proper signing, pavement markings and delineation.

TWO-WAY LEFT-TURN LANE TRANSITION

Figure 8.5G

8.6 RIGHT-OF-WAY/UTILITIES

8.6.1 Right-of-Way

8.6.1.1 Definitions

The following definitions will apply:

1. Permanent ROW. ROW acquired for permanent ownership by the State for activities which are the responsibility of the State for an indefinite period of time. The State obtains the title to the property. Permanent ROW is typically acquired for utility accommodation, fill and cut slopes, etc.

2. Temporary ROW. ROW acquired for the legal right of usage by the State to serve a specific purpose for a limited period of time (e.g., maintenance and protection of traffic during construction). Once the activity is completed, the State yields its legal right of usage and returns the land to its original condition as close as practical.

3. ROW Easements. Rights-of-way acquired with the perpetual right to construct and maintain a public highway and incidental facilities over and across the surface of lands. Types of right-of-way easements include:

   a. highway easements,
   b. utility easements, and
   c. storm sewer easements.

4. Channel ROW. ROW acquired specifically for channel construction and maintenance, which provides the State with a permanent right of ingress and egress. The property owner relinquishes the right to modify the channel dimensions (e.g., slopes).

8.6.1.2 Width

The minimum right-of-way (ROW) width for all functional classifications will be the sum of the travel lanes, outside shoulders, and median width (if applicable) plus the necessary width for fill and cut slopes or for roadside clear zones, whichever is greater. Desirably, the ROW width will accommodate the anticipated ultimate development of the highway and utility facility.

The ROW width should be uniform, where practical. In urban areas, variable widths may be necessary due to existing development; varying side slopes and embankment heights may make it desirable to vary the ROW width; and, ROW limits will likely have to be adjusted at intersections and freeway interchanges. Other special ROW controls should also be considered:

1. At horizontal curves and intersections, additional ROW may be warranted to ensure that the necessary sight distances are available in the future.

2. In areas where the necessary ROW widths cannot be reasonably obtained, the designer may consider the advisability of using steeper slopes, revising grades, or using retaining structures.

3. Special ROW considerations at interchanges are discussed in Chapter Ten.

The designer will coordinate with the Right-of-Way Division on the acquisition of ROW for highway projects.

8.6.1.3 Fencing

In general, the Design Division is responsible for determining the need for fencing along State highways. The following summarizes ODOT fencing policies:

1. Where disturbed by the highway construction, fencing is replaced in-kind.

2. Fencing is generally warranted along access-controlled facilities.

3. Fencing types used by ODOT are woven wire, strand and chain link. The *ODOT Standard Drawings* present the design details for each type.

The designer should refer to the *ODOT Policy Manual*, the plan-in-hand report and survey data for more information.

8.6.2 Utilities

8.6.2.1 Types

In general, all utilities (both public and private) may be classified as pipelines, electric power lines or communication lines:

1. Pipelines. Pipelines may be further divided according to the products they transmit:

   a. gas lines (either high-pressure or medium-pressure transmission lines, distribution lines, vacuum lines or gathering lines);

   b. oil transmission and products lines;

   c. oil lines (gathering lines, waste lines or salt water lines);

   d. water lines;

   e. sanitary sewer lines; and

   f. miscellaneous pipelines.

2. Electric Power Lines. Electric power lines may be transmission lines, distribution lines or service lines. These lines may be aerial, underground or a combination of both.

3. Communication Lines and/or Facilities. Communications lines and/or facilities may be aerial, underground, a combination of both, or electronic transmission towers. Their function may encompass local or regional service.

8.6.2.2 Location

The road designer, in general, is responsible for initially determining utility locations. The following general rules of good and poor practices for utility location should be considered:

1. Utility lines should be located to minimize the need for later adjustment to accommodate future highway improvements and to permit servicing these lines with minimum interference to highway traffic.

2. Longitudinal installations should be located on uniform alignment as near as practical to the right-of-way line to provide a safe environment for traffic operation and preserve space for future highway improvements or other utility installations.

3. To the extent feasible, utilities crossing the highway should cross on a line generally perpendicular to the highway alignment.

4. The horizontal and vertical location of utility lines within the highway right-of-way limits should conform with ODOT clear zone criteria. The location of above-ground utility facilities should be consistent with the clearances applicable to other roadside obstacles.

5. Conditions which are generally unsuitable or undesirable for underground utility line crossings should be avoided. These include, but are not limited to, locations:

a. in deep cuts;

b. near footings of bridges and retaining walls;

c. at cross drains where flow of water, drift or stream bedload may be obstructed;

d. within basins of an underpass drained by a pump if the pipeline transports a liquid or liquefied gas; and

e. in wet or rocky terrain where it will be difficult to attain minimum cover.

6. On longitudinal installations, utility locations parallel to the roadway at or adjacent to the right-of-way line are preferable to minimize interference with the safe operation of the highway, the highway drainage, and the structural integrity of the traveled way, shoulder and embankment.

7. Vertical and horizontal clearances between a pipeline and a structure or other highway or utility facilities should be sufficient to permit maintenance of the pipeline and other facilities.

8.6.2.3 Design Considerations

The overall objective of ODOT is to ensure completion of the required adjustment of all utility facilities in a mutually satisfactory manner in advance of the roadway construction. The Utilities Branch within the Right-of-Way Division is responsible for working with the utility companies to develop the design details for utility relocations or adjustments. The Branch has developed the *ODOT Utilities Manual* to document ODOT policies and procedures. The road designer should ensure that the proposed road design is consistent with the practicalities of utility work. Therefore, the designer should consider the following:

1. General. Title 69, Oklahoma Statutes, provides the State authority for utility adjustments. ODOT policies incorporate the provisions of the FHWA Federal Highway Program Manual FHPM 6-6-3-1 and 6-6-3-2 for Federal-aid projects, where consistent with Oklahoma law. The AASHTO policies on utility relocations are incorporated into ODOT procedures. These include the following:

   a. AASHTO *A Policy on the Accommodation of Utilities within Freeway Right-of-Way.*

   b. AASHTO *A Guide for Accommodating Utilities within Highway Right-of-Way.*

2. Clearance and Cover. Pipelines should have a 4-ft minimum clearance from other utilities, except sanitary sewer lines should be at least 10 ft from water lines. Most pipelines should be at least 3 ft underground or 2.5 ft (when encased in a conduit) under a ditch flowline. The *ODOT Utilities Manual* provides further guidance.

3. Aerial. Electric power lines and/or communication lines that are aerially located should be at least 5 ft from the ROW line to allow for airspace, guywires and luminaries, if required.

4. Detours. Awareness of potential utility conflicts is extremely important when constructing detours near the ROW line or on temporary ROW.

8.7 FRONTAGE ROADS

8.7.1 ODOT Statutory Definitions

1. Frontage Road. A road constructed adjacent and parallel to but separated from the highway and connected thereto at least at each end, for service to abutting property and for control of access.

2. Local Road. A road constructed to provide access to property abutting on or adjacent to the highway and which has but one connection to the highway.

3. Public Road. A road constructed to connect other public roads or streets, but not connected to the highway.

8.7.2 General (AASHTO)

Frontage roads serve numerous functions, depending on the type of facility served and the character of the surrounding area. They may be used to control access to the facility, to function as a street serving abutting properties, and to maintain circulation of traffic on each side of the main highway. Frontage roads segregate local traffic from the higher-speed through traffic and serve driveways of residences and commercial establishments along the highway. Connections between the main highway and frontage roads, usually provided at crossroads, furnish access between through roads and adjacent property. Thus, the through character of the highway is preserved and is less affected by subsequent development along the roadsides.

Frontage roads are used on all types of highways. Their greatest use is adjacent to freeways where their primary function is to distribute and collect traffic between local streets and the freeway interchanges. In some circumstances, frontage roads are desirable on arterial streets both in urban and suburban areas.

Despite their advantages, the use of continuous frontage roads on relatively high-speed arterial streets with intersections at grade may be undesirable. At the cross streets, the various through and turning movements at several closely spaced intersections greatly increase the accident potential. The multiple intersections are also vulnerable to wrong-way entrances. Traffic operations are improved if the frontage roads are located a considerable distance from the main highway at the intersecting crossroads in order to lengthen the spacing between successive intersections along the crossroads. See Section 8.7.4.

Frontage roads may be one-way or two-way and generally are parallel to the roadway for through traffic. They may or may not be continuous, and they may be provided on one or both sides of the arterial. In general, one-way frontage roads must be continuous. On two-way frontage roads, it is desirable that they be continuous; intermittent frontage roads have much less value. Continuous frontage roads significantly improve traffic operations.

8.7.3 Functional Classification

The normal design elements of roadway width, cross slope, horizontal and vertical alignment, etc., should be provided consistent with the functional operation of the frontage road. That is, the same considerations relative to functional classification, design speed, traffic volumes, etc., apply to frontage roads as they would to any other highway.

For high-volume, continuous frontage roads, the desirable functional classification is one level below that of the main highway classification. At a minimum, the functional classification is a collector.

For low-volume, non-continuous frontage roads, the desirable functional classification is a collector. At a minimum, the functional classification is a local road.

8.7.4 <u>Design</u>

8.7.4.1 Design Elements

The selection of the appropriate design criteria is based on the functional classification of the frontage road. Once the functional classification has been determined, the appropriate design speed, lane and shoulder widths, etc., from the tables in Chapters Twelve or Thirteen can be selected.

8.7.4.2 One-Way/Two-Way

From an operational and safety perspective, one-way continuous frontage roads are preferred to two-way. One-way operations may inconvenience local traffic to some extent, but the advantages in reducing vehicular and pedestrian conflicts at intersecting streets often fully compensate for this inconvenience. In addition, there is some savings in pavement and right-of-way width. Two-way frontage roads at high-volume, at-grade intersections complicate crossing and turning movements. Where off ramps join a two-way frontage road, the potential for wrong-way entry is increased. This problem is greatest where the ramp joins the frontage road at an acute angle, thus giving the appearance of an on-ramp to the driver.

Two-way frontage roads may be considered for partially developed urban areas where the adjoining street system is so irregular or so disconnected that one-way operation would introduce considerable added travel distance and cause undue inconvenience. Two-way frontage roads may also be appropriate for suburban or rural areas where points of access to the through facility from the frontage road are widely spaced.

8.7.4.3 Outer Separation

The area between the main highway and a frontage road or street is the outer separation. The separation functions as a buffer between the through traffic on the main highway and the local traffic on the frontage road. This separation also provides space for shoulders and ramp connections to or from the through facility.

The wider the outer separation, the less influence local traffic will have on through traffic. Wider separations lend themselves to landscape treatments and enhance the appearance of both the highway and the adjoining property. Desirably, the outer separation between the through arterial and the frontage road will be 150 ft in rural areas and 80 ft in urban areas. This distance is measured between the edges of the through lanes for the main highway and frontage road. The minimum width of outer separation will be that required for the shoulder adjacent to the main highway, frontage road shoulder and for a median barrier or fence.

Desirably, the outer separation will be wide enough to eliminate the need for a barrier; i.e., all obstacles will be outside of the mainline clear zone and frontage road clear zone. In some cases, fencing may be warranted to separate the mainline and frontage road. Where used and where the fence will be within the clear zone of either the mainline or frontage road, chain link fencing must be used (see the *ODOT Standard Drawings*). Where a median barrier <u>and</u> fencing will be used, the fencing is typically placed on top of the median barrier.

A substantial width is particularly advantageous at intersections with cross streets. A wide outer separation minimizes vehicular and pedestrian conflicts. At intersections, the outer separation should be 300 ft in both rural and urban areas, if practical. The 300-ft dimension is measured from the outside edges of the two travel lanes which border on either side of the outer separation.

8.7.4.4 Access

Connections between the main highway and the frontage road are an important design element. On arterials with slow-moving traffic and one-way frontage roads, slip ramps or simple openings in a narrow outer separation may work reasonably well. Slip ramps from one-way frontage roads and freeways are acceptable. However, newly constructed slip ramps from a freeway to two-way frontage roads are unacceptable because they tend to induce wrong-way entry onto the freeway and may cause accidents at the intersection of the ramp and frontage road. Therefore, on freeways and other arterials with high operating speeds and two-way frontage roads, the access to the freeway should be provided at interchanges or by button-hook ramps. Details for ramp/ frontage road designs are provided in Chapter Ten.

8.8 TYPICAL SECTIONS

The following figures present typical sections which will apply to all new construction and reconstruction projects. Chapter Thirteen discusses how these criteria may be modified for projects on existing highways. The typical section figures are:

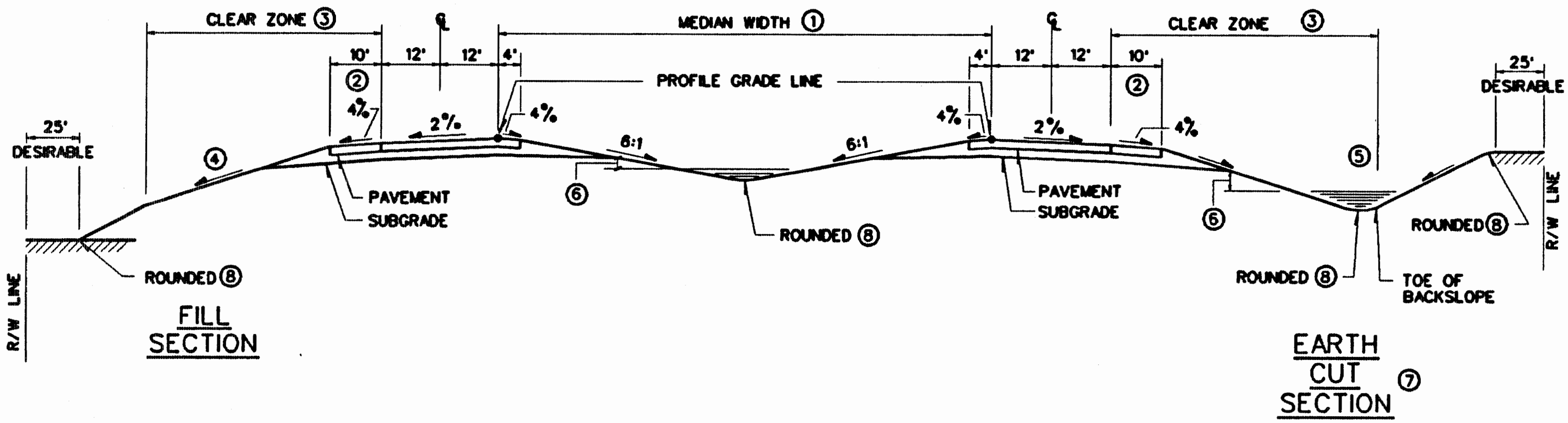
1. Figure 8.8A "Typical Depressed Median Section (Freeways)."

2. Figure 8.8B "Typical Two-Lane Rural Arterial."

3. Figure 8.8C "Typical Raised Median Section (Suburban/Urban Arterial)."

4. Figure 8.8D "Typical Two-Lane Urban Street (Curbed)."

5. Figure 8.8E "Miscellaneous Details."



① The median width will depend upon many factors. These include the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility and field conditions. See Section 8.2 for more discussion.

② See Chapter Twelve for criteria on shoulder widths.

③ The clear zone will vary according to design speed, traffic volumes and side slopes. See Section 11.2.

④ The fill slope criteria will depend upon the height of embankment and clear zone criteria. See Section 8.3 for specific criteria.

⑤ The criteria for fore slope, ditch width and back slope will vary. See Section 8.3 for specific criteria.

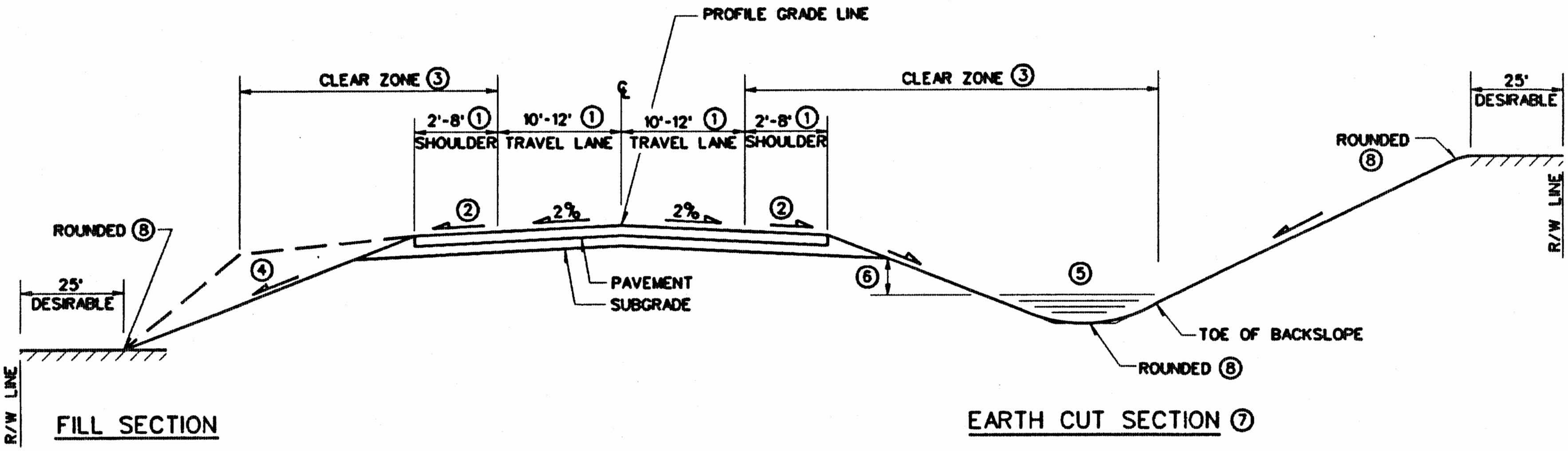
⑥ The depth of ditch should ensure that the flow line for the design discharge is at least 6" below the subgrade interception with the slope. See the *ODOT Drainage Manual* for more information.

⑦ Figure illustrates ditch in earth cut. See Section 8.3 for information on rock cuts.

⑧ See Section 8.3 for details on rounding.

TYPICAL DEPRESSED MEDIAN SECTION
(Freeways)

Figure 8.8A



①	See Chapter Twelve for criteria on lane and shoulder widths.

②	Shoulder cross slopes will be based on surface type. See Chapter Twelve.

③	The clear zone will vary according to design speed, traffic volumes and side slopes. See Section 11.2.

④	The fill slope criteria will depend upon the height of embankment and clear zone criteria. See Section 8.3 for specific criteria.

⑤	The criteria for fore slope, ditch width and back slope will vary. See Section 8.3 for specific criteria.

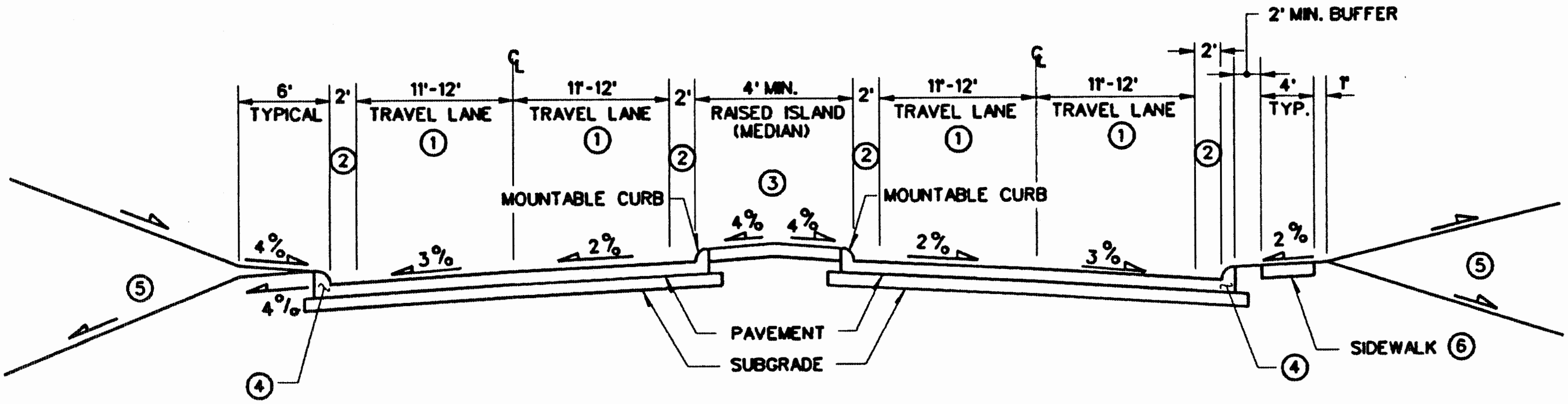
⑥	The depth of ditch should ensure that the flow line for the design discharge is at least 6" below the subgrade interception with the slope. See the *ODOT Drainage Manual* for more information.

⑦	Figure illustrates ditch in earth cut. See Section 8.3 for information on rock cuts.

⑧	See Section 8.3 for details on rounding.

TYPICAL TWO-LANE RURAL ARTERIAL

Figure 8.8B



① See Chapter Twelve for criteria on lane widths.

② Where the design speed is 45 mph or less, the curb offset (for both left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

③ Raised island (median) ≤ 10 ft wide are paved. Medians wider than 10 ft are normally backfilled and sodded. See Section 8.2 for more discussion.

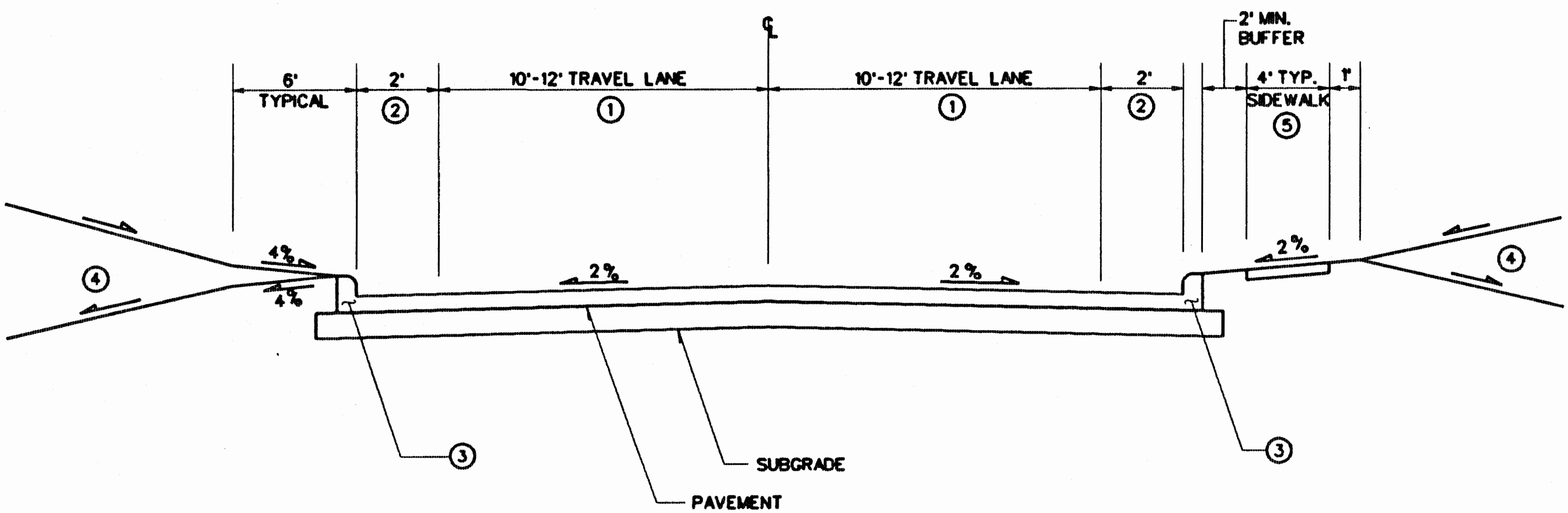
④ Either mountable or barrier curb may be used. See Section 8.1 for more information.

⑤ See Section 8.3 for information on fill and cut slopes.

⑥ See Section 8.1 for information on sidewalk design.

TYPICAL RAISED MEDIAN SECTION
(Suburban/Urban Arterial)

Figure 8.8C



① Lane widths will be based on functional classification and traffic volumes. See Chapter Twelve.

② Where the design speed is 45 mph or less, the curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

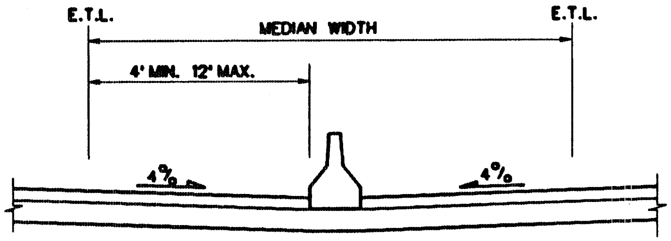
③ Either mountable or barrier curb may be used. See Section 8.1 for more information.

④ See Section 8.3 for information on fill and cut slopes.

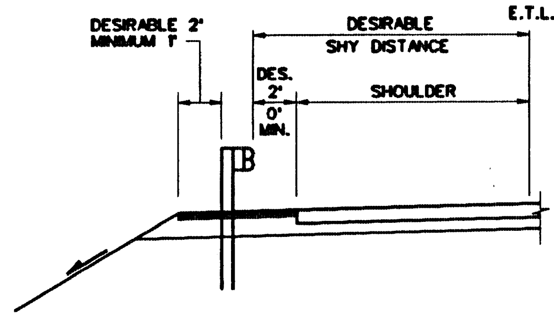
⑤ See Section 8.1 for information on sidewalk design.

TYPICAL TWO-LANE URBAN STREET
(Curbed)

Figure 8.8D

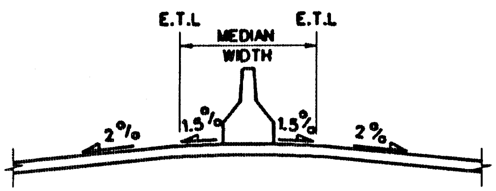


CONCRETE MEDIAN BARRIER
(Wide Left Shoulders)

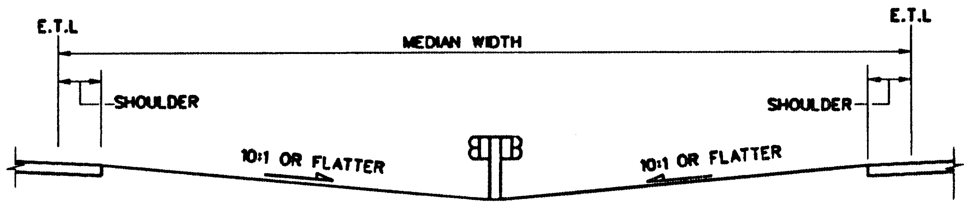


GUARDRAIL WIDENING
(No Curb)

NOTE: SEE ODOT STANDARD DRAWINGS



CONCRETE MEDIAN BARRIERS
(Narrow Left Shoulders)



W-BEAM MEDIAN BARRIER
(Where Warranted)

E.T.L. = EDGE OF TRAVEL LANE

MISCELLANEOUS DETAILS

Figure 8.8E

8.9 REFERENCES

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

2. *ODOT Standard Drawings*, various.

3. *A Policy on Geometric Design of Rural Highways*, AASHTO, 1956.

4. *ODOT Policy on Driveway Regulations for Oklahoma Highways*, Oklahoma Transportation Commission, 1987 Revision.

5. *Standard Specifications for Highway Bridges*, AASHTO, 1989.

6. *Highway Capacity Manual*, Special Report 209, TRB, 1985.

7. *Manual on Uniform Traffic Control Devices*, FHWA, USDOT, 1988.

8. *ODOT Utilities Manual*, ODOT ROW Division, 1987.

9. *ODOT Right-of-Way Specifications Manual and Associated Materials*, ODOT Right-of-Way Division, Engineering Branch, ODOT 1987.

10. *ODOT Policy Manual*, Oklahoma Transportation Commission, Concurrent.

11. *Design of Flexible Channels with Flexible Linings*, FHWA, HEC #15, 1984.

12. *ODOT Drainage Manual*, ODOT Design Division, 1992.

13. *A Policy on the Accommodation of Utilities Within Freeway Right-of-Way*, AASHTO, 1989.

14. *A Guide for Accommodating Utilities within Highway Right-of-Way*, AASHTO, 1981.

15. *Roadside Design Guide*, AASHTO, 1988.

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Chapter Nine

At-Grade Intersections

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Chapter Nine

AT-GRADE INTERSECTIONS

This Chapter discusses the geometric design of at-grade intersections. The intersection is an important part of the highway system. The operational efficiency, capacity, safety and cost of the system depend largely upon intersection design, especially in urban areas. The primary objective of intersection design is to reduce potential conflicts between vehicles, bicycles and pedestrians while providing for the convenience, ease and comfort of those traversing the intersection.

9.1 GENERAL DESIGN CONTROLS

9.1.1 Intersection Alignment

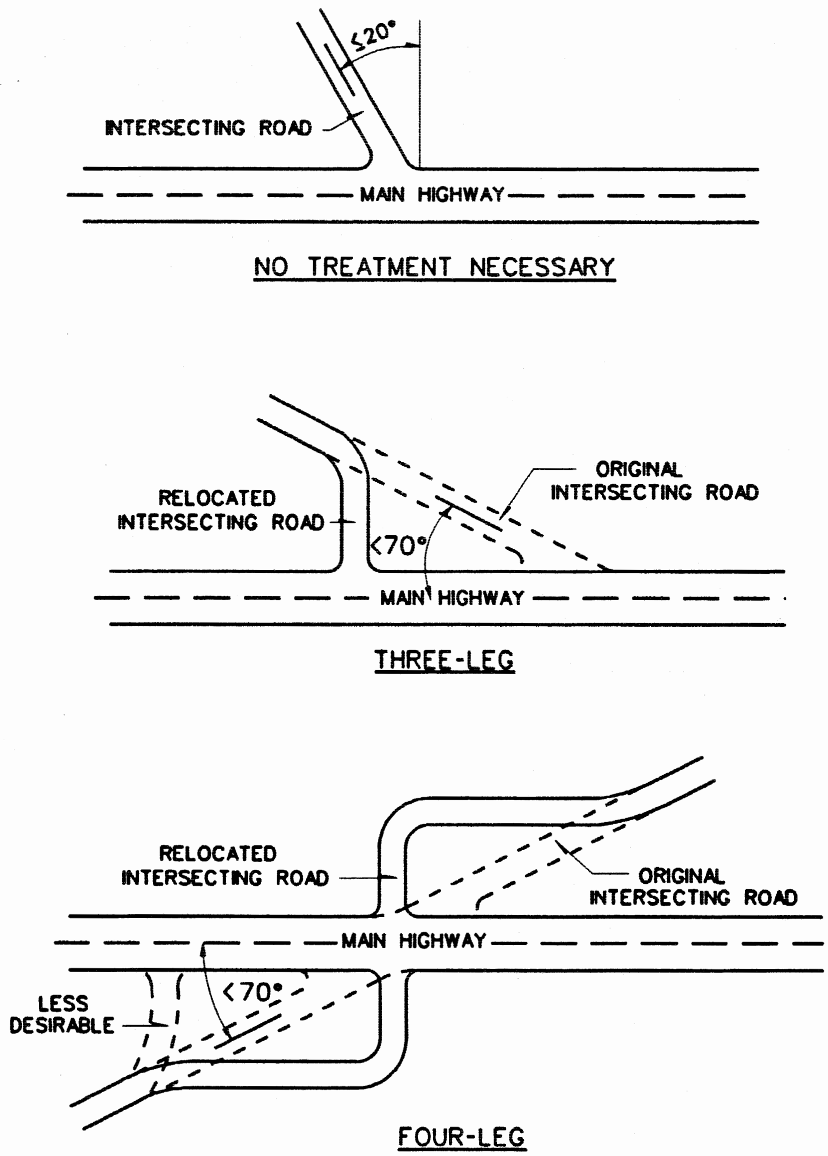
All legs of an intersection should preferably be on a tangent alignment. When a minor road intersects a major road on a horizontal curve, the geometric design of the intersection becomes significantly more complicated, particularly for sight distance, turning movements, channelization and superelevation. If relocation of the intersection is not practical, the designer may be able to realign the minor road to intersect the major road perpendicular to the tangent at a point on the horizontal curve. Although this is an improvement, this arrangement may still result in difficult turning movements if the major road is superelevated.

Roadways should also intersect at approximately right angles. Intersections 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 for vehicles and pedestrians crossing the main traffic flow is increased.
4. The driver's line of sight for one of the sight triangles becomes restricted.

Preferably, the angle of intersection on new construction/reconstruction should be within ±20° of perpendicular. The impact on sight lines and turning movements will not be large. However, intersections outside of this range may warrant more positive traffic control (all stop, traffic signals) or geometric improvements (realignment, greater corner sight distance). Figure 9.1A illustrates various angles of intersection and potential improvements that can be made to the alignment.

Where practical, the realignment options presented in Figure 9.1A should not include sharp horizontal curves on the approaches for the relocated intersecting road. Sharp curves may result in sight distance restrictions or lane encroachments by drivers who smooth out the curve by crossing into the opposing lane, even at low approach speeds.



Note: Short horizontal curves on approaches of intersecting roads should be avoided. See discussion in Section 9.1.1.

TREATMENTS FOR SKEWED INTERSECTIONS

Figure 9.1A

9.1.2 Intersection Profile

The designer should avoid combinations of grade lines that make vehicular control difficult at intersections. The following criteria will apply.

9.1.2.1 Approach Gradient

The gradients of intersecting highways should be as flat as practical on those sections that will be used for storage of stopped vehicles. The gradient of the storage platform desirably will not exceed 2% on each intersecting leg within 100 ft (rural) and 50 ft (urban) of the intersection. Gradients greater than 3% should be avoided, if practical. Where the gradient exceeds 3%, the designer must exercise special care in evaluating the intersection design features (e.g., corner sight distance, stopping sight distance). In general, however, any gradient through the intersection must reflect the practicalities of matching the basic profiles of the intersecting roadways. This is discussed in the following sections.

9.1.2.2 Cross Section Transitions (Stop-Controlled Intersections)

One or both of the approaching legs of the intersection may need to be transitioned (or warped) to meet the cross section of the crossing road. The profile and cross section of the major road will normally be maintained through an intersection, and the cross section of the stop-controlled legs will be transitioned to match the major road cross slope and profile. See Figure 9.1B.

The designer should also consider the probability that a stop-controlled intersection will be converted to a signalized intersection in the future. If it is likely to be converted, the criteria in Section 9.1.2.3 should be considered.

9.1.2.3 Cross Section Transitions (Signalized Intersections)

The cross section of the minor road will typically be transitioned to meet the profile and cross section of the major road. If both intersecting roads have approximately equal importance, both roadways may be transitioned to form a plane section through the intersection. Where compromises are necessary between the two, the smoother riding characteristics should be provided for the roadway with the higher traffic volumes and operating speeds. See Figure 9.1B.

9.1.2.4 Transition Rates

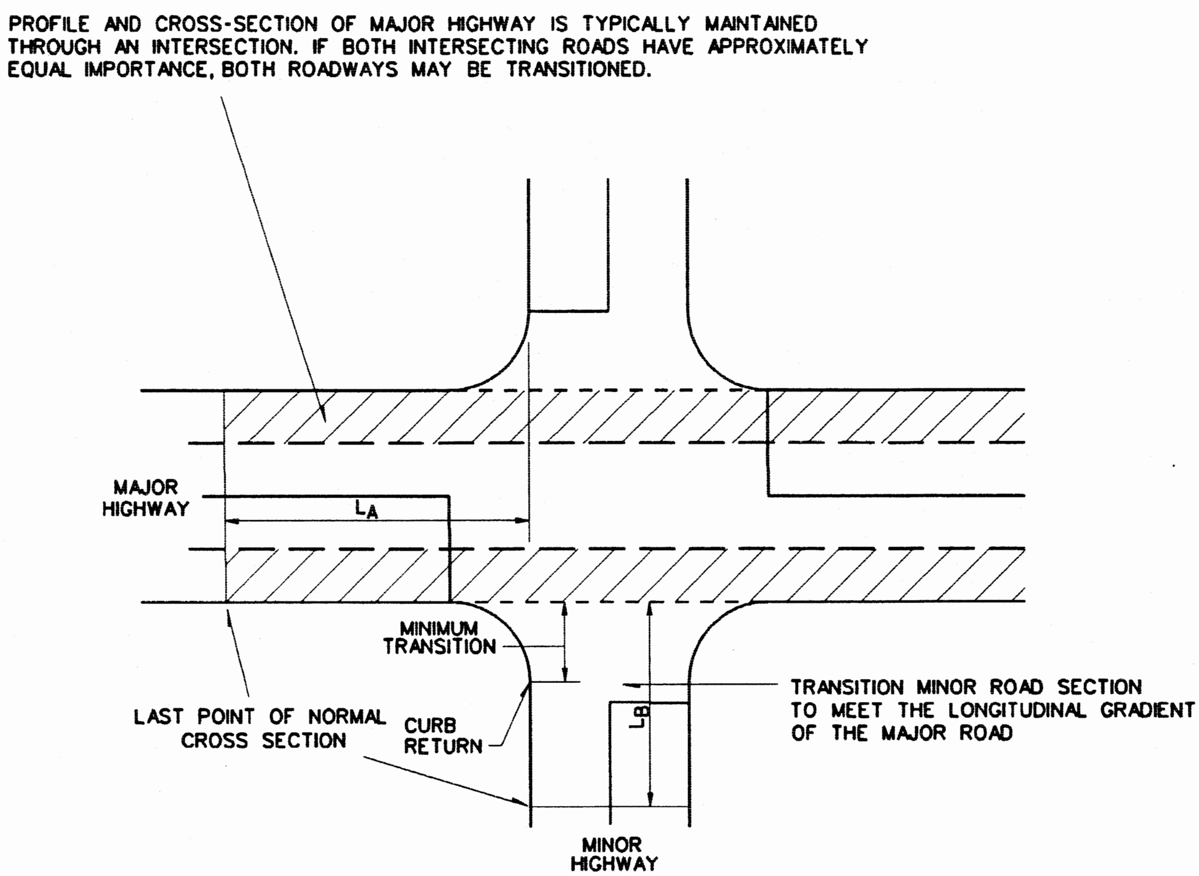
Where one or both intersecting roadways are transitioned, the designer must determine the length and rate of transition from the normal section to the modified section. Desirably, the transition will be designed to meet the general principles of superelevation transition which apply to that roadway (i.e., open-road or low-speed urban street conditions). See Section 6.2 for a complete discussion on superelevation development. Where these criteria are applied to transition rates, the applied design speed is typically:

1. 20 mph for a stop-controlled leg,

2. the highway design speed for a free-flowing leg, or

3. the highway design speed for all legs of a signalized intersection.

At a minimum, the approaching legs of an intersection will be transitioned within the curb returns of the intersection consistent with practical field conditions.



Notes:

1. See discussion in Section 9.1.2.

2. Spot elevate pavement area based on drainage requirements, roadway profiles and the turning path of the design vehicle.

PAVEMENT TRANSITIONS THROUGH INTERSECTIONS

Figure 9.1B

9.1.2.5 Vertical Profile

Where the cross section of the minor road is warped to meet the major road, this will result in angular breaks for traffic on the minor road if no vertical curve is inserted. The following options are presented in order from the most desirable to the least desirable (see Figure 9.1C):

1. Vertical Curves (SSD). Desirably, vertical curves will be used through an intersection which meet the criteria for stopping sight distance as described in Chapter Seven. At stop-controlled legs, the vertical curve should be designed to meet a design speed of 20 mph; at free-flowing legs and at signalized intersections, the design speed of the roadway should be used to design the vertical curve.

2. Sag Vertical Curves (Comfort). For sag vertical curves, the next most desirable option is to design the sag to meet the comfort criteria:

   $$L = \frac{AV^2}{46.5}$$

   Where:

   L = length of vertical curve, ft

   A = algebraic difference between grades, %

   V = design speed, mph

3. Vertical Curves (Minimum Comfort). If a design based on SSD or comfort is not practical, the next most desirable design for vertical curves at intersection approaches is based on the following formulas:

   $K = (.1V)^2$ Sag Curves

   $K = (.07V)^2$ Crest Curves

   $L = KA$

   Where:

   K = the horizontal distance in ft needed to produce a 1% change in the gradient along the curve

   A = algebraic difference between the two tangent grades, %

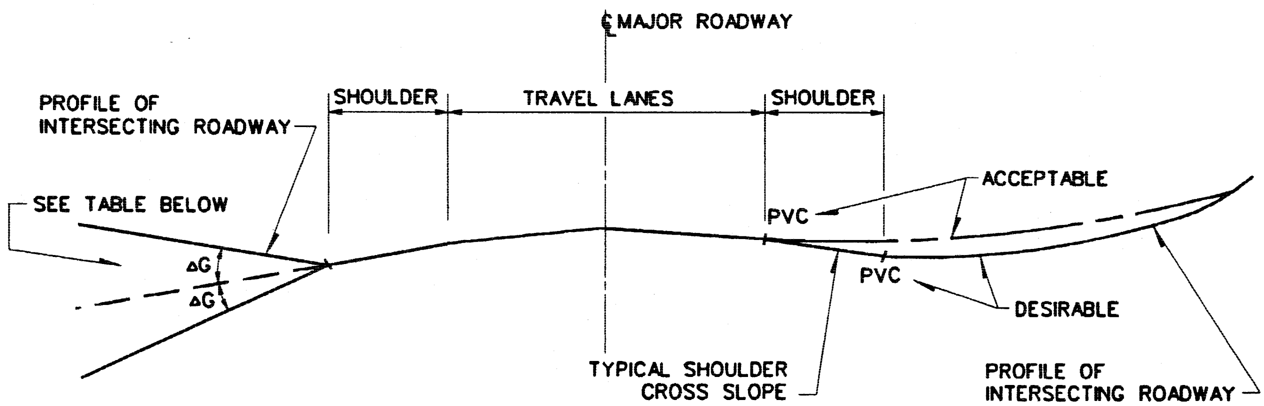
   V = design speed, mph

   L = length of vertical curve, ft

4. Angular Breaks. At some intersections, it may be impractical to provide vertical curves on the approaches; i.e., angular breaks are necessary through the intersection. In some cases, angular breaks may allow other intersection geometric features, such as sight distance, storage platform and drainage, to function better. Figure 9.1C presents a schematic of vertical profiles through an intersection. The figure also indicates maximum angular breaks based on the selected design speed. Use 20 mph for a stop-controlled leg and the roadway design speed for free-flowing legs and legs at signalized intersections. Where angular breaks are used, the minimum chord distance between angle points is 20 ft.

9.1.2.6 Drainage

The profile and transitions at all intersections should be evaluated for impacts on drainage. See the *ODOT Drainage Manual* and Chapter Fifteen for more information.



Maximum Change in Grades Without Vertical Curve (ΔG)		
Design Speed (mph)	Sag	Crest
20	5.0%	8.5%
25	3.2%	6.5%
30	2.2%	4.5%
35	1.6%	3.3%
40	1.25%	2.5%
45	1.00%	2.0%
50	0.80%	1.6%

Notes:

1. See Section 9.1.2 for a discussion on vertical profiles through an intersection.
2. If practical, the gradient of the approach roadway where vehicles may store should not exceed 2%.
3. Actual field conditions will determine the final design.

VERTICAL PROFILES OF INTERSECTING ROADS

Figure 9.1C

9.1.3 Capacity and Level of Service

A capacity analysis should be performed before the detailed design of an intersection. This analysis will influence several geometric design features including the 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 twenty years for new construction and reconstruction. Level of service criteria are provided in the tables of geometric design in Chapters Twelve and Thirteen. Criteria for design-year selection are provided in Chapter Five. Once the level of service and design traffic volumes are determined, the designer should use the *Highway Capacity Manual* for the detailed capacity analyses.

In performing the capacity analysis, the designer should at some intersections evaluate both an unsignalized intersection (short term) and a signalized intersection (long term). This evaluation may reveal that the geometrics of the (current) unsignalized intersection should be designed to accommodate the (future) signalized intersection.

9.1.4 Types of Intersections

At-grade intersections are usually a 3-leg (T shape) or 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, character or composition of traffic, design speed and angle of intersection.

Multileg intersections are those with five or more intersection legs; these should be avoided wherever practical. Where volumes are light and stop control is used, it may be satisfactory to have all intersection legs intersect at a common, all-paved area. At other than minor intersections, safety and efficiency are improved by rearrangements that remove some conflicting movements from the major intersection. These rearrangements are accomplished by realigning one or more of the intersecting legs and combining some of the traffic movements at adjacent subsidiary intersections or, in some cases, making one or more legs one-way away from the intersection.

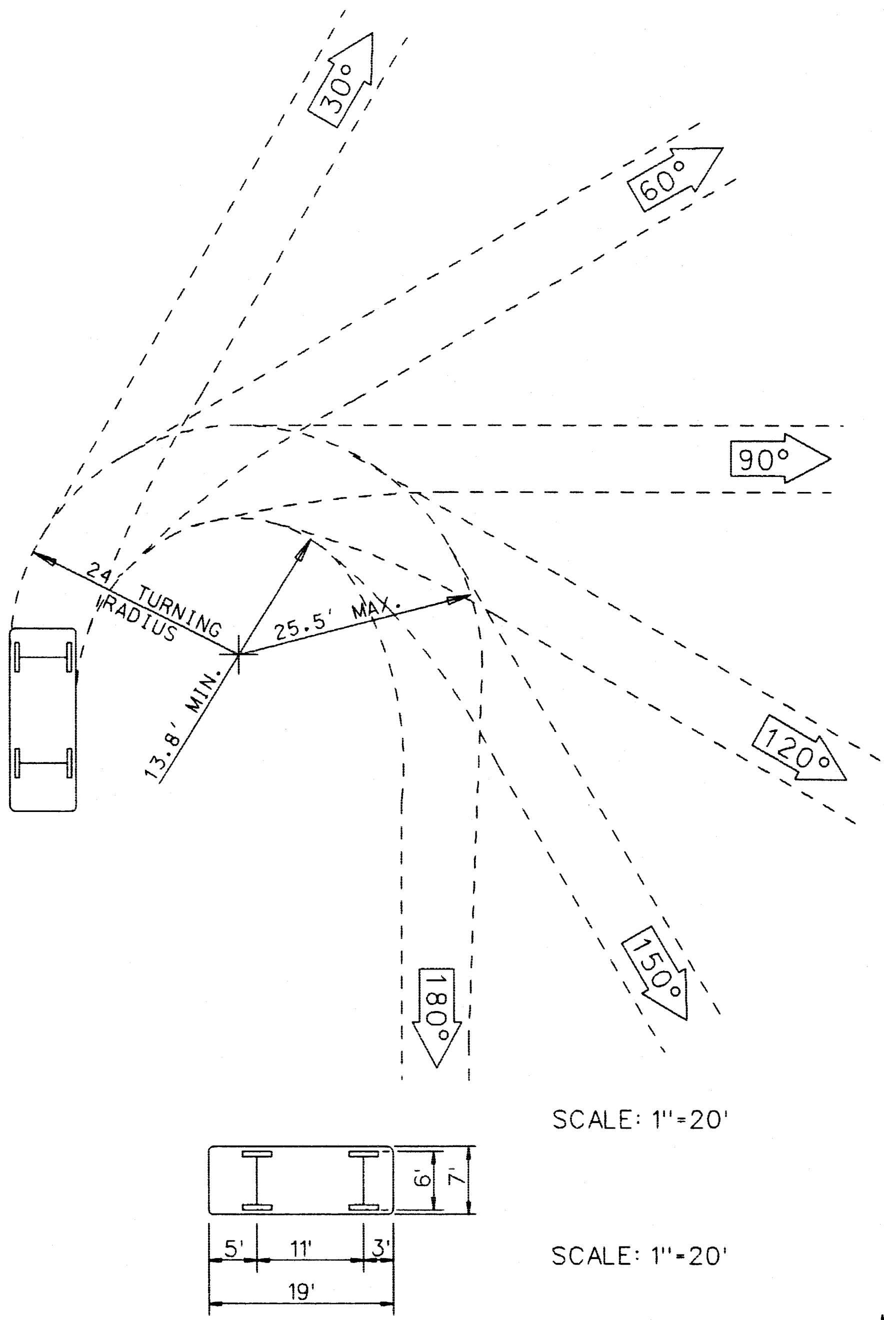
When creating a new intersection, the designer must ensure that there is sufficient distance between adjacent intersections so that they form two distinct intersections. At a minimum, they should be at least 300 ft apart and, desirably, ¼ mile apart when each intersection is signalized. On a suburban or urban arterial with many intersections, regular spacings between intersections typically promote improved traffic operations.

9.1.5 Design Vehicles

9.1.5.1 Types

The following design vehicles are typically used in intersection design:

1. P -- Passenger car and light panel/pickup truck (see Figure 9.1D).
2. SU -- Single Unit Truck or Small Bus (see Figure 9.1E).
3. WB-50 -- Semitrailer combination with an overall wheelbase of 50 ft (see Figure 9.1F).
4. WB-67 -- Semitrailer combination with a 53-ft trailer (see Figure 9.1G).
5. WB-114 -- Semitrailer combination with two 48-ft trailers (see Figure 9.1H).

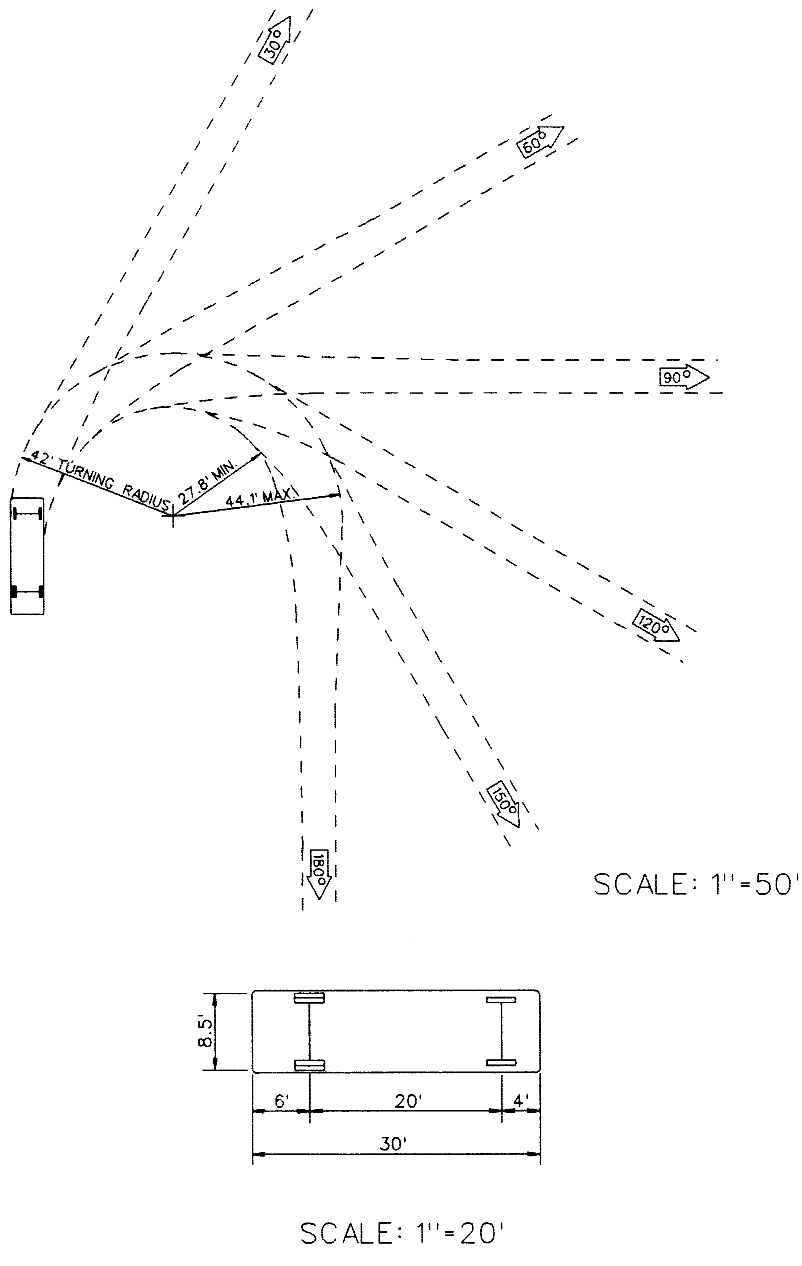


Source: (1)

Note: This turning template shows the turning paths of the AASHTO design vehicle. The paths shown are for the left front overhang and the outside rear wheel, which is the vehicular swept width. The path of the left front wheel (not shown) follows the circular curve of the left front overhang.

MINIMUM TURNING PATH FOR P DESIGN VEHICLE

Figure 9.1D

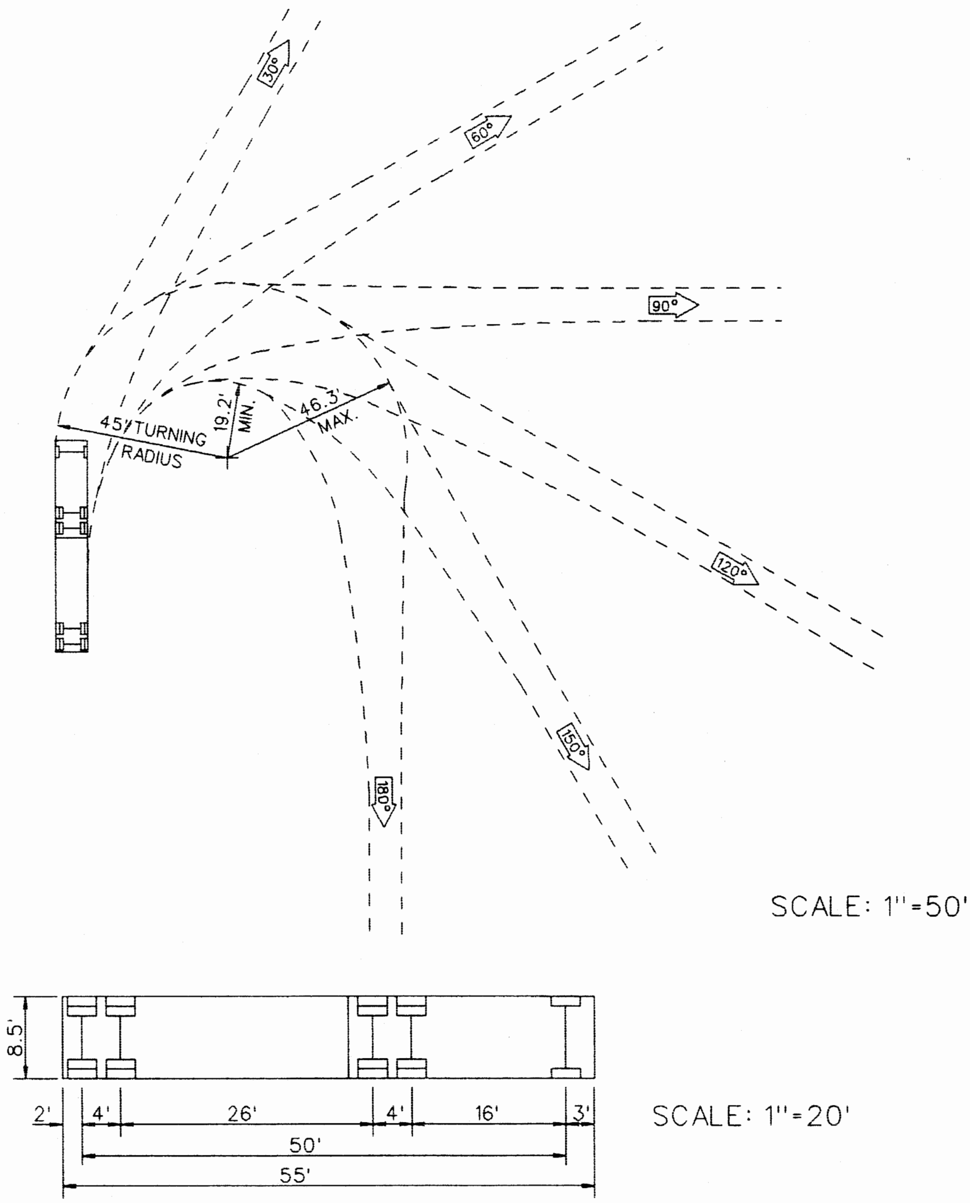


Source: (1)

Note: This turning template shows the turning paths of the AASHTO design vehicle. The paths shown are for the left front overhang and the outside rear wheel, which is the vehicular swept width. The path of the left front wheel (not shown) follows the circular curve of the left front overhang.

MINIMUM TURNING PATH FOR SU DESIGN VEHICLE

Figure 9.1E

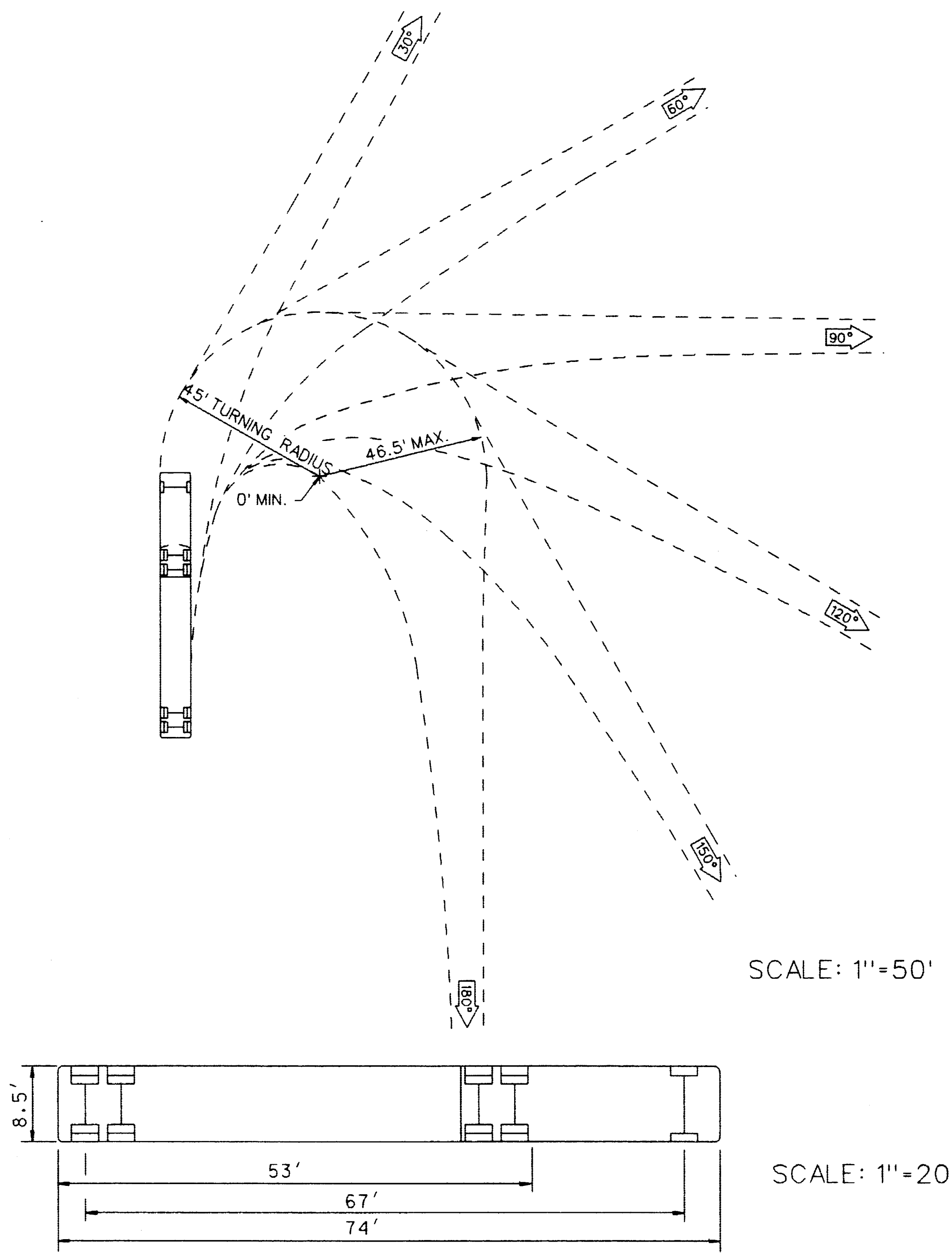


Source: (1)

Note: This turning template shows the turning paths of the AASHTO design vehicle. The paths shown are for the left front overhang and the outside rear wheel, which is the vehicular swept width. The path of the left front wheel (not shown) follows the circular curve of the left front overhang.

MINIMUM TURNING PATH FOR WB-50 DESIGN VEHICLE

Figure 9.1F

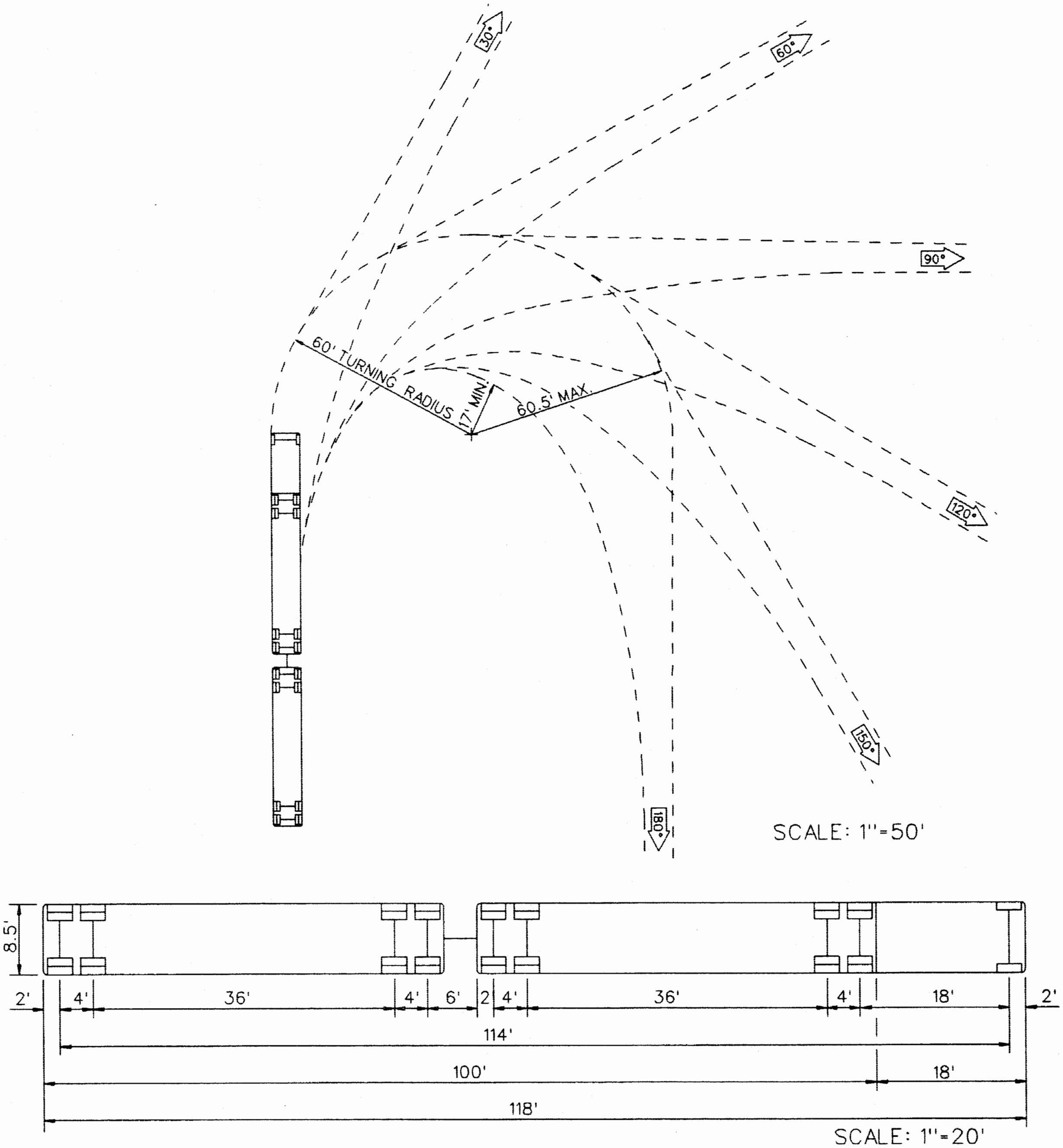


Source: (1)

Note: This turning template shows the turning paths of the AASHTO design vehicle. The paths shown are for the left front overhang and the outside rear wheel, which is the vehicular swept width. The path of the left front wheel (not shown) follows the circular curve of the left front overhang.

MINIMUM TURNING PATH FOR WB-67 DESIGN VEHICLE

Figure 9.1G



Source: (1)

Note: This turning template shows the turning paths of the AASHTO design vehicle. The paths shown are for the left front overhang and the outside rear wheel, which is the vehicular swept width. The path of the left front wheel (not shown) follows the circular curve of the left front overhang.

MINIMUM TURNING PATH FOR WB-114 DESIGN VEHICLE

Figure 9.1H

Section 5.5 presents vehicular dimensions and turning characteristics for all AASHTO design vehicles.

9.1.5.2 Selection

In general, the selected design vehicle should be based on the largest vehicle that will use the intersection with some frequency. The designer should consider the area type and the functional classification of the intersecting highways. Table 9.1A presents the desirable and minimum design vehicles for all intersections, both rural and urban. Selection of the applicable design vehicle is based on the functional class of highways which the vehicle is turning from and onto.

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 a WB-67 design vehicle, but provide painted channelization for a passenger car. This will provide a positive indicator for the more frequent turning vehicle.

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.

In some areas, it may be necessary to use a larger design vehicle than suggested in Table 9.1A. For example, if a WB-67 is known to travel a particular route with frequency, it should be used as the design vehicle instead of the suggested design vehicle in the tables. In general, all State highways to State highway intersections are designed using the WB-67 as the minimum design vehicle. In addition, the WB-114 design vehicle should be used for all turning movements at intersections on routes which meet the ODOT reasonable access criteria for the National Network. The designer should contact the ODOT Planning Division for information on reasonable access routes.

9.1.6 Offset Intersections

In general, 4-leg intersections should be designed such that opposing approaches line up with each other; i.e., there is no offset between opposing approaches. However, this is not always practical. Figure 9.1I presents a diagram of an intersection with an offset between opposing approaches. The following criteria will apply for offset intersection approaches:

1. Maximum Offset. The maximum offset is determined from the application of a taper equal to V:1 applied to the intersection width, where V is the design speed in mph. See Figure 9.1I. In restricted locations and where $V \leq 40$ mph, the applied taper may be $V^2/60$. V is selected as follows:

   a. V = 20 mph for stop-controlled approaches.

   b. V = the roadway design speed for the free-flowing approaches at a stop-controlled intersection.

   c. V = the roadway design speed for both approaches at a signalized intersection.

2. Turning Conflicts. The entire intersection should be evaluated for conflicts which may result from turning vehicles at an offset intersection. For example, offsets where the "jog" is to the left may result in significant interference between simultaneous left-turning vehicles.

Table 9.1A

SUGGESTED DESIGN VEHICLE SELECTION

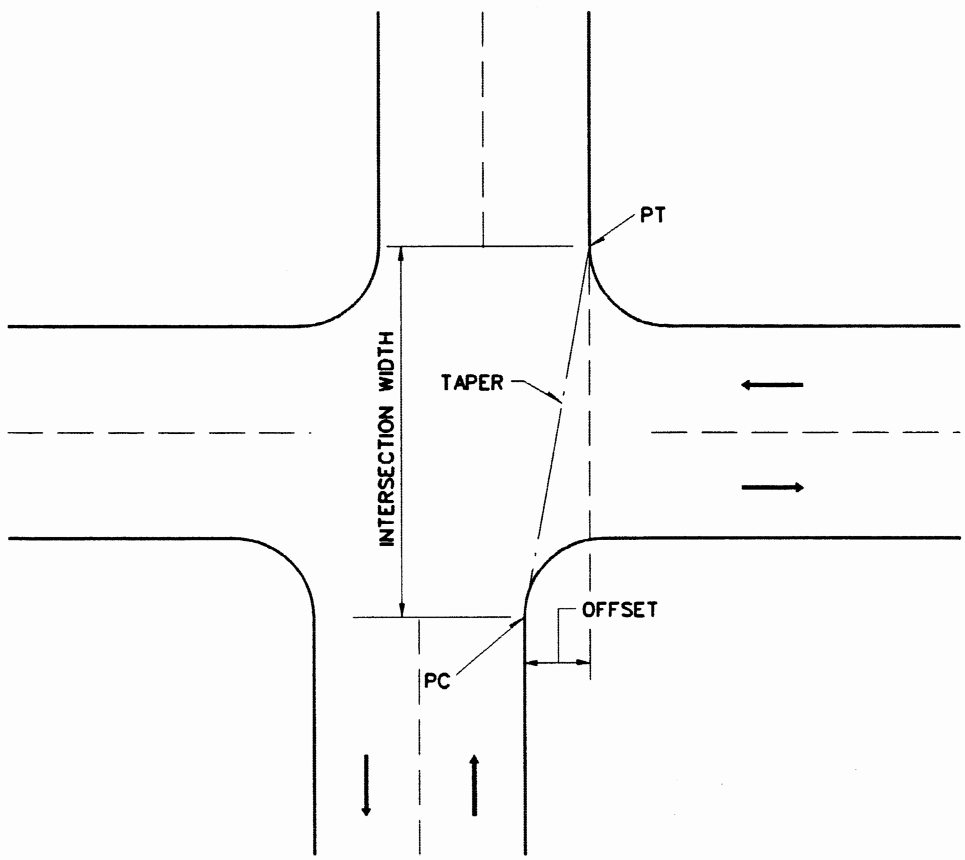
For Turn Made From	For Turn Made Onto	Suggested Design Vehicle	
		Desirable	Minimum
Interchange Ramp	Other Facilities	WB-114	WB-67
Other Facilities	Interchange Ramp	WB-114	WB-67
State Highway	State Highway	WB-114	WB-67
Arterial*	Arterial Collector Local	WB-67 WB-67 WB-50	WB-67 WB-50 SU
Collector*	Arterial Collector Local	WB-67 WB-67 BUS	WB-50 SU SU
Local*	Arterial Collector Local	WB-50 BUS SU	SU SU SU

* These criteria apply where one or both of the intersecting legs are not on the State highway system.

Notes:

1. The WB-114 design vehicle is allowed on all multilane, divided facilities with partial or full control of access on the Oklahoma State Highway System and on those routes providing reasonable access (as part of the National Network) to and from the State highways.

2. The WB-67 design vehicle is allowed on all facilities on the State highway system.



Notes:

1. Desirable taper rate is V:1, where V = design speed.
2. See discussion in Section 9.1.6 for more information.

OFFSET INTERSECTION

Figure 9.1I

3. Evaluation Factors. In addition to potential vehicular conflicts, the designer should evaluate the following at existing or proposed offset intersections:

   a. through and turning volumes;

   b. type of traffic control;

   c. impact on all turning maneuvers;

   d. intersection geometrics (e.g., sight distance, curb/pavement edge radii); and

   e. accident history at existing intersections.

At proposed offset intersections, the designer should coordinate with the Urban Design Division, Geometric Design Branch, and the Traffic Engineering Division on intersection design and traffic control requirements.

9.2 INTERSECTION SIGHT DISTANCE (ISD)

For an at-grade intersection to operate properly, adequate sight distance needs to be provided. The designer should provide sufficient sight distance for a driver to perceive potential conflicts and to perform the actions needed to negotiate the intersection safely.

The additional costs and impacts of removing sight obstructions are often justified. If it is impractical to remove an obstruction blocking the sight distance, the designer should consider providing traffic control devices or design applications (warning signs, traffic signals or turn lanes) which may not otherwise be warranted.

The following sections present ISD criteria for various intersection conditions. Where practical, longer sight distances are always preferred.

9.2.1 No Traffic Control

Intersections between low-volume and low-speed roads/streets may have no traffic control. Drivers approaching these intersections should have sufficient corner sight distance to adjust speed to avoid a collision.

Figure 9.2A presents the applicable ISD criteria for intersections with no traffic control. These criteria are based on that distance needed by a driver approaching the intersection to perceive another vehicle approaching from a crossing road and to avoid a collision. Specifically, the ISD distances represent that distance traveled by either vehicle in 3 seconds at the roadway design speed -- 2 seconds for perception/reaction time and 1 second for brake actuation.

DRIVEWAYS ARE CONSIDERED STOP-CONTR

9.2.2 Stop-Control (Desirable ISD)

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 without being overtaken by an approaching vehicle on the major road.

This section presents desirable ISD criteria which should apply, where practical, to stop-controlled intersections. Section 9.2.3 presents minimum ISD criteria, which will apply to restricted locations.

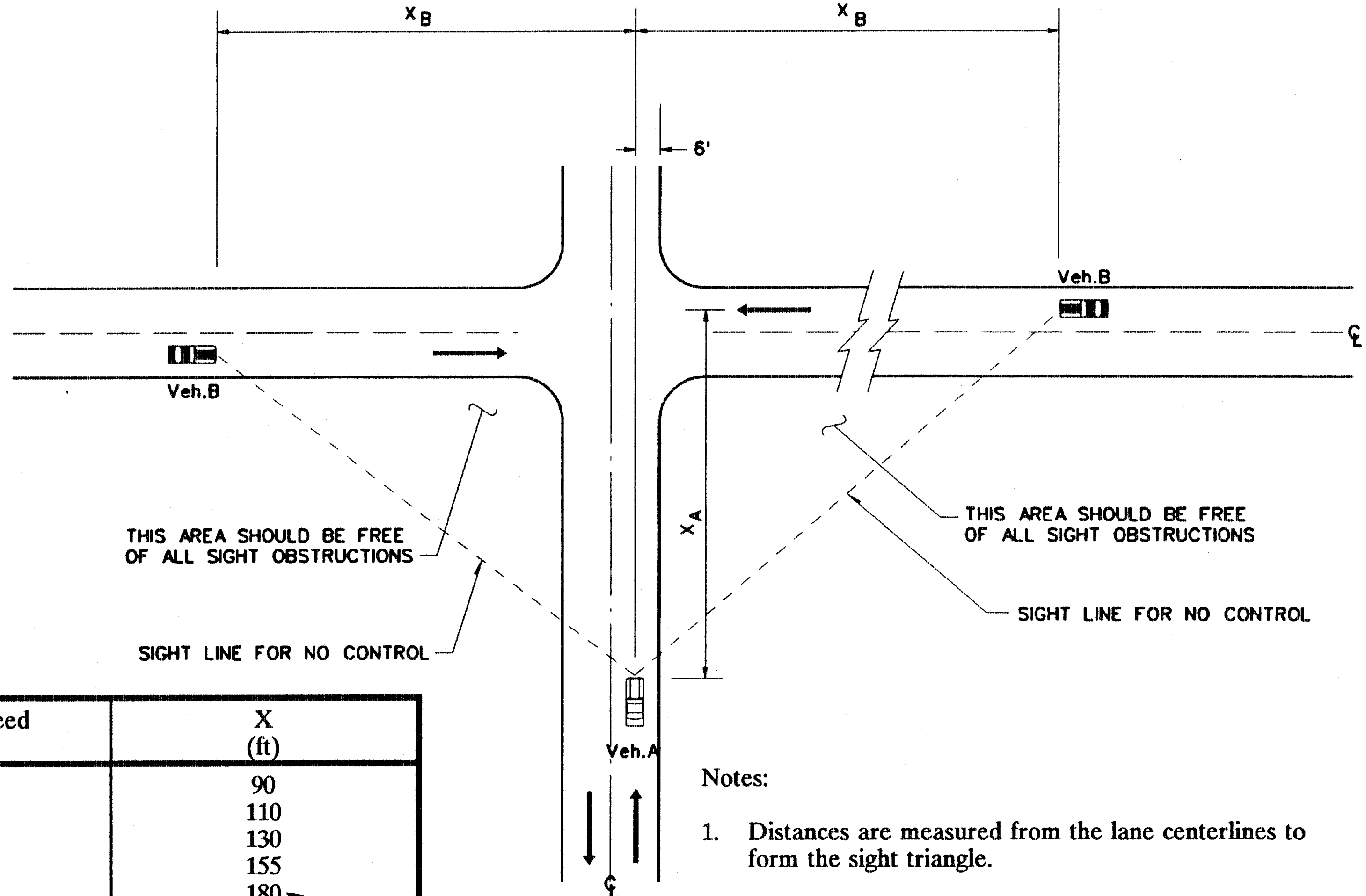
9.2.2.1 Theoretical Discussion

Figure 9.2B illustrates the application of the ISD criteria for stop-controlled intersections. The ISD model, in summary, assumes that a mainline driver approaches an intersection at the design speed as a vehicle enters the highway from a side road ahead of the

per Mohamed, 2-4-09
Rule of Thumb →
place end of GR (i.e. BGT)
50' from edge of
Radius of driveway.
Line of Sight = 3D
Ht of Driver in D/W = 3.5'
Ht of GR = 2.5' (not an obstruction)

$$H = P - D_{np} + R - FD - L$$

(Equation 9-3)



Design Speed (mph)	X (ft)
20	90
25	110
30	130
35	155
40	180
45	200
50	220
55	245
60	265

Notes:

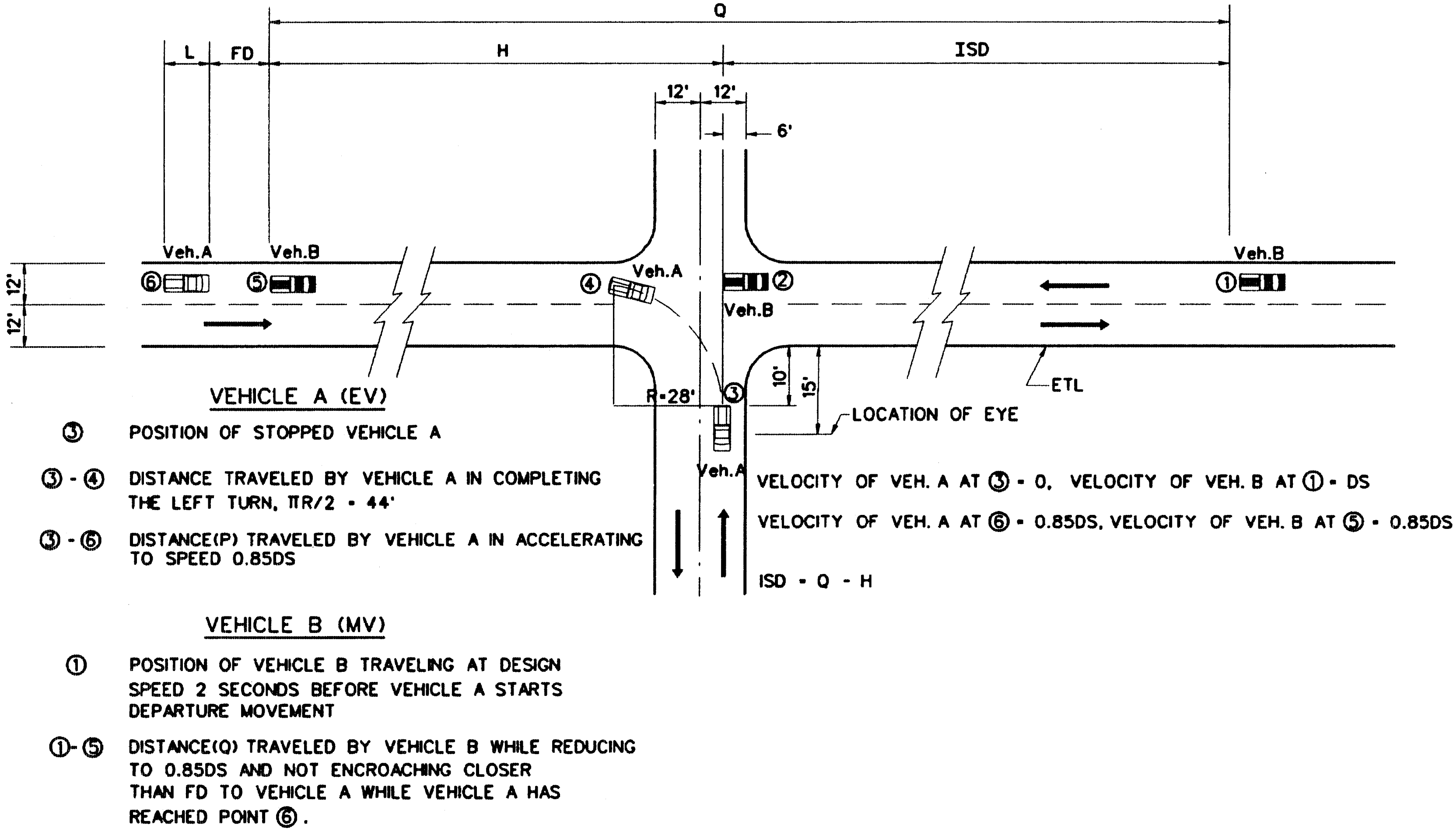
1. Distances are measured from the lane centerlines to form the sight triangle.

2. See Section 9.2.1 for more information.

Source: (1) Revised

INTERSECTION SIGHT DISTANCE
(No Traffic Control)

Figure 9.2A



Notes:
1. See Section 9.2.2 for definition of terms.
2. ISD for right-turning vehicle is determined in a similar manner.

Source: (1) & (3)

INTERSECTION SIGHT DISTANCE AT A STOP-CONTROLLED INTERSECTION (Theoretical)

Figure 9.2B

Where:

ISD = sight distance along major road to the right or to the left of the entering vehicle needed for the turning maneuver, ft

Q = distance traveled by major-road or mainline vehicle (MV) during entry maneuver by minor-road vehicle (EV), ft

H = major road vehicle's distance from intersection when at the assumed following distance from minor road vehicle, ft

V = design speed of major road, mph

D_{dec} = distance MV travels during deceleration from design speed to 85% of design speed, ft

t_{dec} = time MV is decelerating, sec

t_{dec} = $(2 \times D_{dec})/1.47(V + 0.85V)$

t_{85} = time MV is at 85% of design speed during entry maneuver by EV, sec

t_{85} = $t_{as} - t_{ds} - t_{dec}$

t_{as} = total time for entering vehicle (EV) on minor road to enter major road and reach 85% of design speed (including driver perception/reaction time), sec

t_{as} = $t_{ar} + J$

t_{ar} = time for EV on minor road to accelerate from a stop to 85% of design speed, sec

J = EV driver perception time and reaction time (assume 2 seconds) required to actuate clutch or automatic shift, sec

t_{ds} = time major road vehicle is at design speed, sec

t_{ds} = $J + t_{pr}$

t_{pr} = perception/reaction time (assume 2 seconds) for MV driver, sec

D_{np} = $\pi R/2$ = radial distance traveled by EV in negotiating a 90° turn onto major road, ft

R = radius of turn for EV (assumed to be 28 ft for passenger cars and 60 ft for trucks)

P = total distance traveled by EV from stopped position to location where 85% of design speed is reached, ft

FD = safe following distance for MV to trail the EV entering from the minor road, ft

FD = $1.47 \times 0.85V \times t_{FD}$

t_{FD} = 2 seconds

L = length of passenger car, ft

The ISD criteria used by the ODOT are intended to provide both an acceptable level of safety and an economical, constructible design for intersections. The model is intended to assign a reasonable level of responsibility to both the entering vehicle (EV) and mainline vehicle (MV). The following summarizes the major assumptions within the ISD model:

1. Design Vehicle. The selected design vehicle greatly affects the ISD values. The recommended minimum ISD numbers are based on a passenger car;

however, there may be sites where it is desirable to use truck acceleration rates to determine the ISD values.

2. Design Vehicle Length (L). For passenger cars the vehicle length is 20 ft. For trucks, the WB-67 truck length of 74 ft is used. If the WB-50 is used as the design vehicle, then 19 ft can be subtracted from the truck ISD values.

3. Reaction Time of EV (J). The model assumes 2 seconds for the EV driver to release the brake and depress the accelerator.

4. Acceleration Rate of EV. Both the passenger car and truck acceleration rates are from the 1990 *Green Book*; Table 9.2A reproduces these rates.

5. Reaction Time of MV(t_{pr}). This is the time required from the moment the entering vehicle begins its maneuver until the mainline driver releases his foot from the accelerator. This is assumed to be 2 seconds.

6. MV Deceleration (D_{dec}). The MV must decelerate from the mainline design speed to 85% of the mainline design speed (for a turning passenger car) or to 65% of the mainline design speed (for a turning truck). Deceleration is assumed to be 3.3 mph/sec to 55 mph. Deceleration rates to speeds of 50 mph and below are based on Figure II-17 of the 1990 *Green Book.*

7. Reduced Speed of MV. For passenger cars, the ISD model assumes that the MV will reduce its speed to 85% of the mainline design speed. Likewise, this is the speed to which the EV will accelerate before being overtaken by the MV. For the truck ISD model, the reduced speed is 65% of the mainline design speed.

8. Following Distance (FD). This is the distance between the MV and the EV when the EV has accelerated to 85% or 65% of the design speed on the major road. The FD is based on providing two seconds of travel time at the design speed.

9. Eye Location. The ISD values will establish one leg of the sight triangle which needs to be visible to the entering vehicle. The leg on the stop-controlled road or street will be determined by the assumed location of the driver eye. This is established as 15 ft behind the edge of the travel lane (see Figure 9.2B), regardless of the location of the painted stop bar.

10. Height of Eye/Object. The height of eye is 3.5 ft for passenger cars and 8 ft for trucks. The height of object (an approaching passenger car) is 4.25 ft.

9.2.2.2 Application

Figure 9.2C illustrates the application of ISD to a stop-controlled intersection, and Table 9.2B provides the criteria for passenger car and truck ISD values. The designer should also consider the following when determining ISD criteria:

1. Multilane. Table 9.2B applies to both 2-lane and multilane facilities. However, also see Comment #3.

2. Turn Maneuver. Theoretically, there is only a minimal difference in the ISD values between a left- and right-turning vehicle. Consequently, only one value is provided in Table 9.2B.

3. Medians. For a multilane facility which does not have a median wide enough to store a stopped design vehicle, the criteria in Table 9.2B should be used directly. On

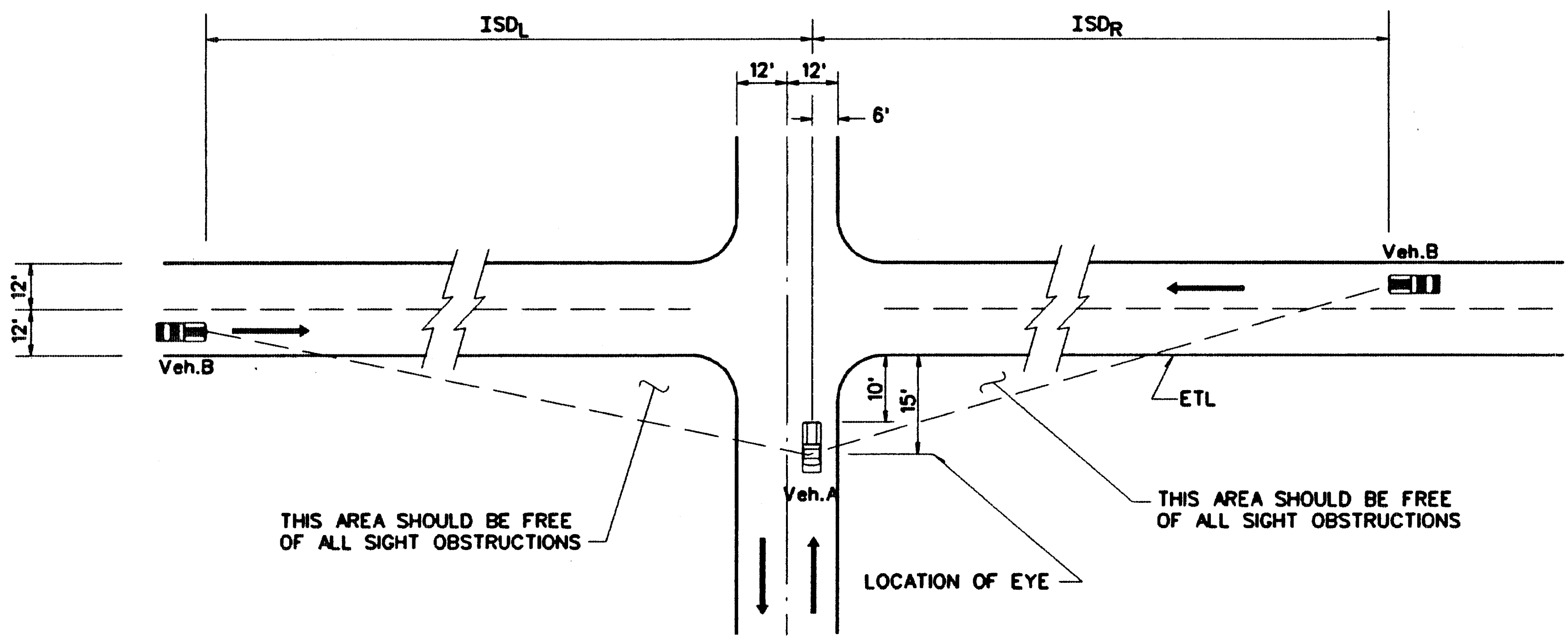
Table 9.2A

ACCELERATION RATES
(From a Stop)

Speed Reached (mph)	Passenger Cars		Trucks*	
	Distance (ft)	T_a (sec)	Distance (ft)	T_a (sec)
15	50	4.5	105	10.8
20	90	6.1	160	13.0
25	140	7.3	290	16.9
30	215	9.4	570	23.8
35	305	11.3	1080	34.5
40	420	13.5	1870	48.9
45	570	15.9	2900	65.4
50	760	18.6	4600	89.8
55	1000	21.7	-	- 121
60	1315	25.4	-	- 161
65	1735	30.0	-	- 209
70	2320	35.9	-	- 265sec.

Source: (1)

* Acceleration rates based on a 300 lb/hp truck.



Note: See Section 9.2.2 for definition of terms.

INTERSECTION SIGHT DISTANCE AT A STOP-CONTROLLED INTERSECTION
(Application)

Figure 9.2C

Table 9.2B

DESIRABLE INTERSECTION SIGHT DISTANCES
(For Stop-Controlled Intersections)

Mainline Design Speed (mph)	Passenger Cars* (ft)	Trucks** (ft)
20	220	325
25	280	400
30	355	495
35	440	595
40	525	705
45	635	845
50	765	995
55	895	1185
60	1035	1420
65	1190	-
70	1375	-

* Reaches 85% of Mainline Design Speed
** Reaches 65% of Mainline Design Speed

Note: See Figure 9.2B for specific application of ISD to intersections.

multilane facilities with a median wide enough to store a stopped design vehicle, the designer should evaluate the ISD requirements in two steps:

a. With the vehicle stopped on the side road, the ISD will be checked to the left on the mainline.

b. With the vehicle stopped in the median, the ISD will be checked to the right on the mainline.

4. Grades. The model assumes that the roadway is relatively level where the entering vehicle is accelerating. If this is not the case, the acceleration times in Table 9.2A may be adjusted. Use the AASHTO *Green Book* to determine the impacts of grades on ISD.

9.2.3 Stop Control (Minimum ISD)

In general, many intersections currently operate with sight distances less than those presented. For practical reasons, minimum ISD criteria should reflect actual field operations. These criteria may be based on typical gaps in the major road traffic that are accepted by the minor road driver.

Therefore, at restricted locations, the following minimum ISD criteria apply to stop-controlled intersections:

1. Passenger Cars. ISD should be available based on 8 seconds of travel time at the design speed.

2. Trucks. Where a truck is selected as the design vehicle, ISD should be available based on 12 seconds of travel time at the design speed.

Table 9.2C presents the minimum ISD criteria. Figure 9.2C illustrates their application. Note that, as indicated in the table at the lower design speeds, the ISD criteria based on an overtaking vehicle (Table 9.2B) are less than the criteria in Table 9.2C. In these cases, the minimum ISD criteria is the lower of the two numbers.

9.2.4 Yield Control

At intersections controlled by a yield sign, drivers on the minor road will typically:

1. slow down as they approach the major road,

2. based on their view of the major road, make a stop/accelerate decision, and

3. either brake to a stop or continue their turning maneuver onto the major road.

Figure 9.2D presents the applicable ISD criteria for intersections controlled by a yield sign. These criteria are based on the assumption that the stop/accelerate decision is made at 10 mph. Therefore, the leg of the sight triangle on the minor road is determined by the stopping sight distance (SSD) for 10 mph, or 50 ft. The leg of the sight triangle on the major road is based on many of the same assumptions used for stop-controlled intersections. See Section 9.2.2 for specific information. In summary, the assumptions for the major leg at a yield-controlled intersection are:

4. Design Vehicle. The passenger car is used.

5. Entering Vehicle (EV). The EV acceleration rate is from the 1990 AASHTO *Green Book*. The EV accelerates from 10 mph to 85% of the design speed of the major road.

Table 9.2C

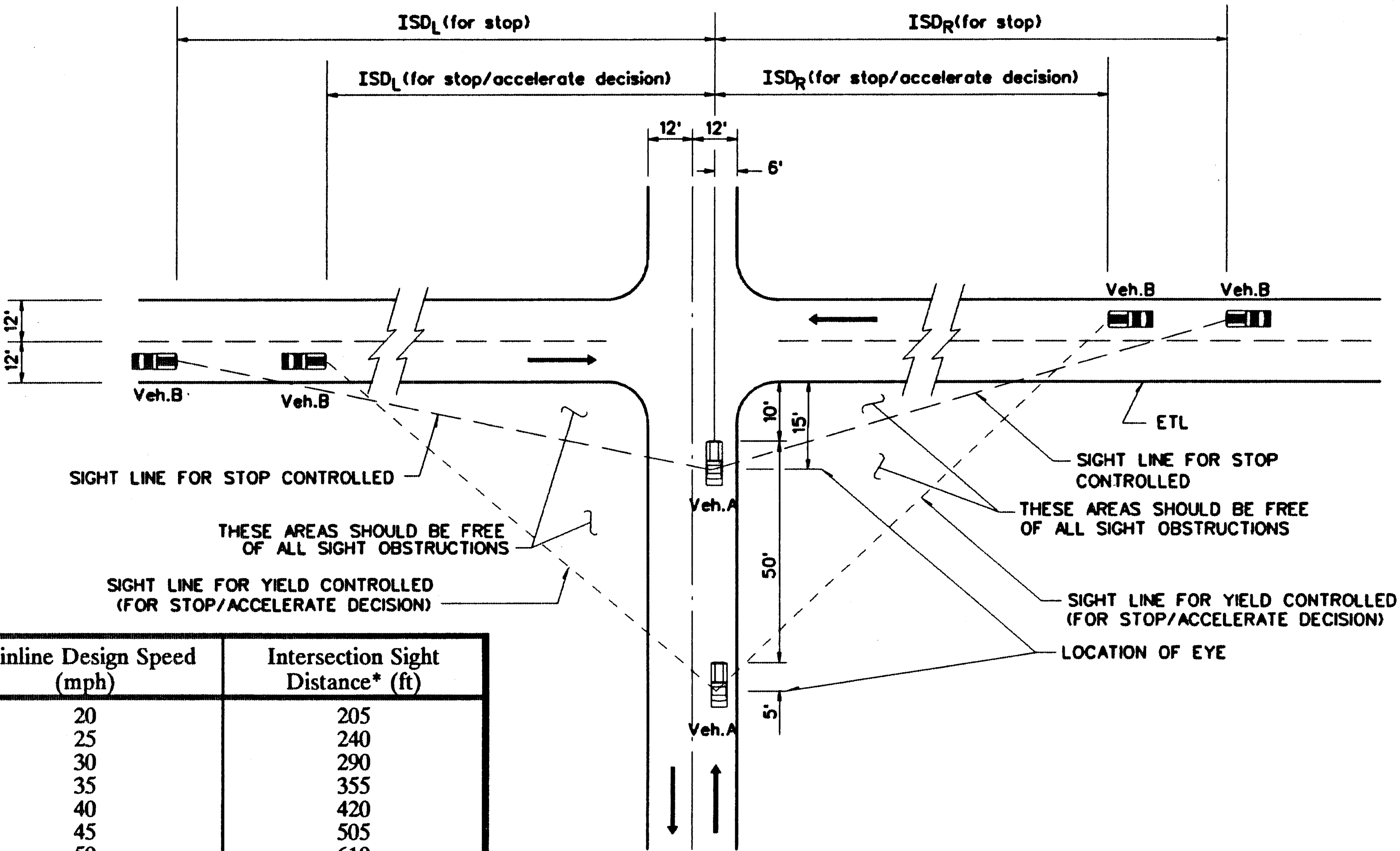
MINIMUM INTERSECTION SIGHT DISTANCES
(For Stop-Controlled Intersections)

Mainline Design Speed (mph)	Passenger Cars (ft)	Trucks (ft)
20	235*	355*
25	295*	440*
30	355	530*
35	415	620*
40	470	705
45	530	795
50	590	880
55	645	970
60	705	1060
65	765	1145
70	825	1235

* These values exceed the desirable ISD criteria in Table 9.2B. Use lower value for minimum design.

Notes:

1. ISD criteria for passenger cars are based on 8 seconds of travel time at the design speed.
2. ISD criteria for trucks are based on 12 seconds of travel time at the design speed.
3. See Figure 9.2B for specific application of ISD to intersection.



Mainline Design Speed (mph)	Intersection Sight Distance* (ft)
20	205
25	240
30	290
35	355
40	420
45	505
50	610
55	720
60	835
65	970
70	1125

* These are the ISD criteria for the stop/accelerate decision point (see Section 9.2.4). See Sections 9.2.2 and 9.2.3 for ISD criteria for stop condition.

INTERSECTION SIGHT DISTANCE AT A YIELD-CONTROLLED INTERSECTION (Passenger Cars)

Figure 9.2D

6. Mainline Vehicle (MV). The MV decelerates from the mainline design speed to 85% of this speed.

7. Following Distance (FD). The FD is based on providing two seconds of travel time at the design speed between the MV and EV.

8. Height of Eye/Object. The height of eye is 3.5 ft and the height of object (an approaching passenger car) is 4.25 ft.

These criteria will define the legs of the sight triangle for a yield-controlled intersection where the driver on the minor road chooses to continue (i.e., he does not stop). The ISD must also be checked assuming the driver stops at the intersection. The ISD criteria in Sections 9.2.2 or 9.2.3 will apply for this evaluation.

9.2.5 Stopped Vehicle Turning Left

At all intersections on a major road (undivided or narrow median) 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. The driver will need to see straight ahead a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection. Figure 9.2E illustrates the ISD theoretical assumptions for a stopped vehicle turning left. Table 9.7D presents the ISD values, which are based on the following equation:

$ISD = 1.467\ V(t+2)$, where:

V = design speed, mph

t = time, seconds, for traversing the distance "d"

d = distance traveled by the turning vehicle (MTV) to clear the outer travel lane of opposing traffic, ft (for passenger cars, $d = 35' + A$ and for trucks, $d = 90' + A$)

A = distance between center of lane from where MTV turns to edge of outer travel lane of opposing traffic

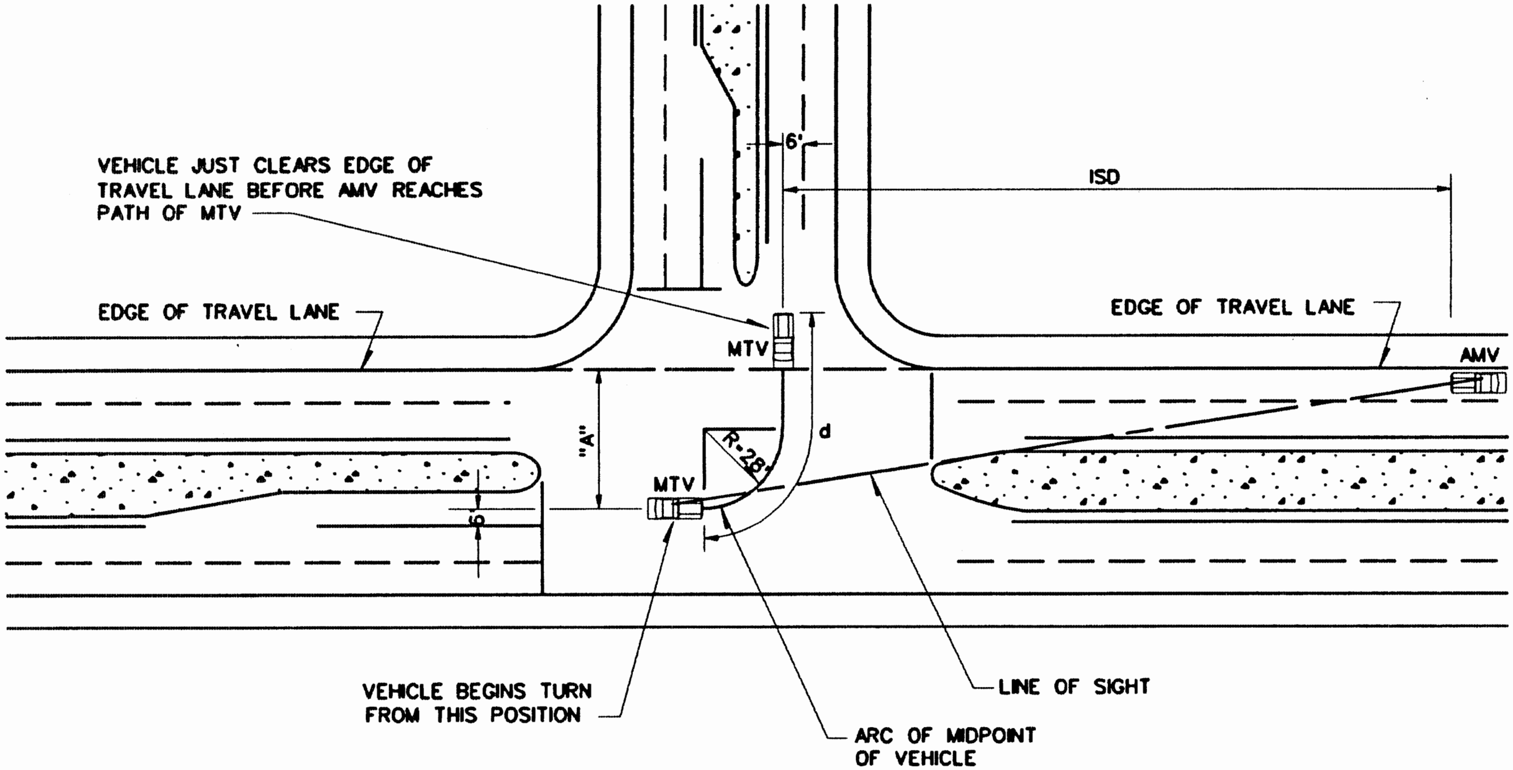
The following summarizes the major assumptions within the ISD model:

1. Design Vehicle. This will be either a passenger car (P) or, if there is a warranting number of trucks making the left turn, a truck should be considered as the design vehicle.

2. Mainline Turning Vehicle (MTV) Action. The MTV will move forward beyond the stop line, always remaining in line with the lane from which it will turn. It will stop when its front bumper is a distance equal to the turning radius away from the center of the lane into which it will turn. When it begins its turning maneuver, the MTV will turn at this radius until it is lined up with the lane on the crossroad and then travel in a straight line to complete the clearing maneuver. The MTV will always turn into the inside through travel lane.

3. Approaching Mainline Vehicle (AMV) Action. The AMV will travel at the design speed (V) and will maintain this speed through the intersection (i.e., it will not slow down).

4. Turning Radius. This will be 28 ft for the mid-section of the MTV, assuming a P design vehicle. This radius will be constant throughout the turn.



Notes:
1. See Table 9.2D for ISD values.
2. See Section 9.2.5 for discussion and application.

INTERSECTION SIGHT DISTANCE FOR A STOPPED VEHICLE TURNING LEFT

Figure 9.2E

Table 9.2D

INTERSECTION SIGHT DISTANCE

(Passenger Car Turning-Left from Major Road)

"A" (ft)	Design Speed of Major Road								
	20	25	30	35	40	45	50	55	60
18	195	245	295	340	390	440	485	535	585
20	200	245	295	345	395	445	495	540	590
25	205	255	305	355	405	455	505	560	610
30	210	260	315	365	420	470	520	575	625
35	215	270	320	375	430	480	535	590	645
40	220	275	330	385	440	495	550	605	660
45	225	285	340	395	450	510	565	620	675
50	235	290	350	405	465	520	580	635	695
55	240	295	355	415	475	535	595	650	710
60	240	300	360	420	480	540	600	660	720

(WB-67 Turning-Left from Major Road)

"A"	Design Speed of Major Road								
	20	25	30	35	40	45	50	55	60
18	430	535	640	750	855	960	1070	1175	1280
20	430	535	645	750	860	965	1075	1180	1285
25	435	545	650	760	870	975	1085	1195	1300
30	440	550	660	770	880	985	1095	1205	1315
35	450	560	675	785	895	1010	1120	1230	1345
40	460	575	690	805	920	1030	1145	1260	1375
45	470	585	705	820	940	1055	1175	1290	1405
50	480	600	720	840	960	1080	1200	1320	1440
55	490	615	735	860	980	1105	1225	1350	1470
60	500	625	750	875	1000	1125	1250	1375	1500

Note: See Figure 9.2E for application of ISD criteria.

5. Acceleration Rate. The MTV will accelerate at the rates in Table 9.2A.

6. Reaction Time of MTV. The model assumes two seconds for the turning driver to release the brake and depress the accelerator.

7. Clearance Interval. The clearance between the MTV and the AMV is assumed to be zero.

8. Effect of Median Width. This represents additional distance the MTV must traverse. The model assumes the MTV will not move laterally to the left within the intersection area, even if the opportunity is available.

9. Effect of Exclusive Left-Turn Lane. If one is present, the MTV will turn from it; if one is not present, the MTV will turn from the inner through lane. If a dual left-turn lane is present, the MTV will turn from the outer left-turn lane, which will yield a greater ISD need.

For application, the ISD criteria for a stopped vehicle turning left will apply to all opportunities for this maneuver at all intersections. The following procedure should be used:

10. Find the ISD value from Table 9.2D for the applicable design speed and "A" value at the intersection.

11. Use the ISD value to locate the front bumper of the AMV as indicated in Figure 9.2D. This will represent the "object" location.

12. Find the "eye" location as indicated in Figure 9.2D. The eye will be 10 ft behind the bumper of the MTV.

13. The line of sight between the eye and object should be clear of all obstacles. The height of eye is 3.5 ft for passenger cars and 8 ft for trucks. The height of object is 4.25 ft.

9.2.6 Signal-Controlled Intersections

The minimum ISD requirements in Table 9.2B or 9.2C will generally apply to a signalized intersection. This is reasonable because of the increased driver workload at intersections and the potential conflicts involved when vehicles turn onto or cross the highway. These include:

1. violation of the signal,

2. right turns on red,

3. signal malfunction, and/or

4. use of flashing yellow/red mode during part of the day.

Another sight distance consideration at signalized intersections is the visibility of the traffic control device. At signalized rural intersections, for example, this desirably should be decision sight distance. See Chapter Five.

The ISD criteria in Table 9.2B or 9.2C have a significant application for vehicles making a right turn on red. If these criteria cannot be met, the Traffic Engineering Division may recommend prohibiting right turn on red at the intersection.

9.2.7 Obstruction-Free Sight Triangle

The designer should use the criteria in Section 9.2 to establish the sight triangle which should be clear of any obstructions impairing the driver's line of sight. The designer should note that not every item within the sight triangle is necessarily a sight obstruction. Obstacles of limited dimension may not block the required line of sight (e.g., utility poles, mailboxes). The identification of sight obstructions should be made on a site-by-site basis.

The designer should consider potential sight obstructions which are not always present. For example, parked cars may become obstructions if within the sight triangle. This may lead the designer to seek a prohibition against parking within the intersection sight triangle.

Once the sight triangles have been defined, it is important that these areas remain free of obstructions in the future. Therefore, it is desirable to secure the needed right-of-way and/or property easements to ensure the future availability of the area. The designer should coordinate with the Right-of-Way Division to achieve this objective.

9.3 AUXILIARY LANES

When the turning maneuver for left- and right-turning vehicles occurs in the through travel lanes, it disrupts the flow of through traffic. To minimize potential conflicts or increase capacity, the use of auxiliary lanes may be warranted for at-grade intersections to improve the level of service and safety at the intersection.

9.3.1 Warrants for Right-Turn Lanes

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

1. at any unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria in Figure 9.3A;
2. at any intersection where a capacity analysis determines a right-turn lane is necessary to meet the level-of-service criteria;
3. as a general rule, at any signalized intersection where the right-turning volume is greater than 300 vph and where there is greater than 300 vphpl on the mainline; or
4. at any intersection where the accident experience, existing traffic operations or engineering judgment indicates that a right-turn lane will significantly improve operations.

9.3.2 Warrants for Left-Turn Lanes

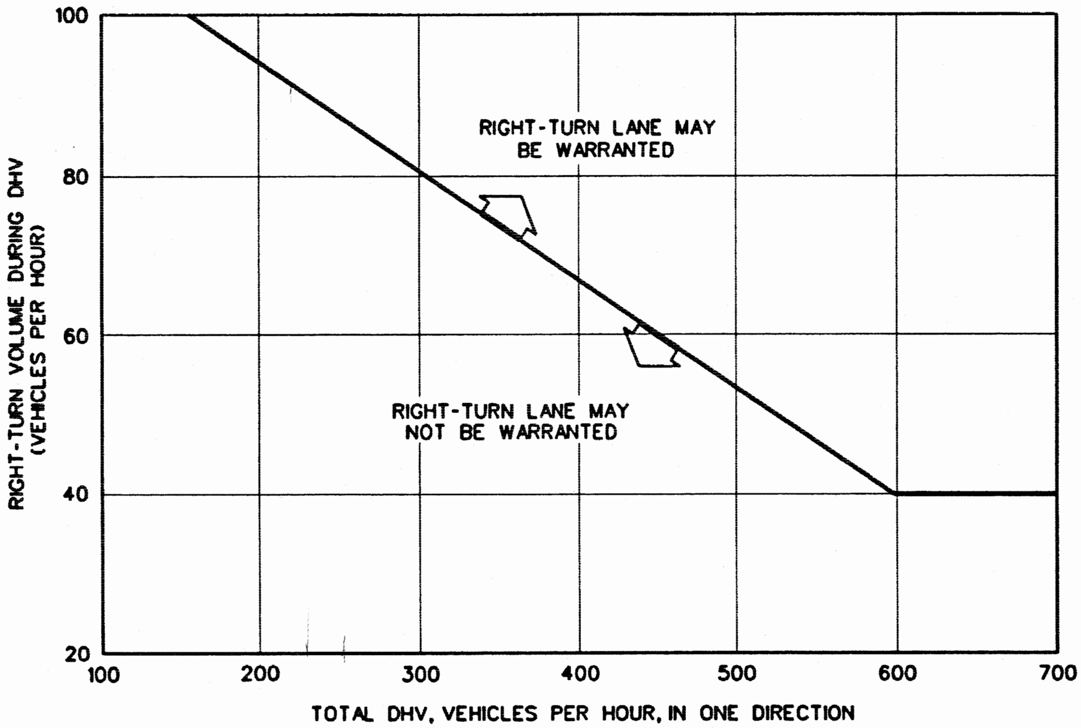
The accommodation of left turns is often the critical factor in proper intersection design. Left-turn lanes can significantly improve both the level of service and intersection safety. Exclusive left-turn lanes should be considered:

1. at all free-flowing approaches on principal, high-speed rural highway intersections with other arterials or collectors;
2. at intersections on divided urban and rural highways with a median wide enough to accommodate a left-turn lane, regardless of traffic volumes;
3. at any unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria in Figures 9.3B, C or D;
4. at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria;
5. as a general rule, at any signalized intersection where the left-turning volume is 100-150 vph (for a single turn lane) or 300 vph (for a dual turn lane); or
6. at any intersection where the accident experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates that a left-turn lane will significantly improve operations.

9.3.3 Design of Auxiliary Turn Lanes

The following basic criteria will apply to the design of auxiliary turn lanes:

1. Length. Section 9.3.3.1 presents the criteria for determining the length of a turn lane.



Notes: For highways with a design speed below 50 mph *and* 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 = 40 mph
DHV = 250 vph
Right Turns = 100 vph

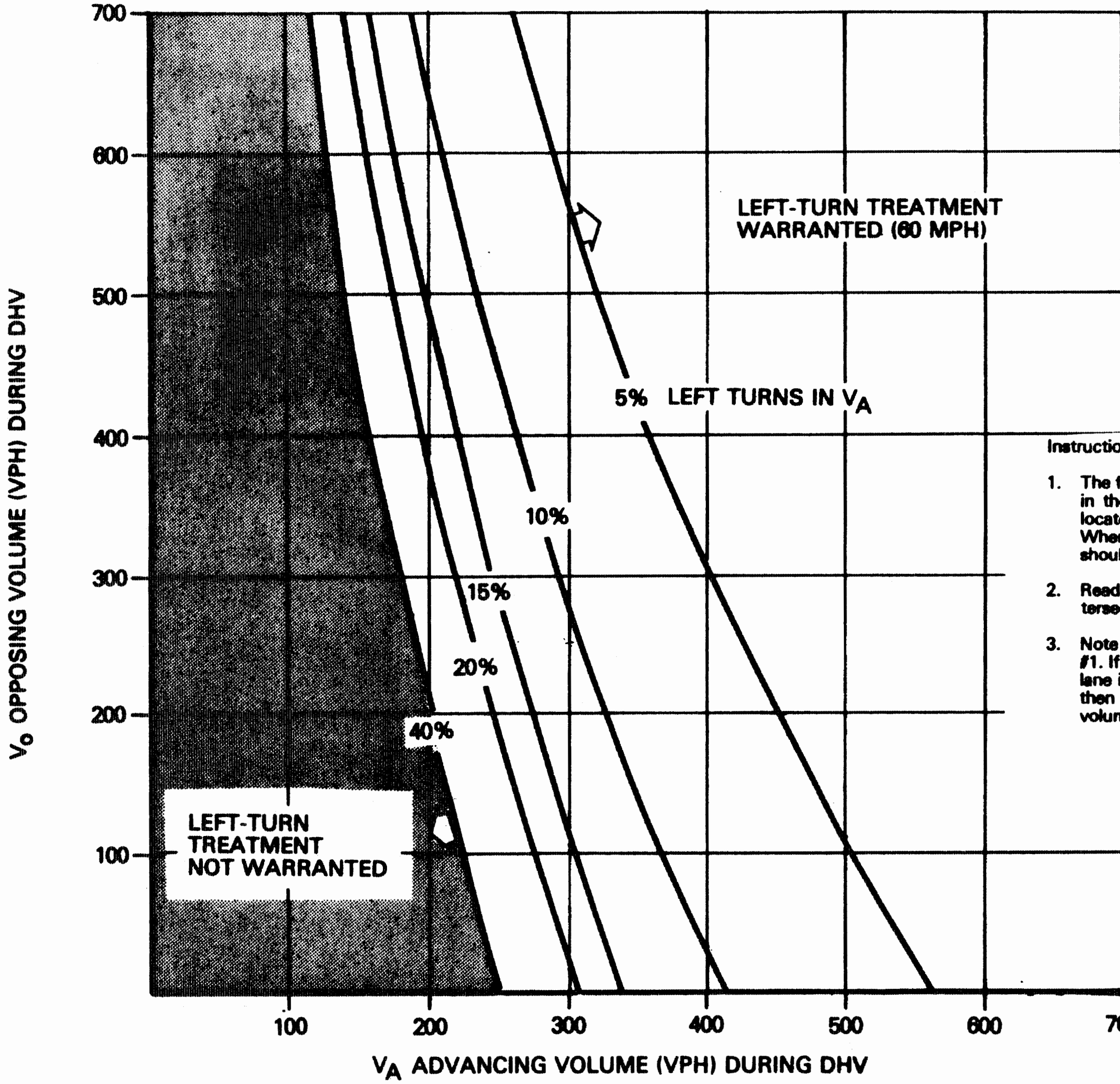
Problem: Determine if a right-turn lane is warranted.

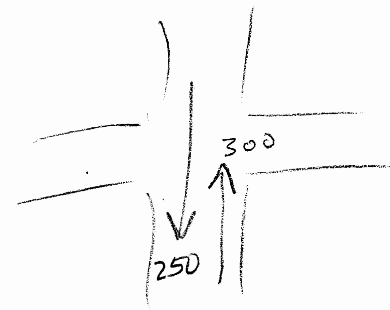
Solution: To read the vertical axis, use 100 - 20 = 80 vph. The figure indicates that an exclusive right-turn lane is not warranted, unless other factors (e.g., high-accident rate) indicate a lane is needed.

Source: (5)

GUIDELINES FOR RIGHT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS

Figure 9.3A





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.

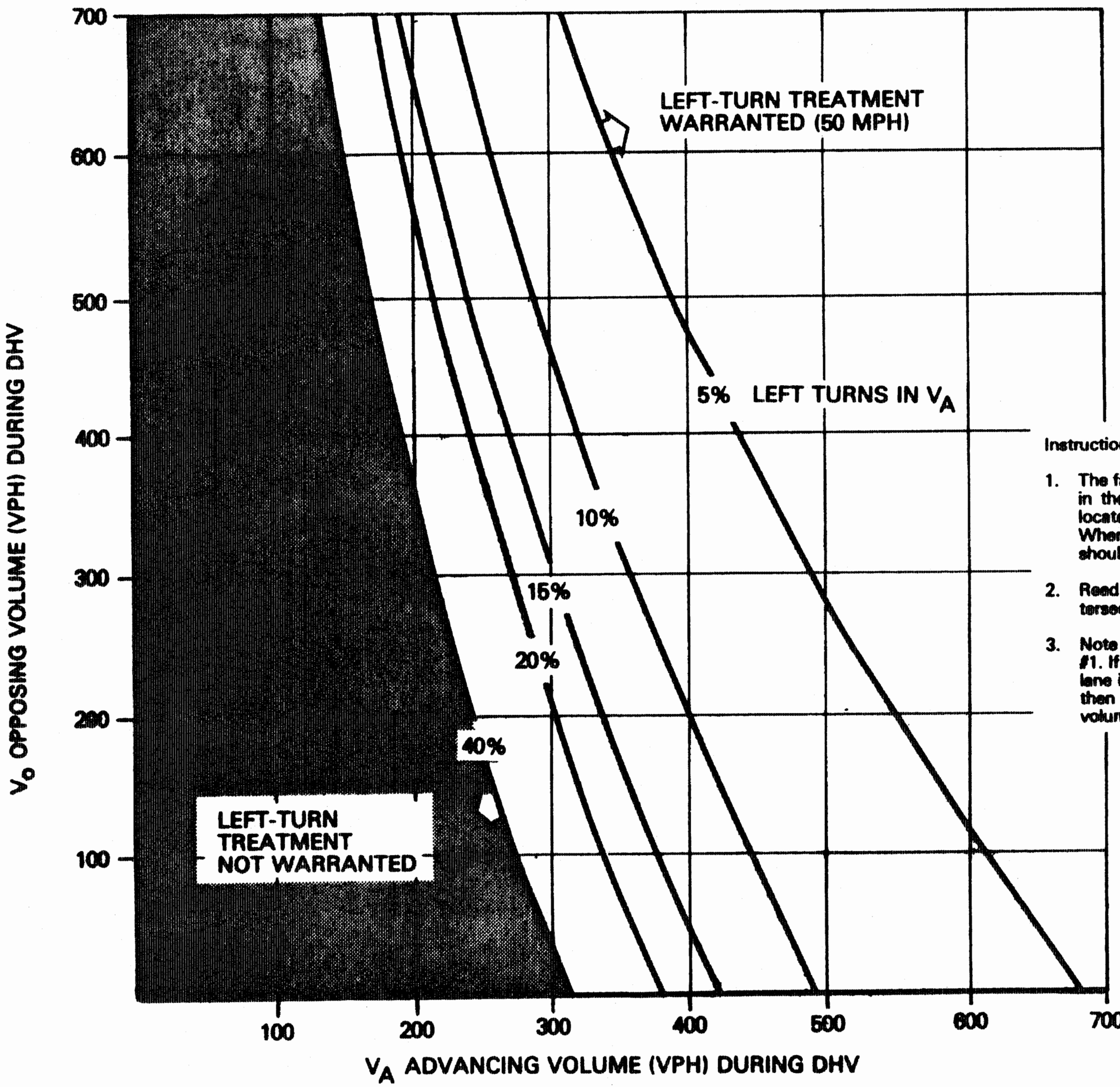
2. Read V_A and V_O into the chart and locate the intersection of the two volumes.

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 is not warranted based on traffic volumes.

Source: (9)

VOLUME WARRANTS FOR LEFT-TURN LANE AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (60 MPH)

Figure 9.3B



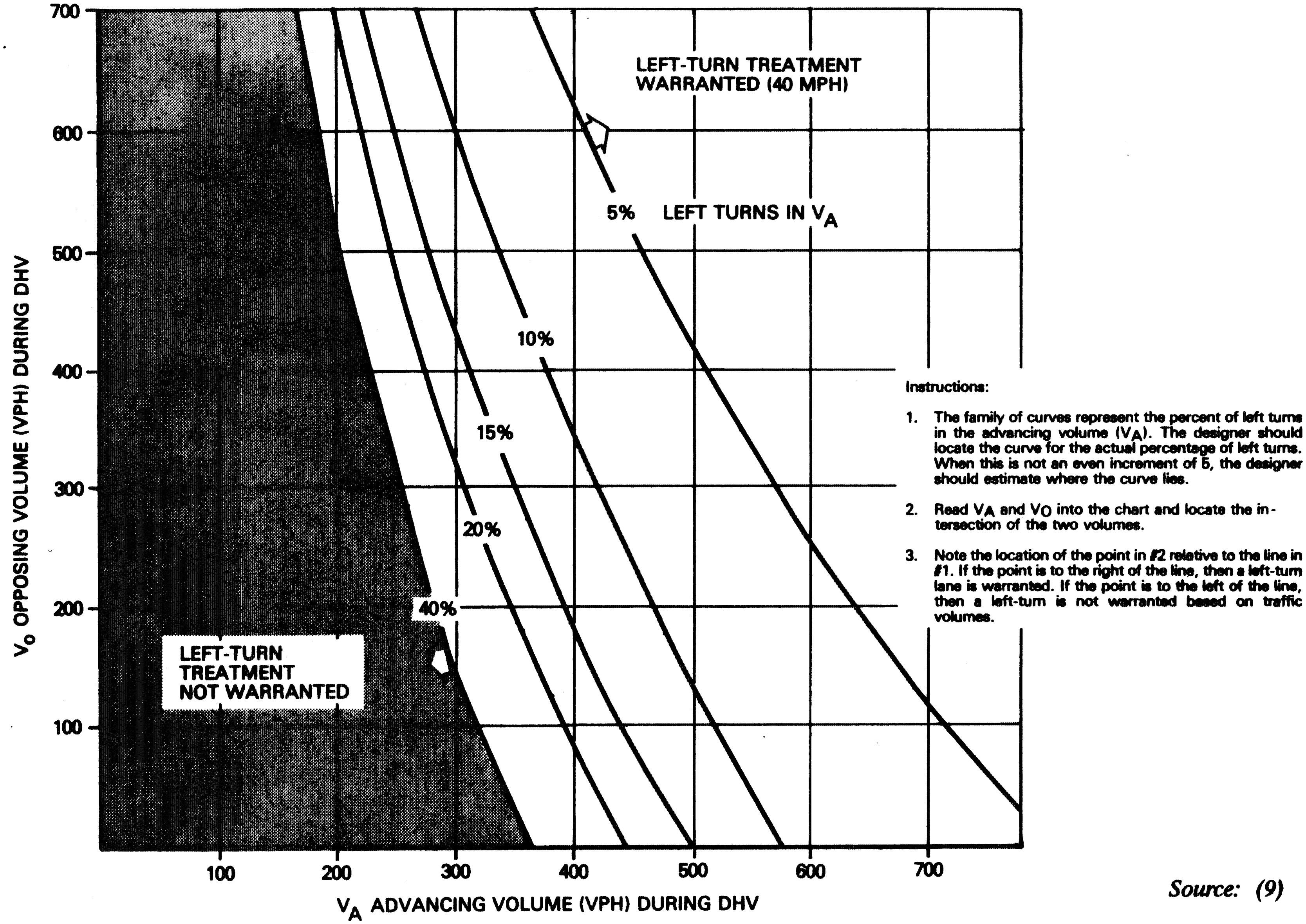
Instructions:

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 is not warranted based on traffic volumes.

Source: (9)

VOLUME WARRANTS FOR LEFT-TURN LANE AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (50 MPH)

Figure 9.3C



Source: (9)

VOLUME WARRANTS FOR LEFT-TURN LANE AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (40 MPH)

Figure 9.3D

2. Width. The width of the turn lane should be determined relative to the functional class, urban/rural location and project scope of work. Chapters Twelve and Thirteen present the applicable widths for auxiliary lanes; they also provide the criteria for the applicable shoulder widths adjacent to auxiliary lanes.

3. DHV. The volume of vehicles during the design hour (DHV) will impact several design elements for turn lanes (e.g., warrant, length). See Chapter Five for a discussion on traffic volumes. Chapters Twelve and Thirteen present applicable criteria for selection of a future design year.

9.3.3.1 Length of Auxiliary Turn Lanes

Desirably, the length of a right- or left-turn lane at an intersection should allow both safe vehicular deceleration and storage of turning vehicles outside of the through lanes. This improves safety and the intersection level of service. However, it is often not practical to provide a turn lane length which provides for deceleration. Therefore, in many cases, the full-width length will only be sufficient for storage.

The length of auxiliary turn lanes will be determined by some combination of its taper length (L_T), deceleration length (L_D) and storage length (L_S) and by the mainline functional classification. Table 9.3A presents the length considerations for the various speeds and traffic controls. Figure 9.3E illustrates a schematic of auxiliary lanes at an intersection. The following will apply:

1. Taper. ODOT typically uses the straight taper for entrance into a turn lane. Taper lengths (L_T) should be as long and as smooth as practical so that drivers will be encouraged to use the full length of the taper. Table 9.3B provides recommended taper rates for various design speeds.

2. Deceleration. Desirably, all vehicular deceleration will occur within the taper and full width of the turn lane; however, this is often impractical. Consequently, some or all deceleration may occur prior to the beginning of the taper. Table 9.3C provides various distances (L_D) for different speed reductions. The table allows the designer to determine the benefits and consequences of a given auxiliary turn lane length.

pg. 9.3(9)

3. Storage (Signalized Intersections). The storage length (L_S) for turn lanes should be sufficient to store the number of vehicles likely to accumulate in the design hour. Note that traffic volumes are based on the future design year as indicated in the geometric design tables in Chapters Twelve and Thirteen. The recommended storage length criteria for signalized intersections follow:

   a. Figure 9.3F illustrates the method to determine the recommended storage length for turn lanes at a signalized intersection when the *v/c* ratio is known. The figure applies directly to all left-turn lanes and to right-turn lanes where there are no right turns on red. Where right turns on red are allowed, L_S will be determined from Figure 9.3F minus the number of right turns on red during the DHV.

   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 9.3F. The *v/c* ratio is determined by a ca-

Table 9.3A

FUNCTIONAL LENGTHS OF AUXILIARY TURN LANES

Type of Traffic Control	Highway Design Speed (mph)		
	50 - 60	40 - 45	30 - 35
Traffic Signal*	$L_D + L_S$	$L_D + L_S$	L_S
Stop Control (Stop Approach)	L_S	L_S	L_S
Stop Control (Free-flowing Approach)	High Vol: $L_D + L_S$ Low Vol: L_S	High Vol: $L_D + L_S$ Low Vol: L_S	L_S

Source: (4) Revised

* At signalized T-intersections, the functional length of turn lanes on the truncated leg is L_S.

L_T = Length of Bay Taper
L_D = Length of Deceleration (Full or Partial) (Table 9.3C)
L_S = Length of Storage

Note: See Figure 9.3E for a definition of terms.

Table 9.3B

RECOMMENDED BAY TAPER RATES

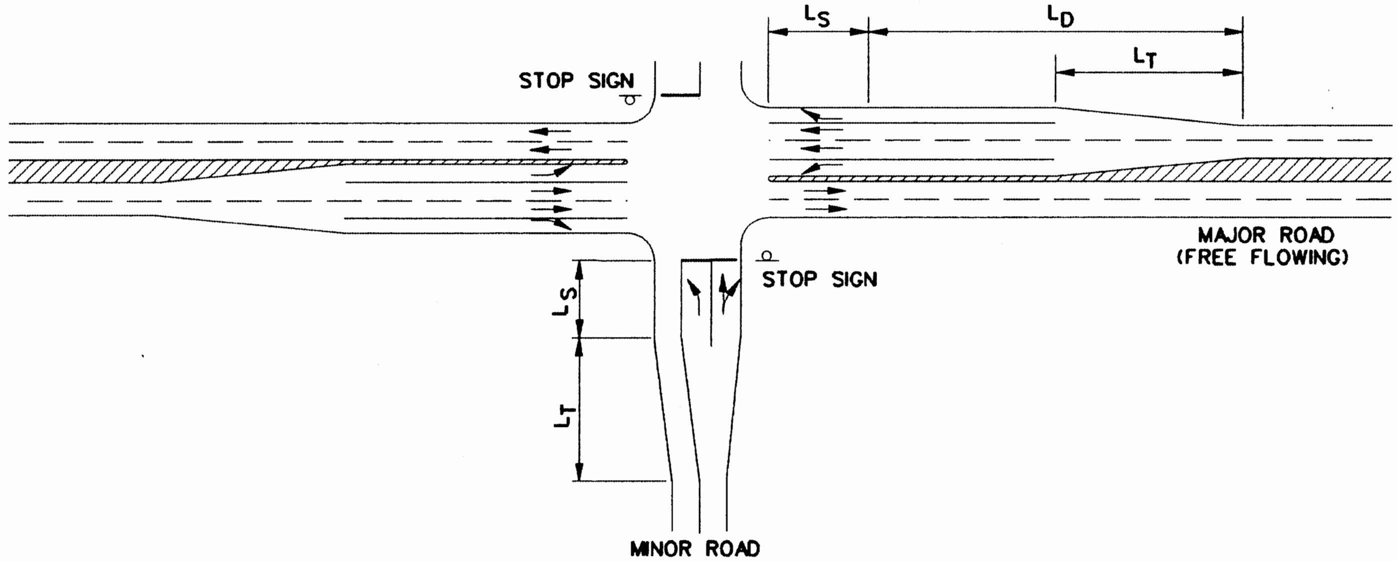
Design Speed (mph)	Taper Rate
$V \leq 30$	8:1
$30 < V < 50$	10:1
$50 \geq V$	15:1

Source: (1) Revised

The following minimum values may apply in restricted locations:

1. Right-Turn Lanes. A 4:1 bay taper may be used where painted channelization is used.

2. Left-Turn Lanes. In severely restricted locations, a 4:1 bay taper may be used where painted channelization is used.



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 (full or partial)
- L_S = Storage length

FUNCTIONAL LENGTH OF AUXILIARY TURN LANES
(Typical Flared Intersection)

Figure 9.3E

Table 9.3C

DECELERATION DISTANCES FOR TURN LANES

Design Speed (mph)	Average Running Speed* (mph)	Vehicular Speed @ Beginning of Taper (mph)	L_D** (Taper plus Full Width Auxiliary Lane) (ft)
70	58	58 50 40 30 20 10	615 355 225 130 60 35
60	52	52 50 40 30 20 10	530 455 230 100 55 20
50	44	44 40 30 20 10	435 260 150 65 20
40	36	36 30 20 10	315 160 65 20
30	28	28 20 10	235 85 35

* Average running speed assumed for calculations.

** This is the distance needed to allow the vehicle to reduce speed from "speed @ beginning of taper" to zero.

Notes:

1. This table was developed from criteria in Reference 16.

2. This table allows the designer to evaluate the consequences of a given length of turn lane. For example, if the mainline design speed is 50 mph and the length of turn lane plus taper is 150 ft (not including storage length), then the vehicle will decelerate from its travel speed on the mainline to 30 mph in the through travel lane. Whether or not this is reasonable will depend upon the through volumes, turning volumes, available space, construction costs, etc.

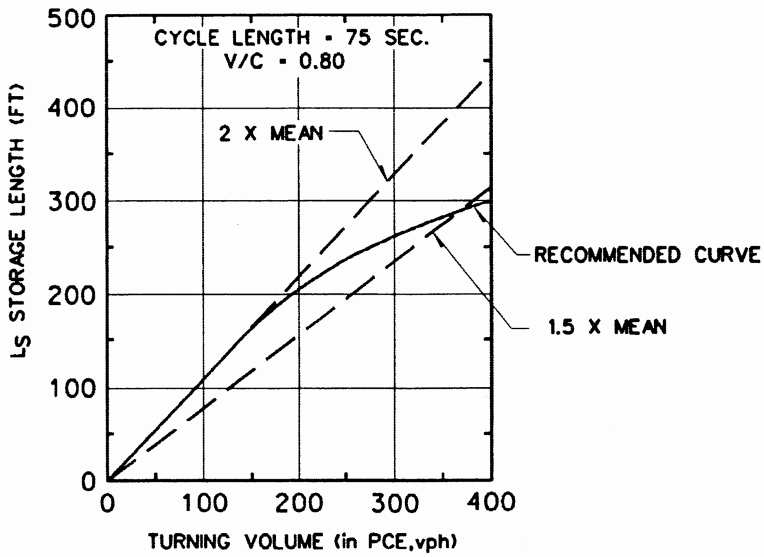
EXAMPLE:

Cycle: 90 secs
v/c : 0.85
Lt-turn Vol: 100 vph

Permitted Phase
Opposing Vol: 700 vph

SOLUTION:

PCE's = 100 x 3 = 300 vph
L_s = 260' (for v/c = 0.80 and Cycle = 75 secs)
L_s Adjustment Factor = 1.24'
L_s = (1.24)260 = 322'
L_s = 325' (for v/c = 0.85 and Cycle = 90 secs)



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

Passenger Car Equivalent (PCE's) (Left-Turn Lanes Only)

Type of Turn	Opposing Volume (vph)	Passenger Car Equivalent (PCE)
Protected	—	1.05
Permitted	0 to 199	1.1
	200 to 599	2.0
	600 to 799	3.0
	800 to 999	4.0
	≥ 1000	5.0

Source: (8) Revised

Notes:
1. Use this method when *v/c* is known.
2. Figure applies to exclusive left-turn lanes and exclusive right-turn lanes where there are no right turns on red. Where right turns on red occur, subtract their number from the right-turn volume before using figure.
3. See minimum storage length discussion in Section 9.3.3.1.
4. The values obtained from the graph at the top of this page are for a cycle length of 75 sec and a *v/c* ratio of 0.80. For other values of the *v/c* ratio and any other cycle length, the length of storage obtained from the graph should be multiplied by the storage length adjustment factor.

RECOMMENDED STORAGE LENGTH FOR SIGNALIZED INTERSECTIONS

Figure 9.3F

pacity analysis as described in the *Highway Capacity Manual*.

pg 9.3(10)

Figure 9.3F presents a recommended curve to determine L_S. This curve should be used regardless of cycle length. The curves labeled "1.5 x Mean" and "2 x Mean" are presented for informational purposes.

pg. 9.3(12)

b. Figure 9.3G illustrates a method to determine the recommended storage length for turning lanes at a signalized intersection when the *v/c* ratio is not known or cannot be calculated. The storage length should be based on the cycle length and the traffic volumes during the design hour. For a 90-second cycle or less, the storage length should be based on 2 times the average number of vehicles that would store per cycle during the design hour (the desirable value). For cycle lengths mores than 90 seconds, the storage length should be based on 1.5 times the average number of vehicles that would store per cycle during the design hour (the minimum value).

c. The minimum turn lane length is 125 ft where the selected design vehicle is the WB-114 and 100 ft where the design vehicle is the WB-67. At other intersections, the minimum turn lane length is 50 ft on low-speed facilities (V≤45 mph) and/or where the turn lane is preceded by painted channelization. The minimum turn lane length for all other intersections is 100 ft.

d. The designer should ensure that the turn lane length exceeds the storage length of the adjacent through lane. Otherwise, a vehicular queue in the through lane will block entry into the turn lane by turning vehicles. L_S for the turn lane is calculated by the same method as that used for a turn lane.

4. Storage Length (Unsignalized Intersections). The storage length should be the number of turning vehicles likely to arrive in an average 2-minute period within the design hour. The following provides recommended storage lengths for right- and left-turn lanes at an unsignalized intersection, assuming 25 ft storage length per vehicle:

$$L_S = \frac{DHV}{30\ PHF} \ x\ 25 = 0.833\ DHV/PHF$$

Where:

L_S = storage length, ft

DHV = design hourly volume for turns, vph

PHF = peak-hour factor

For example, if the turning volume DHV = 100 vph and PHF = 0.85, then:

L_S = 0.833(100)/0.85

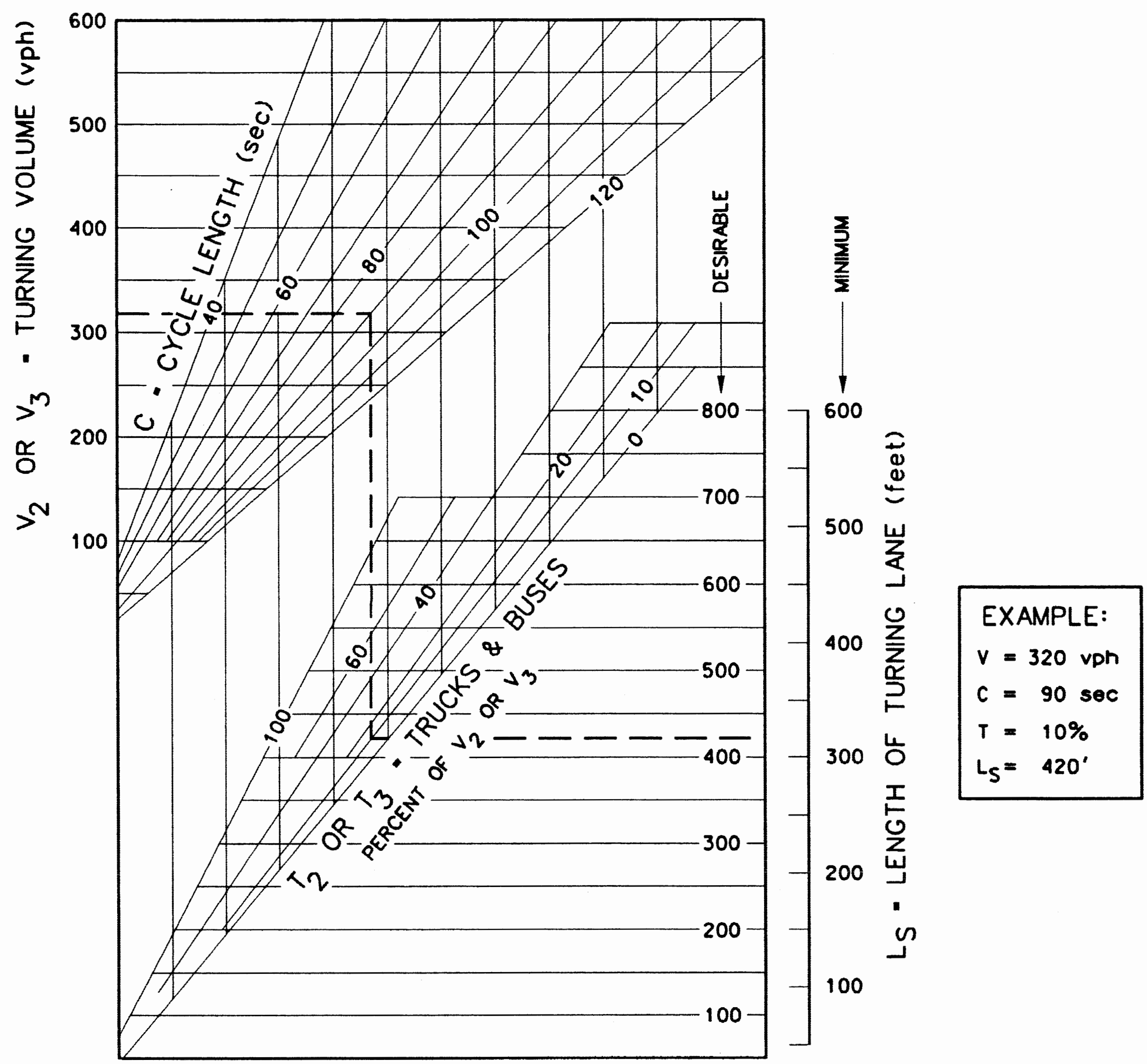
L_S = 98′, subject to minimum storage requirements

The minimum storage requirements for turn lanes at unsignalized intersections is the same as that for signalized intersections (see Comment #3c).

9.3.3.2 Typical Treatments for Auxiliary Turn Lanes

The following presents typical treatments for right- and left-turn lanes:

1. Right-Turn Lanes. Figure 9.3H illustrates the typical development of an exclusive right-turn lane.



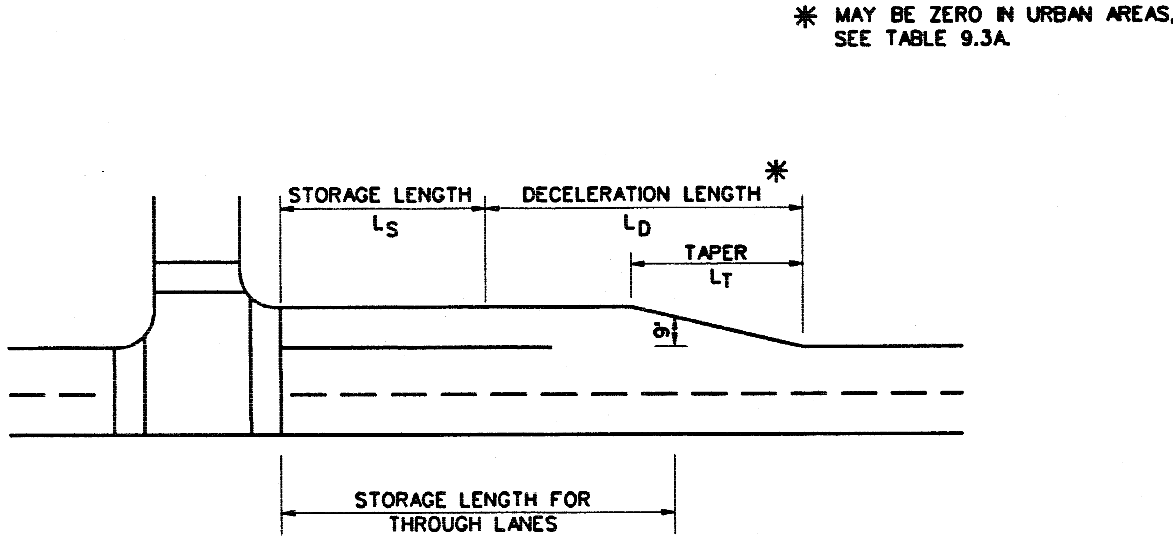
Notes:

1. Use this graph when *v/c* is not known.
2. The "desirable" L_S is based on 2 times the average rate of arrival per cycle during the DHV; the "minimum" L_S is based on 1.5 times the average rate of arrival per cycle during the DHV.
3. Storage lengths are based on a 25′ length for passenger cars and a 40′ length for trucks.

Source: (4)

RECOMMENDED STORAGE LENGTH (L_S) FOR SIGNALIZED INTERSECTIONS

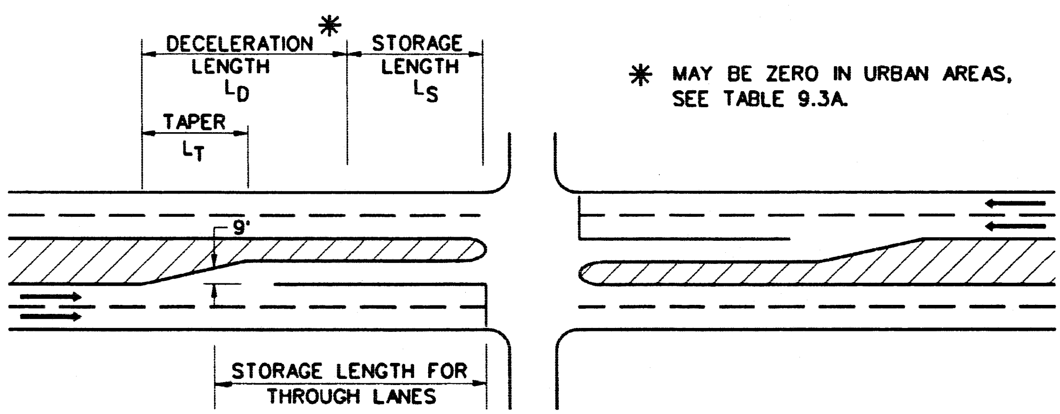
Figure 9.3G



Note: See Section 9.3 for criteria on taper length, deceleration length and storage length.

TYPICAL RIGHT-TURN LANE
(Stop or Signal Control)

Figure 9.3H



Note: See Section 9.3 for criteria on taper length, deceleration length and storage length.

TYPICAL LEFT-TURN LANE

Figure 9.3I

2. Channelized Left-Turn Lanes. On divided highways, the design presented in Figure 9.3I will apply to the development of an exclusive left-turn lane in the median. Figure 9.3J illustrates a desirable approach to develop a channelized left-turn on an undivided highway.

9.3.4 Extension of Additional Through Lanes

To meet the level-of-service criteria, it may be necessary to add through lanes approaching the intersection. However, these additional lanes should be extended beyond the intersection to fully realize the capacity benefits. Figure 9.3K provides criteria for determining how far these lanes should be extended beyond the intersection.

The recommended minimum full-width lengths of the through lane extension (D_E) are those distances needed for the stopped vehicle to accelerate to 5 mph below the average running speed of the highway. Desirably, the full-width lengths of the extension will be the stored vehicle length which will cross the intersection during a green cycle.

The distances in Figure 9.3K may or may not be sufficient for the vehicle to merge into the "primary" through lane. Therefore, the criteria in Figure 9.3K should be used for preliminary design purposes. For final design and capacity analyses, the designer should coordinate with the Traffic Engineering Division and the Urban Design Division, Geometric Design Branch to determine the final length.

The desirable taper length at the end of the additional through lane will be based on the following formulas:

$L = WV$, where the design speed is ≥ 45 mph, or

$L = WV^2/60$, where the design speed is ≤ 40 mph.

where:

L = lane taper length
W = width of added lane
V = design speed

At a minimum, a 200 ft - 300 ft taper should be used at the end of the additional through lane.

9.3.5 Dual Turn Lanes

9.3.5.1 Warrants

Dual right- and/or left-turn lanes should be considered when:

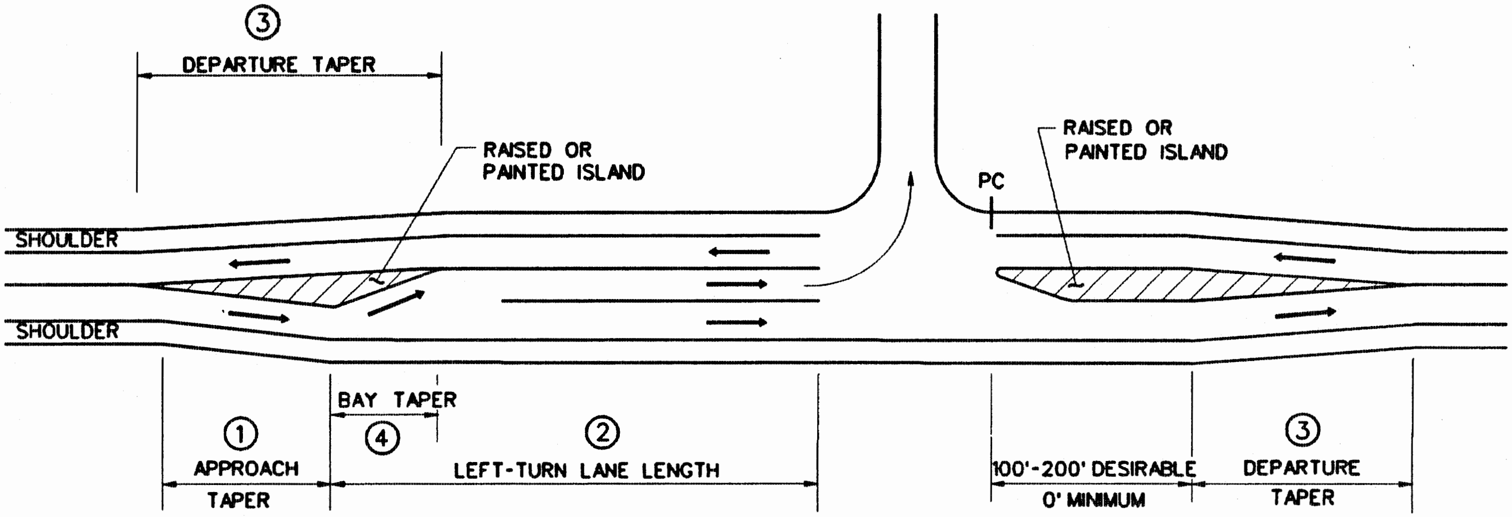
1. there is insufficient space to provide the necessary length of a single turn lane because of restrictive site conditions (e.g., closely spaced intersections);

2. the necessary length of a single turn lane becomes prohibitive;

3. as a general rule, there are 300 or more left-turning vehicles in the design hour; and/or

4. the necessary time for a protected left-turn phase for a single lane becomes unattainable to meet the level-of-service criteria.

Dual right-turn lanes do not work as well as dual left-turn lanes because of the more restrictive turning movements. If practical, the designer should find an alternative means to accommodate a high number of right-turning vehicles. For example, a turning roadway may accomplish this purpose.



① The preferred approach taper rate is V:1, where V is the design speed. For V≤40 mph, it is acceptable for the approach taper to be (V^2/60):1.

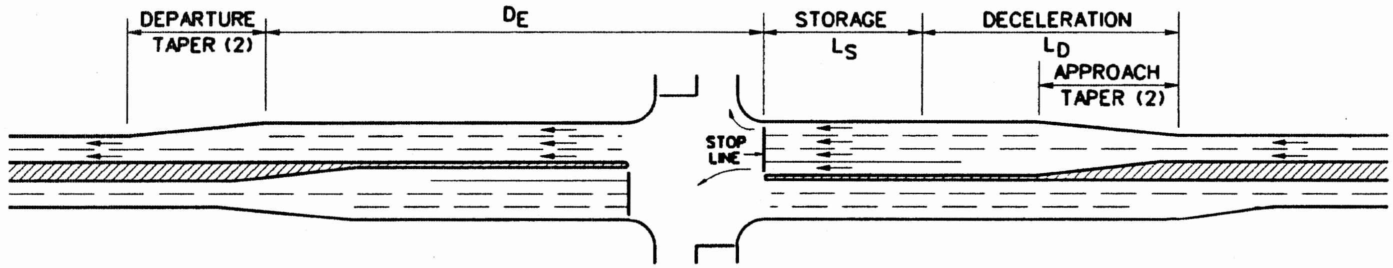
② See Section 9.3 for determination of left-turn lane length.

③ The preferred departure taper rate is V:1, where V is the design speed. For V≤40 mph, it is acceptable for the departure taper to be (V^2/60):1.

④ See Table 9.3B for bay taper rates.

CHANNELIZED LEFT-TURN LANE ON TWO-LANE HIGHWAY

Figure 9.3J



Design Speed (mph)	D_E (ft) (Minimum)
30	190
35	285
40	380
45	570
50	760
55	965
60	1170

Notes:

1. The minimum D_E is that distance required by the vehicle to accelerate from a stop to 5 mph below the average running speed.

2. The desirable taper distance is calculated from:

   $L = WV (V \geq 45)$ or $L = WV^2/60 (V \leq 40)$

   where: L = taper length, ft
   W = 12 ft
   V = design speed, mph

   The minimum departure taper is 200′ - 300′.

3. These criteria are for preliminary design purposes. See discussion in Section 9.3.4.

EXTENSION OF ADDITIONAL THROUGH LANES

Figure 9.3K

The lower efficiency of dual right-turn lanes, as compared to dual left-turn lanes, is reflected in the capacity methodology for signalized intersections in the *Highway Capacity Manual* (HCM). Specifically, the adjustment factor for dual left-turn lanes (f_{LT}) is 0.92; for dual right-turn lanes, f_{RT} = 0.75. See Chapter Nine of the HCM.

9.3.5.2 Design

For dual turn lanes to work properly, several design elements must be carefully evaluated. Figure 9.3L presents both dual right- and left-turn lanes to illustrate the more important design elements. The designer should consider the following:

1. Throat Width. Because of the off-tracking characteristics of turning vehicles, the normal width of two travel lanes may be inadequate to properly receive two vehicles turning simultaneously. The throat width will be determined by the application of the turning templates for the design vehicles (see Comment #4). The designer can expect that the receiving width for dual left-turn lanes will be approximately 30-36 ft. For dual right-turn lanes, a 36-ft throat width can be expected. When determining the available throat width, the designer can assume that a strengthened paved shoulder, if present at the receiving throat, can be used to accommodate dual turns.

2. Pavement Marking Guidelines. As illustrated in Figure 9.3L, pavement markings can effectively guide two lines of vehicles turning side-by-side. The Traffic Engineering Division will determine the selection and placement of any special pavement markings.

3. Opposing Left-Turn Traffic. If simultaneous, opposing dual left turns will be allowed, the designer should ensure that there is sufficient space for all turning movements. This is a factor at all signalized intersections, but dual left-turn lanes with their side-by-side vehicles can cause special problems. If space is unavailable, it may be necessary to alter the signal phasing to allow the two directions of traffic to move through the intersection on separate phases. Consequently, the intersection layout may need to be coordinated with the Traffic Engineering Division and Urban Design Division, Geometric Design Branch. As a recommendation, 30 ft should be available between opposing flows of traffic. See Figure 9.3L for the measurement of the 30-ft dimension.

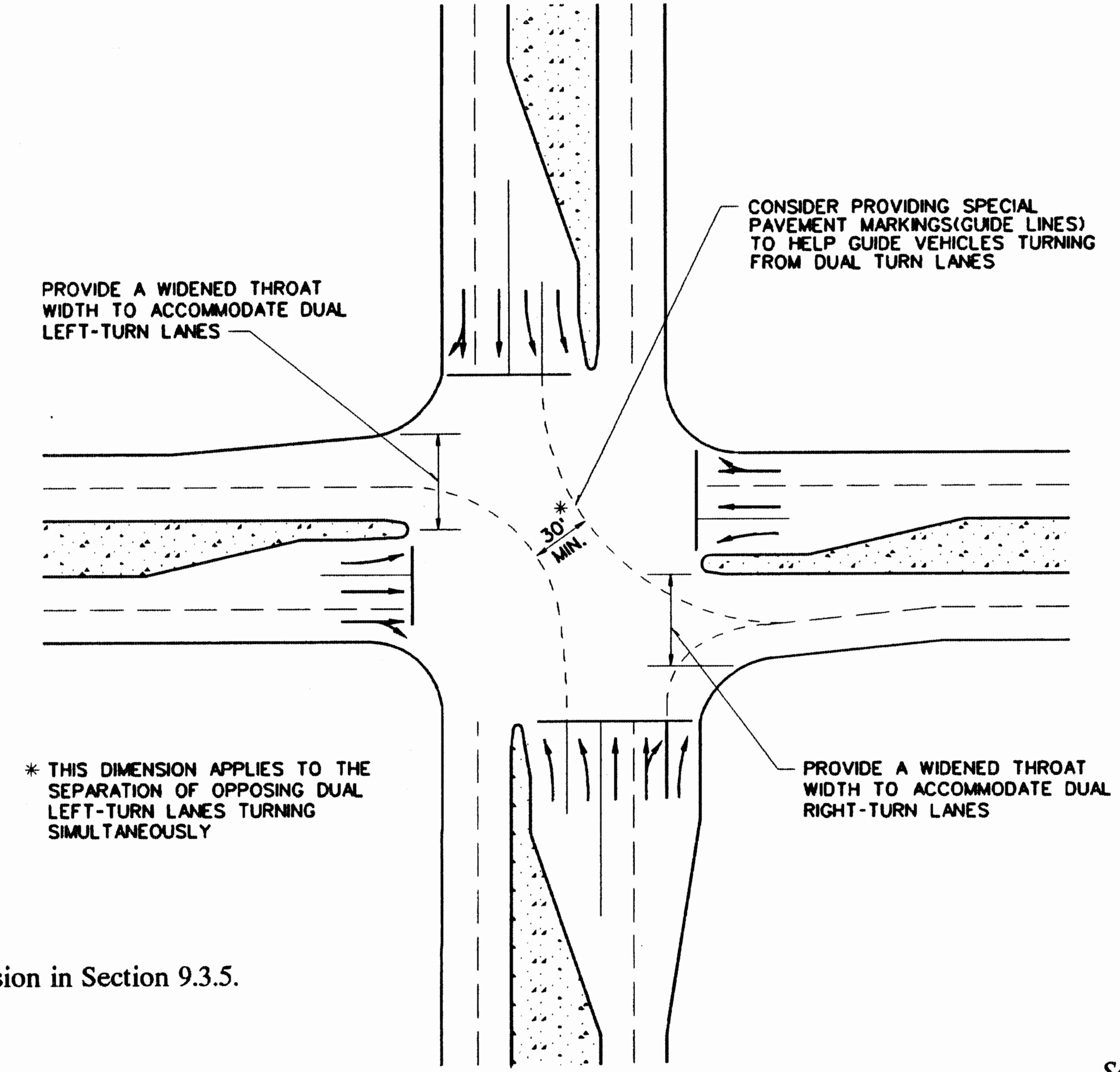
4. Turning Templates. All intersection design elements for dual turn lanes should be checked by using the applicable turning templates. The designer should assume that the selected design vehicle will turn from the outside lane of the dual turn lane. Desirably, the inside vehicle should be a SU but, as a minimum, the other vehicle can be assumed to be a P vehicle turning side-by-side with the selected design vehicle. Ultimately, the final design and selected design vehicles will be determined on a case-by-case basis.

9.3.6 Acceleration Lanes

It may be warranted to provide an acceleration lane for turning vehicles at an intersection to allow these vehicles to accelerate before merging with the through traffic.

9.3.6.1 Acceleration Lanes for Right-Turning Vehicles (Warrants)

The following provides general guidelines for when an acceleration lane for right-turning vehicles may be considered:



Note: See discussion in Section 9.3.5.

Source: (4) Revised

SCHEMATIC OF DUAL TURN LANES

Figure 9.3L

1. where necessary to meet the level-of-service criteria;

2. where a turning roadway is used (see Section 9.6);

3. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility;

4. where there is a significant history of rear-end and/or sideswipe accidents;

5. where there is inadequate intersection sight distance available; and/or

6. where there are high volumes of trucks turning onto the mainline.

In general, acceleration lanes for right-turning vehicles are applicable to high-speed, rural facilities. In urban areas, restricted conditions often preclude their use.

9.3.6.2 Acceleration Lanes in Medians (Warrants)

Acceleration lanes in the median are a specialized design which may have a site-specific application. The following provides general guidelines for when an acceleration lane in the median should be considered for left-turning vehicles:

1. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility. The acceleration lane may reduce the need for a signalized intersection;

2. where there is a significant history of rear-end and/or sideswipe accidents;

3. where there is inadequate intersection sight distance available; and/or

4. where there are high volumes of trucks turning onto the mainline.

The designer should see Reference (6) for additional guidance.

9.3.6.3 Design Criteria

The following design criteria should be considered when designing an acceleration lane at an intersection:

1. Types. Most acceleration lanes from at-grade intersections are the parallel design. However, a taper design may be more appropriate when a turning roadway is used. Chapter Ten provides additional information on the two types of acceleration lanes.

2. Lengths. Right-turn and median acceleration lanes should follow the criteria as presented in Chapter Ten for interchanges. The "controlling curve" at an intersection is the design speed of the turning roadway or the speed at which a vehicle can make the right turn, usually 10 mph or less. However, acceleration lanes in the median may warrant additional length because of traffic operational considerations for the inside through lane. Truck acceleration lengths may be considered when there is a substantial number of turning trucks.

3. Taper. The end of a parallel acceleration lane should be designed with a taper of 200 ft to 300 ft.

4. Sight Distance. The designer should ensure that adequate sight distance is available to the merge point of the acceleration lane.

9.4 TWO-WAY LEFT-TURN LANES (TWLTL)

Designs using the two-way left-turn lane (TWLTL) are often a cost-effective method to accommodate a continuous left-turn demand and to reduce delay and accidents. These lanes will often improve operations on roadways which were originally intended to serve the through movement but now must accommodate the demand for accessibility created by changes in adjacent land use.

9.4.1 Warrants

9.4.1.1 General

The physical conditions under which a TWLTL should be considered typically include:

1. areas with a high number of driveways per mile (e.g., 45 driveways total per mile on both sides);
2. areas of high-density commercial development; and
3. areas with substantial mid-block left turns.

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

9.4.1.2 Functional Class

An undivided, 4-lane urban or suburban arterial is the most common candidate for the implementation of a TWLTL. This is commonly referred to as a 5-lane facility. The use of a TWLTL on a 2-lane arterial (i.e., a 3-lane facility) may also be appropriate.

9.4.1.3 Traffic Volumes

Traffic volumes are a significant factor in the consideration of a TWLTL. When evaluating its use based on traffic volumes, the designer should use volumes projected for the project design year (see Chapters Twelve and Thirteen). As general guidance, the following should be used:

1. On 4-lane highways, a TWLTL will often be advantageous for traffic volumes between 10,000 and 25,000 ADT with a significant number of left-turning vehicles. On 2-lane highways, a TWLTL will often be advantageous for traffic volumes between 5000 and 12,000 ADT.
2. For traffic volumes greater than 30,000 ADT and/or greater than 1000 DDHV, a raised median should be considered; however, a 6-lane highway with a TWLTL (i.e., a 7-lane facility) may be the more advantageous design selection, especially where roadside development is extremely dense.
3. The decision on whether to provide a TWLTL for traffic volumes between 25,000 and 30,000 ADT will be determined on a case-by-case basis. See Reference (7) for additional information.

9.4.1.4 Pedestrians

Pedestrian crossing volumes are also a consideration because of the large paved area which must be traversed when a TWLTL is present (i.e., no pedestrian refuge exists).

9.4.1.5 Speed

The design speed on a highway facility is a major factor in TWLTL applications. Experience indicates that design speeds from 25 mph to 50 mph will properly accommodate a TWLTL on a 5-lane or 7-lane facility. For design speeds higher than 50 mph, their use is not recommended. For $V \geq 50$ mph, a rural transition section is an option. See Section 9.4.3.

Three-lane facilities may be appropriate where the design speed is 45 mph or less.

9.4.1.6 Accident History

The designer should review and evaluate the available accident data to determine if high numbers of accidents have occurred. On high-volume urban or suburban arterials, traffic conflicts often result because of a significant number of mid-block left turns combined with significant opposing traffic volumes and/or limited sight distance. A TWLTL may reduce these conflicts.

9.4.2 Design Criteria

9.4.2.1 Lane Width

Recommended lane widths for a TWLTL are presented in Chapters Twelve and Thirteen. Existing highways that warrant the installation of a TWLTL are often located in areas of restricted right-of-way, and conversion of the existing cross section may be difficult. To obtain the TWLTL 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 TWLTL.

The designer should evaluate the trade-offs between the benefits of the TWLTL and the negative impacts of modifying the existing cross section elements. This may involve a capacity analysis or an in-depth evaluation of the existing accident history.

9.4.2.2 Intersection Treatment

At intersections with public roads, the TWLTL should either be terminated in advance of the intersection, to allow the development of an exclusive left-turn lane, or be extended up to the intersection. In most cases where the TWLTL is extended up to the intersection, the pavement marking will switch from two opposing left-turn arrows to one left-turn arrow only. When determining the intersection treatment, the following should be considered:

1. Signals. At signalized intersections the TWLTL should be terminated because these intersections will normally warrant an exclusive left-turn lane.
2. Turning Volumes. The left-turn demand into the intersecting road is a factor in determining the proper intersection treatment. As general guidance, if the minimum storage length will govern (Section 9.3), it will probably be preferable to extend the TWLTL up to

the intersection (i.e., provide no exclusive left-turn lane).

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

   a. On facilities where $V \leq 30$ mph and/or lower traffic volumes exist, the minimum uninterrupted length of a TWLTL should be 300-400 ft.

   b. On facilities where $V > 30$ mph and/or higher traffic volumes exist, the minimum uninterrupted length of a TWLTL should be 500-600 ft.

   The final decision on the length of the TWLTL will be based on site conditions in coordination with the Traffic Engineering Division and the Urban Design Division, Geometric Design Branch.

9.4.2.3 Railroad Crossings

A TWLTL should not extend across a railroad/highway grade crossing. The TWLTL is striped out in advance of the crossing on both sides by a distance of 100-ft desirable and 50-ft minimum. The designer should coordinate with the Traffic Engineering Division.

9.4.3 Rural Transition Section

9.4.3.1 Warrants

In some cases, a rural transition section may be appropriate, which is a variation of the TWLTL. The rural transition section provides a design which may be advantageous in areas which have both urban and rural features. The use of this section should be considered in transitional areas where design speeds are 50 mph or higher and on the following facilities:

1. major collectors on the State highway system with design year ADT between 8000 and 18,000;

2. principal arterials (other than freeways and arterials with partial control of access) with design year ADT between 5000 and 18,000; and

3. other arterials with design year ADT between 7200 and 18,000.

The rural transition section may also be considered in urban areas on facilities which meet these criteria. For this purpose, an urban area is defined as an area currently developed or having probable future development (within the forecast period) as strip commercial or lot development of 0.5 acres or less and where at-grade access is allowed.

9.4.3.2 Design

The rural transition section is designed according to the typical criteria in Table 9.4A.

Table 9.4A

RURAL TRANSITION SECTION
(Typical Design Criteria)

Design Feature	Design Criteria
Facility Type	4-lane divided
Travel Lanes	12′
Outside Shoulders	8′ paved
Median	16′ paved flush (asphalt) with rumble strips adjacent to inside travel lane

9.4.4 4 Lane/5 Lane Design Guidelines

The following tables are intended for use by the designer for determining the best typical section on both rural and urban routes. It gives guidance of which there is still much room for overlap. All features of a particular route should be considered to come up with the best alternative. The designer should not focus on one particular feature when selecting a typical section, but rather look for a preponderance of the features which make that the most suitable alternative. It is advisable to conduct a thorough site review of each project to check for site specific traffic generators which may dictate a change in the typical section.

Table 9.4B

4 LANE/5 LANE DESIGN GUIDELINES

Feature	4- Lane Undivided		4-Lane Divided	5-Lane w/TWLTL(a)	
	Curbed	10' Shoulders		Curbed	10' Shoulders
Functional Class	Collectors or Minor Arterials		Principal Arterials	All Routes (c)	
Traffic Volumes	DHV < 1000 vph ADT < 10,000 vpd		DHV > 1000 vph ADT > 7200 vpd	DHV <1000 vph ADT <30,000 vpd	DHV < 1000 vph ADT < 20,000 vpd
Projected Development	Low to Moderate		Low - Isolated Traffic Generators	Moderate to High	
Intersection Density (d)	≤ 4 intersections./mi.		≤ 4 int/mi.	> 4 intersections/mi.	
Driveway Density	≤ 45 drives/mi. 40% or more Commercial	≤45 drives/mi. Predominately Residential	≤ 45 drives/mi.	> 45 drives/mi. Predominately Commercial	> 45 drives/mi. Predominately Residential
Pedestrian Traffic	Curb or sidewalk warranted (b)	Minimal Pedestrian Traffic	Minimal Pedestrian Traffic	Curb or sidewalk warranted (b)	Minimal Pedestrian Traffic
Design Speed (e)	45 mph	65 mph	65 mph+	45 mph	55/65 mph (g)
Access Control	None	None	Full or Partial (f)	None	None

Footnotes:

(a) Complete TWLTL warrants are covered in section 9.4 of the Design Manual.

(b) See Section 8.1.5 of the Design Manual for curb warrants and Section 8.1.6 of the Design Manual for sidewalk warrants.

(c) 5-Lane section should be avoided on NHS routes except where there is existing high density commercial development.

(d) Intersections include any public streets or roads as well as drives which generate a large volume of left turns, such as those into malls or large retail outlets.

(e) Design speed noted in the table may of course be increased. Posted speed may be based on appropriate speed studies.

(f) Full control required on interstate or other facilities designed to Interstate standards. Partial control (access at section line and quarter section line) is preferred, but not required, on all others.

(g) In order to increase to a 65 mph design speed, the median must be 16' paved with rumble strips adjacent to the inside travel lanes.

Table 9.4C

ADVANTAGES (+) AND DISADVANTAGES (-) OF THE DESIGN ALTERNATIVES

Advantage/Disadvantage	4 Undivided		4 Divided	5 w/TWLTL	
	Curbed	Shoulders		Curbed	Shoulders
Cost					
Initial	-	+	-	-	+
Maintenance	+	+	-	+	+
R/W Needs	+	+	-	+	+
Design Speed	-	+	+	-	-
Capacity	-	-	+	+	+
Delay Time	-	-	+	+	+
Control of Future Improvements	-	-	+	-	-
Access to Adjoining Properties	+	+	-	+	+
Left Turn Storage	-	-	+	+	+
Accident Reduction					
Angle	-	-	+	-	-
Rear End	-	-	+	+	+
Head On	-	-	+	+	+
Sideswipe	-	-	+	-	-
Pedestrian Safety					
Length Crossing	+	+	-	-	-
Refuge @ Median	-	-	+	-	-
Refuge Along Facility	+	-	-	+	-
Side Street Traffic					
Crossing Highway	-	-	+	-	-
Turning Left Onto Highway	-	-	+	+	+
Mail Delivery [a]	-	+	+	-	+
School Bus [b]	-	+	+	-	+

(a) Assuming mail boxes are located on opposite sides of the highway (mail delivery vehicle must go up one side and down the other).

9.5 TURNING RADII (Right Turns)

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

9.5.1 Design for Pavement Edge/Curb Line

Once the designer has selected the design vehicle (Section 9.1), several other factors must be considered to determine the proper pavement edge/curb line. The following sections present several of the basic parameters the designer needs to consider.

9.5.1.1 Inside Clearance

Desirably, the selected design vehicle will make the right turn while maintaining approximately a 2-ft clearance from the pavement edge or curb line and, at a minimum, will not come closer than 9 inches.

9.5.1.2 Encroachment

To determine the acceptable encroachment, the designer should evaluate several factors. The following will apply:

1. Urban. In all cases, there will desirably be no encroachment by the design vehicle. Where this is not practical, the designer must determine the acceptable encroachment for a right-turning design vehicle at urban intersections. The acceptable encroachment will be based on an assessment of the following factors:

    a. turning traffic volumes,

    b. through traffic volumes,

    c. right-of-way impacts,

    d. construction impacts,

    e. at existing intersections, accident history,

    f. type of traffic control,

    g. number of lanes in each direction,

    h. one-way or two-way operation, and

    i. functional classes of intersecting facilities.

2. Rural. For rural intersections, the selected design vehicle should not encroach onto the adjacent lane on the road from which the turn is made, or into the opposing lanes of traffic onto the road which the turn is made. 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. The adjacent lane refers to the lane immediately to the left of the design vehicle.

Desirably, the selected design vehicle will be able to make the right turn while remaining entirely in the right through lane. However, if there are two or more lanes in the same direction on the road onto which the turn is made, the design vehicle can occupy both travel lanes. If an acceleration lane is provided for the turning vehicle, the design vehicle should make the turn entirely within the acceleration lane.

The following is additional guidance in the determination of vehicular encroachment:

3. At all intersections school buses, garbage trucks, delivery trucks, etc., should be able

to physically make the turn without backing up and without impacting parked cars or obstacles behind curbs.

4. At all intersections with State highways, the WB-67 design vehicle should be able to physically make the turn without backing up and without impacting obstacles.

5. The designer should consider maintenance impacts from encroachment by turning vehicles beyond curbs or pavement edges.

9.5.1.3 Parking Lanes/Shoulders

At many intersections, parking lanes and/or shoulders will be available on one or both approaches, and this additional roadway width may be carried through the intersection. This will greatly ease the turning problems for large vehicles at intersections with small curb radii. Figure 9.5A illustrates the turning paths of several design vehicles for radii of 15 ft or 25 ft, where 8-ft to 10-ft parking lanes are provided. The presence of a shoulder 8 ft to 10 ft in width will have the same effect as a parking lane. After application of the selected design vehicle, the designer may find that even a 5-ft radius with a parking lane/shoulder may be sufficient.

Figure 9.5A also illustrates the minimum distance to restrict parking before the PC (15 ft) and after the PT (20-40 ft) on the cross street to accommodate vehicular turning. Section 17.1 presents specific criteria for the location of on-street parking relative to intersections. The designer should check the proposed design with the applicable turning template and encroachment criteria. The designer should not consider the beneficial effects of a parking lane if the lane will be used for through traffic for part of the day.

Where there are low turning volumes, the typical shoulder pavement structure may be used. However, where there are high volumes of turns and/or where there are significant numbers of turning trucks, a strengthened shoulder pavement should be constructed for vehicular turning paths. Chapter Sixteen discusses pavement design.

9.5.1.4 Pedestrians

Larger turning radii create long stretches of pavement which pedestrians must cross. Designs using a simple curve with taper offsets, a 3-centered compound curve or a turning roadway (see Section 9.6) may minimize crossing times or distances.

9.5.1.5 Types of Turning Designs

Once the designer has determined the basic turning parameters (e.g., design vehicle, encroachment, inside clearance), it is necessary to select a type of turning design configuration for the curb return or pavement edge which will meet these criteria and will fit the intersection constraints. The design may use one or more of the following basic types:

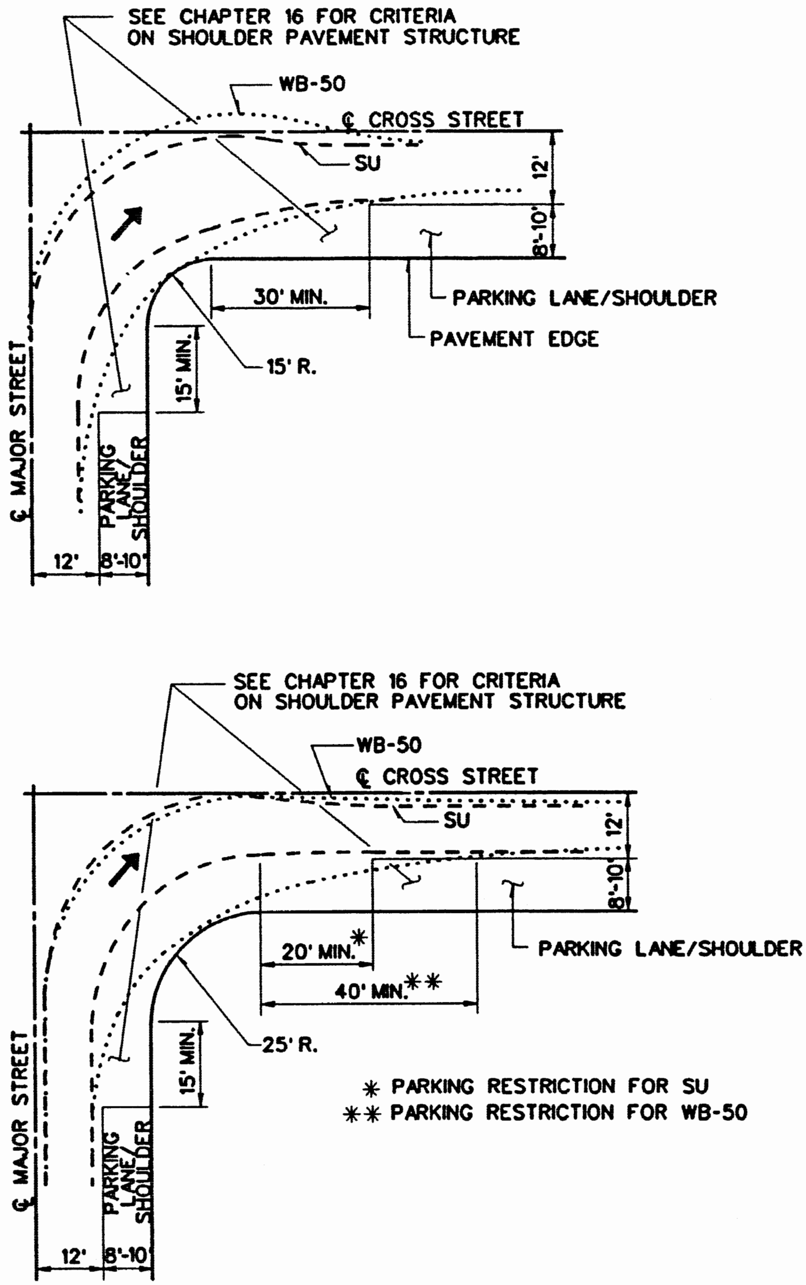
1. simple radius,

2. simple radius with entering and exiting tapers,

3. simple radius with an exiting taper only, or

4. 3-centered symmetrical compound curve.

Figure 9.5B illustrates all four basic turning designs. Each design type has its advantages and disadvantages. The simple radius is the easiest to design and construct and, therefore, it is the most common. The 3-centered symmetrical compound curve arrangement provides the "best" fit to the transitional turning paths of vehicles. However, the



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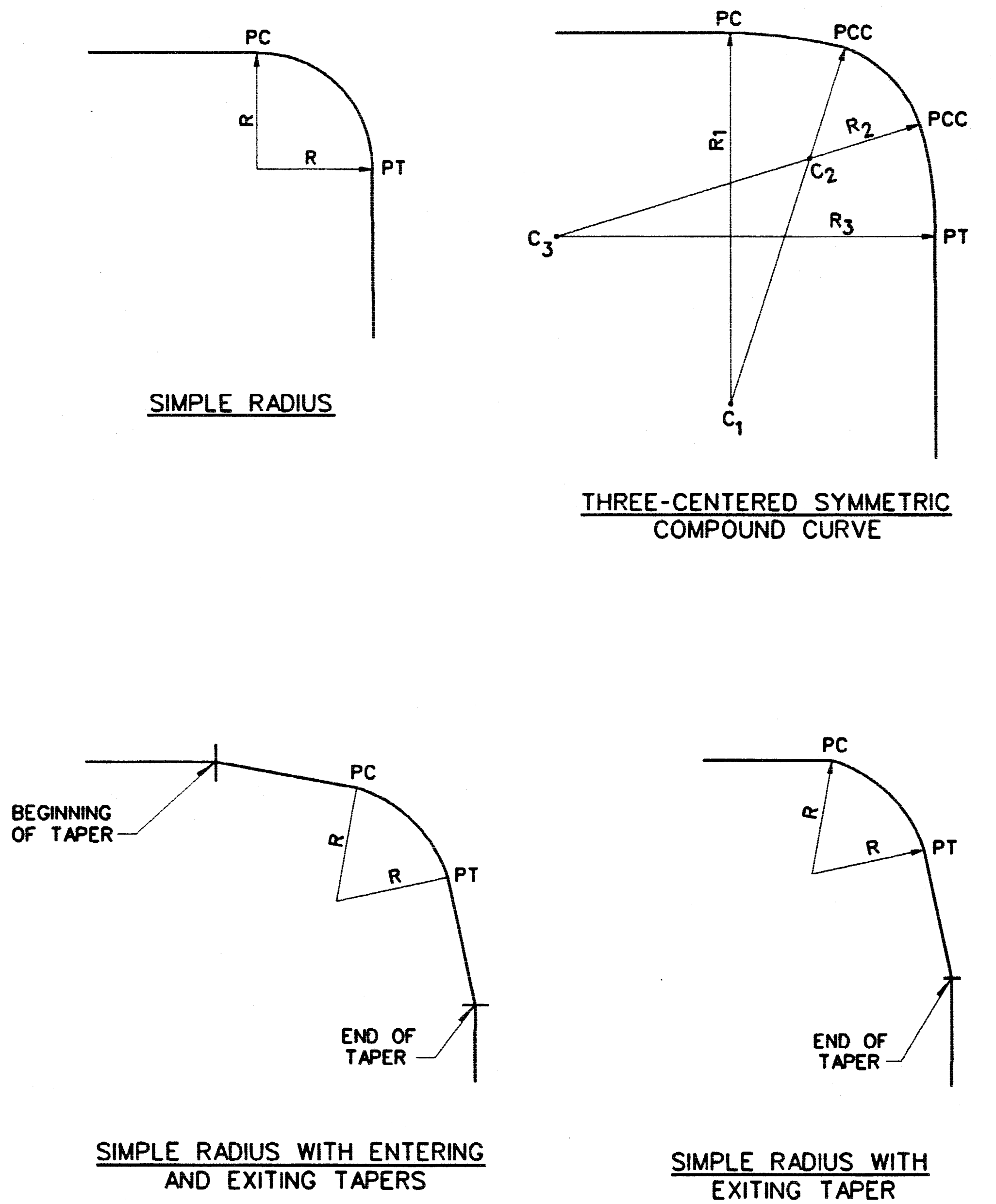
EFFECT OF CURB RADII AND PARKING ON TURNING PATHS

Figure 9.5A



TYPES OF INTERSECTION TURNING DESIGNS

Figure 9.5B

designer should also consider the benefits of the simple radius with an entering and/or exiting taper. Its advantages as compared to other designs include:

a. The simple radius with tapers when compared to the compound curvature arrangement is easier to design, survey and construct, and produces a minimal amount of additional pavement.

b. To accommodate a specific design vehicle, a simple radius requires greater intersection pavement area than a radius with tapers. Another benefit is the reduced right-of-way impact in the intersection corners.

c. A simple radius results in greater distances for pedestrians to cross than a radius with tapers.

d. For angles of turn greater than 90°, a radius with tapers is a better design than a simple radius, primarily because less intersection area is required.

9.5.1.6 Turning Template

To determine the final design, the designer uses a turning template for the selected design vehicle. The template is applied to the intersection layout to determine how best to meet the criteria for turning radii design.

9.5.2 <u>Turning Radius Design Steps</u>

Figure 9.5C illustrates the many factors which should be evaluated in determining the proper design for right-turns at intersections. The following steps apply:

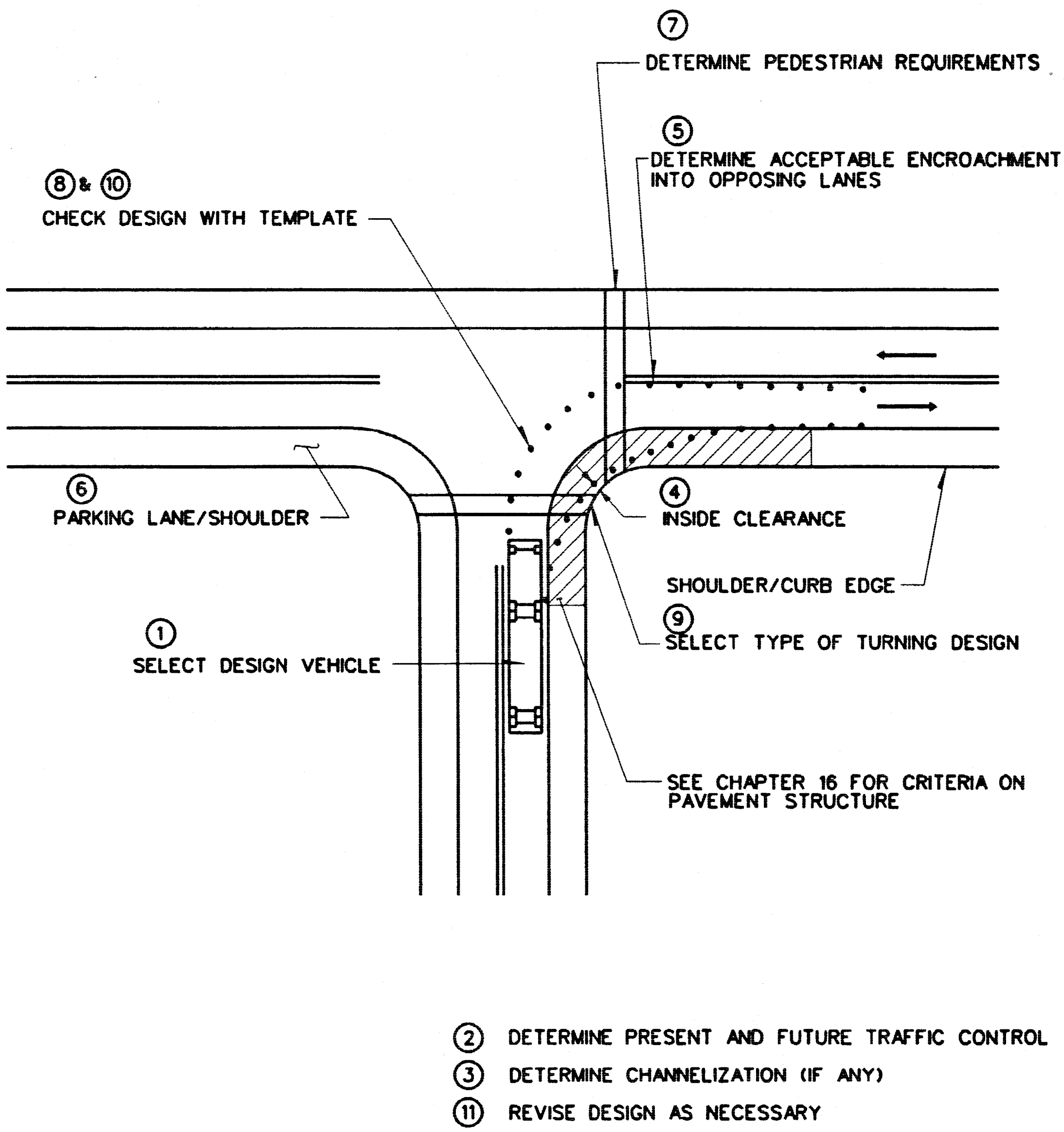
1. Select the design vehicle (Table 9.1A).
2. Determine the present and future traffic control.
3. Determine appropriate channelization (through/turn-lane requirements, islands, etc.).
4. Determine the acceptable inside clearance (Section 9.5.1.1).
5. Determine the acceptable encroachment (Section 9.5.1.2).
6. Consider the benefits of any parking lanes or shoulders (Section 9.5.1.3).
7. Determine pedestrian requirements (Section 9.5.1.4) and determine island placement where needed.
8. Place the selected design vehicle template on the paper, superimpose the template on the intersection drawing (1"=20′ or 1"=30′ preferred), and try the various turning design configurations (Comment #9).
9. Select the type of turning design configuration (Section 9.5.1.5):

   a. simple radius,

   b. simple radius with entering and exiting tapers,

   c. simple radius with an exiting taper only, or

   d. 3-centered compound curve.
10. Check all proposed designs with the applicable vehicular turning templates.
11. Revise the design as necessary to accommodate the right-turning vehicle <u>or</u> determine that it is not practical to meet this design because of adverse impacts.



TURNING RADII DESIGN STEPS

Figure 9.5C

9.6 TURNING ROADWAYS

Turning roadways are channelized areas (separated by an island) at at-grade intersections which allow a moderate-speed, free-flowing right turn. Interchange ramps are not considered turning roadways.

9.6.1 Warrants

9.6.1.1 All Intersections

The warrants for providing a turning roadway are based on the area (urban or rural), the functional classification and the geometrics of the intersection. The designer should consider using turning roadways when:

1. it is desirable to allow right turns at speeds of 15 mph or more;
2. the angle of turn is greater than 90°;
3. the volume of right turns is high, the turning movement is from a high-volume road or it is desirable to remove right turns away from a signal;
4. the turning roadway is needed to meet the level-of-service criteria;
5. it is desirable to reduce the intersection paved area. As a guide, if the unused pavement area based on the criteria for right turns (Section 9.5) will be at least 100 sq ft, then a turning roadway should be considered;
6. pedestrian volumes are high and a pedestrian refuge is a desirable feature;
7. there are significant numbers of rear-end accidents. Turning roadways allow vehicles to make the turning movements at higher speeds and consequently should reduce these types of accidents;
8. the selected design vehicle is a semi-trailer combination; and/or
9. the island area is needed for the placement of traffic control devices.

When a turning roadway is warranted, a major consideration may be the availability of sufficient right-of-way.

The following sections discuss warrants more specifically for rural and urban intersections.

9.6.1.2 Rural

Turning roadways are commonly provided at the following rural intersections:

1. State Highway to State Highway (All-Way Stop). A turning roadway is typically provided for all right-turn movements.
2. State Highway to State Highway (Stop Control on Minor Highway). A turning roadway is typically provided for right-turning vehicles from the stop-controlled road to the major road. They are usually not provided from the major road to the stop-controlled road.
3. All Others. For all other intersections, the decision on whether a turning roadway is warranted is determined on a case-by-case basis.

9.6.1.3 Urban

For urban intersections, the need for a turning roadway is determined on a case-by-case analysis based on the factors previously listed. However, the designer should consider the following:

1. Turning roadways commonly require more right-of-way than simple intersections.

2. Turning roadways may be provided for right-turning vehicles from the stop-controlled road to the major road.

3. Existing turning roadways are usually removed when converting an intersection from unsignalized to signalized. When converting a highway from a 2-lane to a 4-lane facility, the need to retain existing turning roadways will be determined based on an intersection analysis.

9.6.2 Design Criteria

Figure 9.6A illustrates a typical design for a turning roadway. This figure presents a turning roadway with a simple curve radius with an exiting taper, although a simple curve is often acceptable. Chapter Ten also presents may figures for the typical design of an at-grade intersection between an interchange ramp and crossing road. These figures include turning roadway designs for various combinations of design vehicles, urban/rural location and type of crossing road.

9.6.2.1 Design Speed

Typically, the design speed for a turning roadway will be in the range of 5-20 mph. A turning roadway at a low design speed (e.g., 15 mph) will still provide a significant benefit to the turning vehicle regardless of the speed on the approaching highway.

9.6.2.2 Width

Turning roadway widths depend on the edge of pavement radius and design vehicle selected. See Table 9.6A. Table 9.1A should be used to select the design vehicle. However, the designer should also consider the possibility that larger vehicles will use the turning roadway. If there are 10 to 20 or more larger vehicles a day using the turning roadway, the larger design vehicle widths should be selected.

Uncurbed sections provide greater latitude to the occasional larger vehicle. Curbed sections limit the options available to the larger truck. Consequently, if a curb is present on the mainline approaching the turning roadway, the gutter width should be maintained throughout the turning roadway.

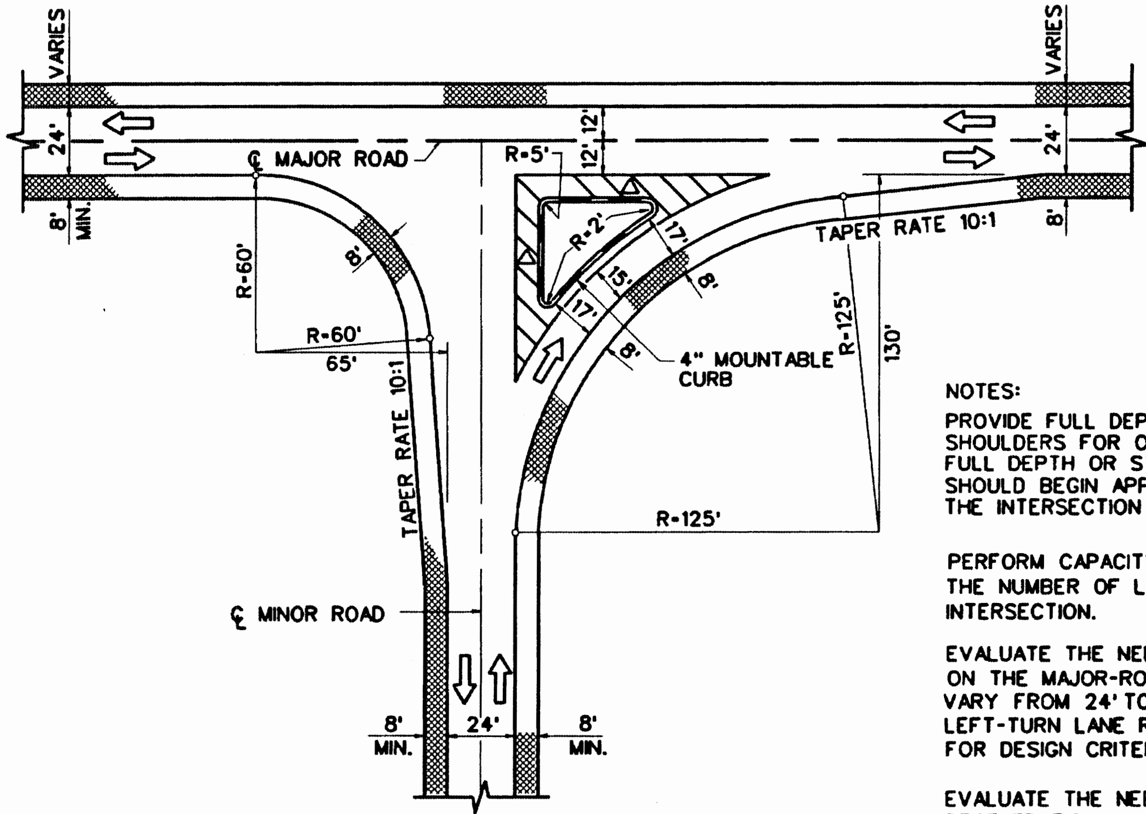
The full-depth pavement structure will be provided for the entire turning roadway width.

9.6.2.3 Horizontal Curvature (Theoretical Discussion)

The theory for horizontal curvature assuming turning roadway conditions differs from the theoretical assumptions for open-road conditions, which are discussed in Chapter Six. In comparison, turning roadway conditions are less restrictive, which reflects more restrictive field conditions, less demanding driver expectation, and a driver acceptance of a less generous design. Chapter III of the 1990 *Green Book* discusses the theoretical assumptions for horizontal curvature on turning roadways in detail.

9.6.2.4 Curvature Arrangement

At most turning roadways, a simple curve with an exiting taper is the typical curvature arrangement. However, the designer can also consider the use of a simple radius throughout or a 3-centered compound curve, if determined to be a more practical design. If a 3-centered curve is used, the radius of the flatter curve should be desirably 1.75 times the radius of the sharper curve but no more than twice the radius of the sharper curve. In addition, the lengths of the entering and



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING. FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

PERFORM CAPACITY ANALYSIS TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE INTERSECTION.

EVALUATE THE NEED FOR LEFT-TURN LANES ON THE MAJOR-ROAD. PAVEMENT WIDTH MAY VARY FROM 24' TO 40' DEPENDING ON LEFT-TURN LANE REQUIREMENTS. SEE SECTION 9.3 FOR DESIGN CRITERIA.

EVALUATE THE NEED FOR ACCELERATION AND DECELERATION LANES ON THE MAJOR ROAD. SEE SECTION 9.3 FOR DESIGN CRITERIA.

△ = OFFSET ISLAND TO BACK OF SHOULDER.

TYPICAL TURNING ROADWAY BASED ON WB-67 DESIGN VEHICLE
(Stop Control on Minor Road)

Figure 9.6A

Table 9.6A

TURNING ROADWAY WIDTHS

Radius on Inner Edge of Traveled Way (ft)	Minimum Pavement Width Exclusive of Shoulders (ft)	Total Pavement Width (W) for Selected Design Vehicle		
		SU (ft)	WB-67 (ft)	WB-114 (ft)
50	18	25	29	37
75	17	23	26	33
100	16	22	26	31
150	16	21	24	25
200	16	21	21	23
300	15	20	20	20
400	15	20	20	20
500	15	20	20	20
Tangent	15	19	19	19

Source: (1) Revised

Notes:

1. If a barrier curb is used on one side, an additional 1′ should be added to the values in the table.

2. If a barrier curb is used on both sides, an additional 2′ should be added to the values in the table.

3. The total pavement width (including shoulders) should accommodate both a) the largest design vehicle expected to use the turning roadway, and b) the Case IIB widths from Table III-21 of the 1990 *Green Book*. For turning roadways where the turning truck volume exceeds 10%, the designer should compare the widths in Table 9.6A to the Case IIC widths in Table III-21 of the *Green Book* and use the larger of the two.

exiting curves should meet the criteria in Table 9.6B.

Table 9.6B

LENGTHS OF ENTERING/EXITING CURVES (3-Centered Curve)

Radius (ft)	100 or less	150	200	250	300	400	500+
Minimum Length (ft)	40	50	60	80	100	120	140
Desirable Length (ft)	60	70	90	120	140	180	200

Source: (1)

9.6.2.5 Minimum Radius

Table 9.6C presents minimum radii for various turning roadway conditions. The basic equation given in the table is used directly to calculate the minimum radius. As discussed in Section 9.6.2.6, a range of superelevation rates may be used. Therefore, Table 9.6C presents minimum radii for several assumed superelevation rates.

9.6.2.6 Superelevation

Superelevation development on turning roadways does not need to meet the rigid criteria of open highways. The practicality of quickly introducing and removing a curve at an intersection requires flexibility. In many cases at design speeds of 10-20 mph, the superelevation rate throughout the roadway will be 2%, the typical cross slope. The preferred maximum allowable superelevation rate is 0.06; however, a rate of 0.08 may be acceptable under restricted conditions. Table 9.6D presents ranges of superelevation rates for various design speeds. These ranges represent a practical balance between the desire to provide superelevation and the typical distance available for superelevation development.

Figure 9.6B illustrates a schematic of superelevation development at a turning roadway. The actual development will depend upon the practical field conditions combined with a reasonable consideration of the theory behind horizontal curvature. The following presents criteria which should be met:

1. No change in the normal cross slope is necessary up to Section B-B. Here, the width of the turning roadway is about 2 ft.

2. The full width of the turning roadway should be attained at Section D-D. The amount of superelevation at D-D will depend upon practical field conditions. Assuming adequate superelevation transition distance is available, the designer should provide approximately 75% of the full superelevation at this point.

3. Beyond Section D-D, the turning roadway pavement should be rotated as needed to provide the required superelevation for the design speed of the turning roadway.

4. Table 9.6E presents desirable and minimum superelevation transition lengths for combinations of design speed and superelevation rate. Desirably, when a turning roadway is superelevated, the transition length will meet the criteria presented in Chapter Six for relative longitudinal slope. For open roadways, the relative slope is measured between the centerline of the roadway and either pavement edge. Because the turning roadway widths are often in the range of a 2-lane roadway, the relative longitudinal slope is also measured between the centerline of the turning roadway and either pavement edge.

Table 9.6C

MINIMUM RADII FOR TURNING ROADWAYS

Turning Roadway Design Speed* (V, mph)	Maximum Side Friction (f)	Assumed Super-Elevation** (e)	Minimum Radius (ft)
15	.32	.02 .04 .06 .08	45 45 40 40
20	.27	.02 .04 .06 .08	95 85 80 80
25	.23	.02 .04 .06 .08	170 155 145 135
30	.20	.02 .04 .06 .08	275 250 230 215
35	.18	.02 .04 .06 .08	410 375 340 315
40	.16	.02 .04 .06 .08	595 535 485 445

Source: (1) Revised

* For design speeds greater than 40 mph, use open-roadway conditions. See Chapter Six.

** See Table 9.6D for recommended range of superelevation rates for a given radius and design speed.

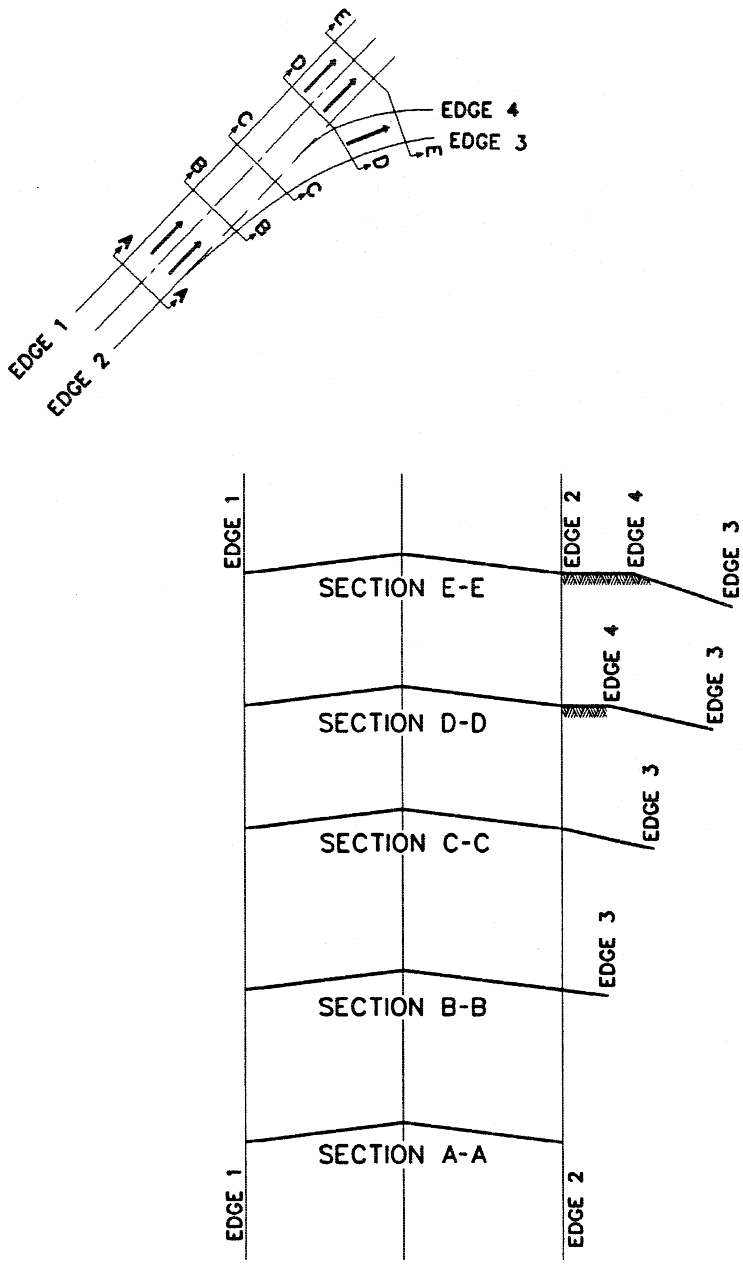
Notes:
1. e_{max} = 0.06 is preferred.
2. Minimum radii are based on the following equation: $R_{min} = \frac{V^2}{15(e+f)}$

Table 9.6D

SUPERELEVATION RATES
(Turning Roadways)

Radius (ft)	Range in Superelevation Rate for Turning Roadways with Design Speed (mph) of					
	15	20	25	30	35	40
50	.02-.06	—	—	—	—	—
90	.02-.06	.02-.06	—	—	—	—
150	.02-.05	.02-.06	.04-.06	—	—	—
230	.02-.04	.02-.06	.03-.06	.06	—	—
310	.02-.03	.02-.04	.03-.06	.05-.06	—	—
430	.02-.03	.02-.03	.03-.05	.04-.06	.06	—
500	.02-.03	.02-.03	.03-.05	.04-.06	.06	—
600	.02	.02-.03	.02-.04	.03-.05	.05-.06	—

Source: (1) Revised



Source: (1)

DEVELOPMENT OF SUPERELEVATION AT TURNING ROADWAY TERMINALS

Figure 9.6B

Table 9.6E

SUPERELEVATION TRANSITION LENGTHS
(Turning Roadways)

Design Speed (mph)	Superelevation Rate	Transition Length (ft)	
		Desirable	Minimum
15 - 25	.02 .04 .06	75′ 75′ 101′	34′ 68′ 101′
30	.02 .04 .06	100′ 100′ 108′	36′ 72′ 108′
35	.02 .04 .06	115′ 115′ 115′	38′ 77′ 115′
40	.02 .04 .06	125′ 125′ 126′	42′ 84′ 126′

Notes:

1. Transition lengths assume:

   a. relative longitudinal slopes for open roadways between centerline and pavement edge (see Chapter Six);
   b. 24′ turning roadway width (12′ used for equation calculations);
   c. main highway is on a tangent or curving to the right;
   d. desirable lengths are subject to a length based on 2 seconds of travel time; and
   e. minimum lengths are calculated lengths; i.e., the 2 seconds of travel time does not govern.

2. For turning roadway widths less than 24′, use table values. For turning roadway widths greater than 24′, increase minimum transition lengths in a straight-line proportion.

3. Where superelevation of the main roadway produces an adverse cross slope rollover, transition lengths will be determined on a case-by-case basis.

4. See discussion in Section 9.6.2 for more information on superelevation development.

For open roadways, calculated superelevation transition lengths are subject to minimum values based on 2 seconds of travel time at the design speed. However, because of the often restrictive nature of turning roadways, it is acceptable to use the calculated superelevation transition lengths at a minimum. This is reflected in Table 9.6E

5. The superelevation treatment for the exiting portion of the turning roadway should be similar to that described for the entering portion. However, the designer should consider the traffic control for the merging maneuver. For stop-control, the superelevation on the turning roadway should match the cross slope on the merging highway or street.

9.6.2.7 Cross Slope Rollover

Table 9.6F presents the maximum allowable algebraic difference in the cross slopes between the mainline and turning roadway where these two are adjacent to each other. In Figure 9.6B, these criteria apply between Section A-A and Section D-D. This will likely be a factor only when a superelevated mainline is curving to the left.

Table 9.6F

PAVEMENT CROSS SLOPE AT TURNING ROADWAY TERMINALS

Design Speed of Curve at Section D-D (mph) (Figure 9.6B)	Maximum Algebraic Difference in Cross Slope at Crossover Line (percent)
15-20	5 - 8
25-30	5 - 6
> 30	4 - 5

Source: (1)

9.6.2.8 Acceleration Lanes

An acceleration lane for the merging portion of the turning roadway is required for free-flowing merging movements. A lane may also be justified for turning roadways with yield control. Acceleration lanes will provide the turning roadway vehicles with an opportunity to accelerate before merging. The length of the full-width acceleration lane will be based on the design speed of the turning roadway and the design speed of the mainline. Desirably, this acceleration length will be the same as that used for entrance ramps at interchanges (see Chapter Ten).

9.7 CHANNELIZED ISLANDS

Several of the treatments described in this chapter require channelized islands within the intersection area. The design of islands should consider several site-specific functions, including definition of vehicular paths, separation of traffic movements, prohibition of movements, protection of pedestrians and placement of traffic control devices.

9.7.1 Types of Islands

Islands can be grouped into the following functional classes. Most islands serve at least two of these functions:

1. Directional Islands. Directional islands control and direct traffic movements and guide the driver into the proper channel.

2. Divisional Islands. 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.

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 phasing is used. Refuge islands may also serve as areas for the installation of traffic control devices, primarily stop signs.

9.7.2 Selection of Island Type

Channelized islands may be some combination of flush or raised, paved or textured, concrete/bituminous or turf, and triangular or elongated. Selection of an appropriate type of traffic island should be based on traffic characteristics, cost considerations and maintenance needs. Flush islands are appropriate:

1. on high-speed rural highways to delineate separate turning lanes;

2. in constrained locations where vehicular path definition is desired, but space for larger, raised islands is not available;

3. to separate opposing traffic streams on low-speed streets; and/or

4. for temporary channelization during construction.

Raised islands are appropriate:

5. where a primary function of the island is to provide pedestrian refuge;

6. where a primary or secondary island function is the location of traffic signals, signs or various fixed objects;

7. where the island is intended to prohibit or prevent traffic movements;

8. on low- to moderate-speed highways where the primary function is to separate high volumes of opposing traffic movements; and/or

9. at locations requiring positive delineation of vehicular paths, such as at major-route turns or intersections with unusual geometry.

Channelized islands with barrier curbs should only be used where the design speed is 45 mph or less. However, mountable curbs are the preferred choice at all design speeds. See the discussion on curbs in Section 8.1.

9.7.3 Island Design

9.7.3.1 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 recommended minimum size is 100 sq ft. Raised elongated islands should not be less than 4-ft wide and 25-ft long. In restricted areas, an elongated island may be flush and 2-ft wide.

The corners of curbed islands should be constructed with noses of 2-ft to 5-ft radii.

9.7.3.2 Delineation

Channelized islands should be delineated based on their size, location and function. Raised islands outlined by barrier curbs present the most positive means of delineation. Islands delineated by mountable curb, pavement markings or rumble strips should be used in areas where space is limited or on highways with higher design speeds. On flush islands, it may be appropriate to complement the pavement markings with raised reflectors.

Raised pavement markings, raised reflectors, 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 divisional curbed islands.

The designer should coordinate any delineation treatments for the island with the Traffic Engineering Division.

9.7.3.3 Island Offsets

Curbed islands should be offset from the approaching roadway for the comfort and safety of the driver. The following will apply:

1. <u>No Shoulders, No Right-Turn Lane</u>. Where the approaching roadway has no shoulders (i.e., it is curbed) and no right-turn lane, the nose of the island should be offset 4 ft to 6 ft from the edge of the travel lane. It will then be tapered to a 2-ft offset.

2. <u>With Shoulders, No Right-Turn Lane</u>. Where the approaching roadway has shoulders but no right-turn lane, the curbed island should be offset a distance equal to the shoulder width from the edge of the travel lane.

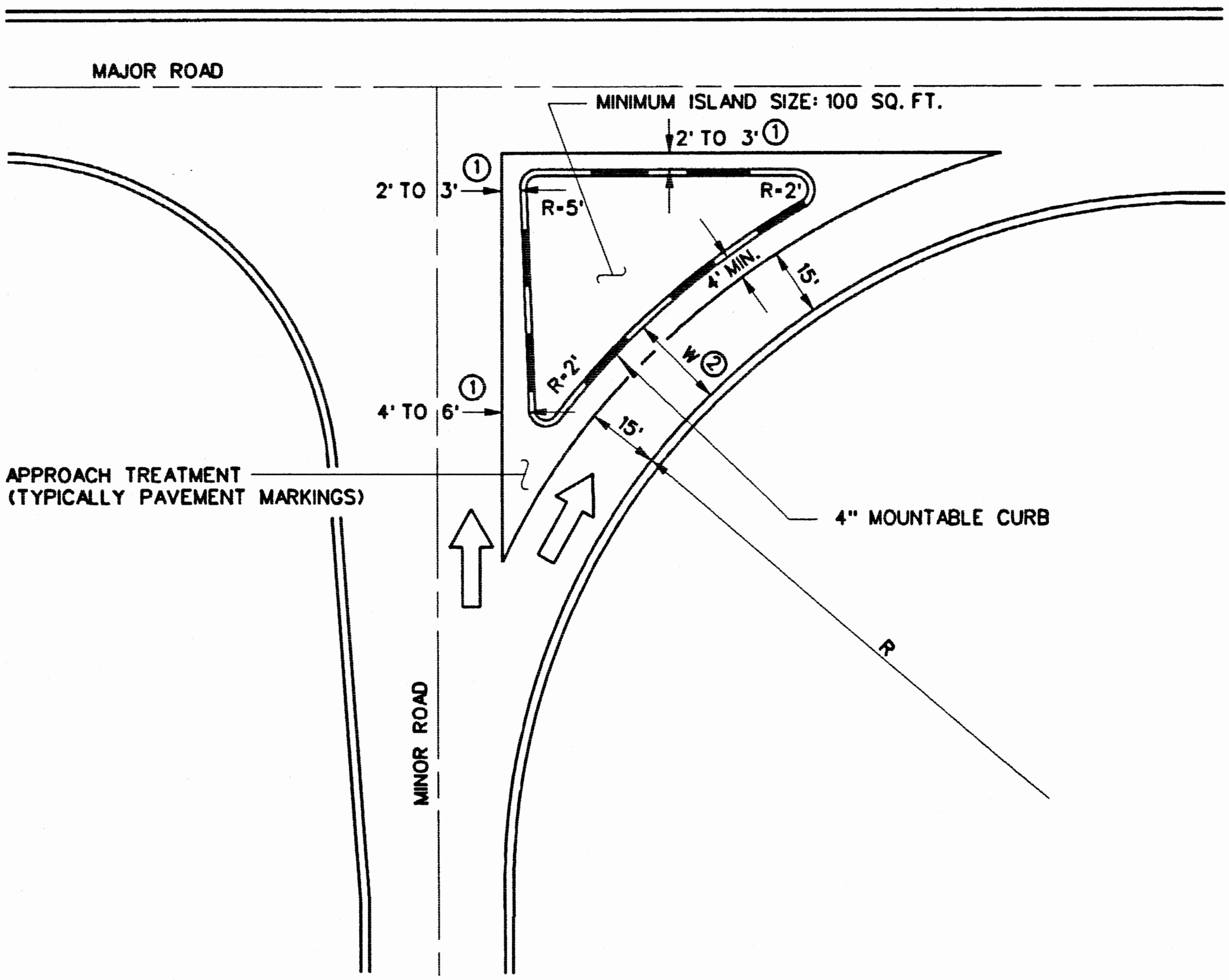
3. <u>With Right-Turn Lane</u>. Where the roadway has a right-turn lane approaching the intersection, the curbed island should be offset a minimum of 2 ft from the outside edge of the right-turn lane. This applies to a right-turn lane on either roadway.

In addition, the designer needs to ensure that the island will not interfere with the turning movement of a truck turning from the opposite side on a 4-legged intersection. If there is a conflict, the island should be set back further or made flush.

9.7.4 Typical Channelized Islands

Each channelized intersection must be studied individually considering turning volumes, traffic lane configurations, potential conflicts and practical signing arrangements. The following presents several typical island treatments:

1. Figure 9.7A "Typical Curbed Triangular Island (Roadway Without Shoulders)."
2. Figure 9.7B "Typical Curbed Triangular Island (Roadway With Shoulders)."
3. Figure 9.7C "Typical Divisional Island."
4. Figure 9.7D "Example of a Channelized Intersection (Rural)."



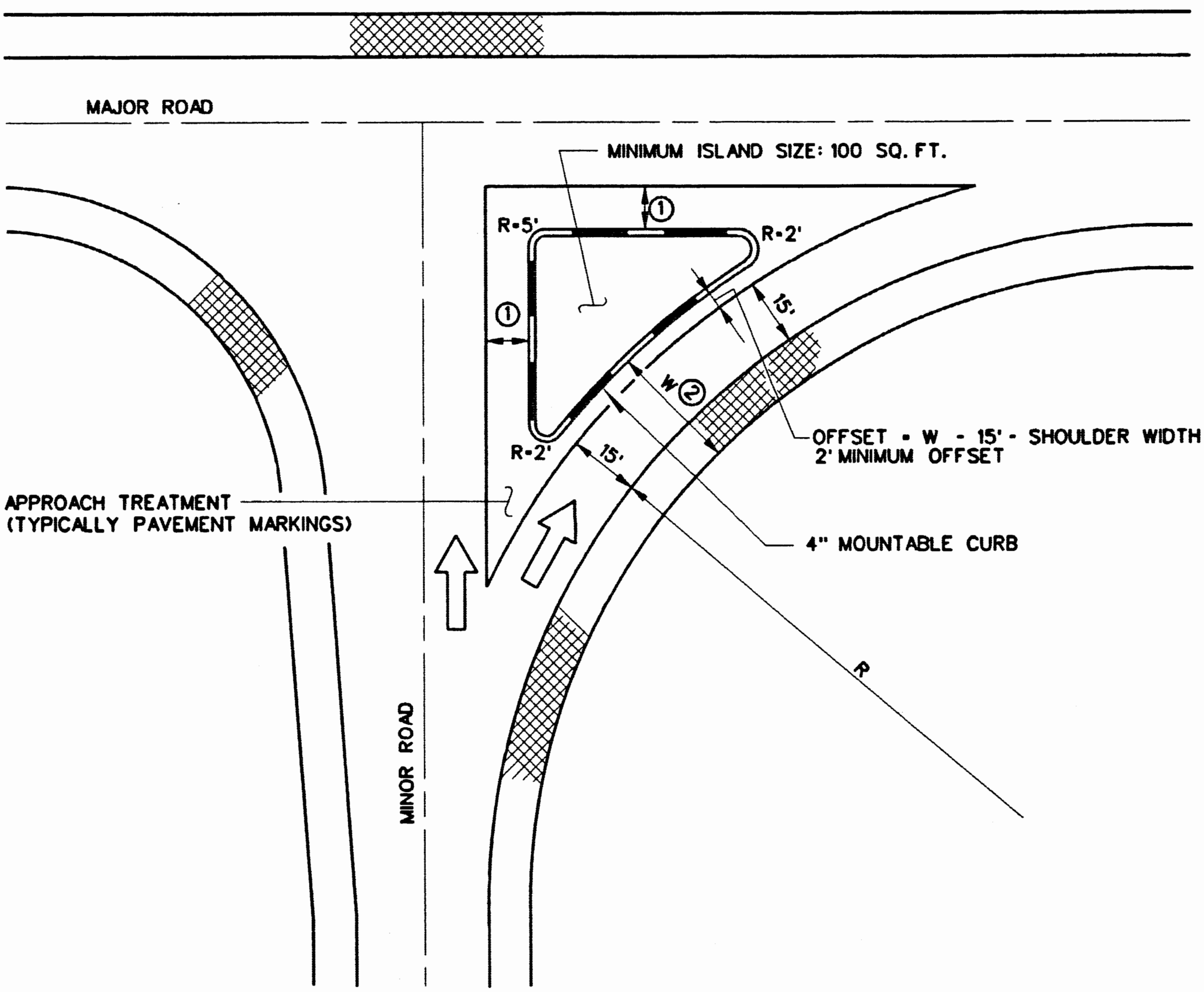
Notes:

① Where a right-turn lane is present in advance of the island, the island should be offset a minimum distance of 2′ from the outside edge of the right-turn lane.

② The turning roadway width (W) is based on the radius of the inner edge of pavement (R) and the turning path of the selected design vehicle. See Table 9.6A and apply the turning template of the design vehicle.

CURBED TRIANGULAR ISLAND
(Roadway Without Shoulders)

Figure 9.7A



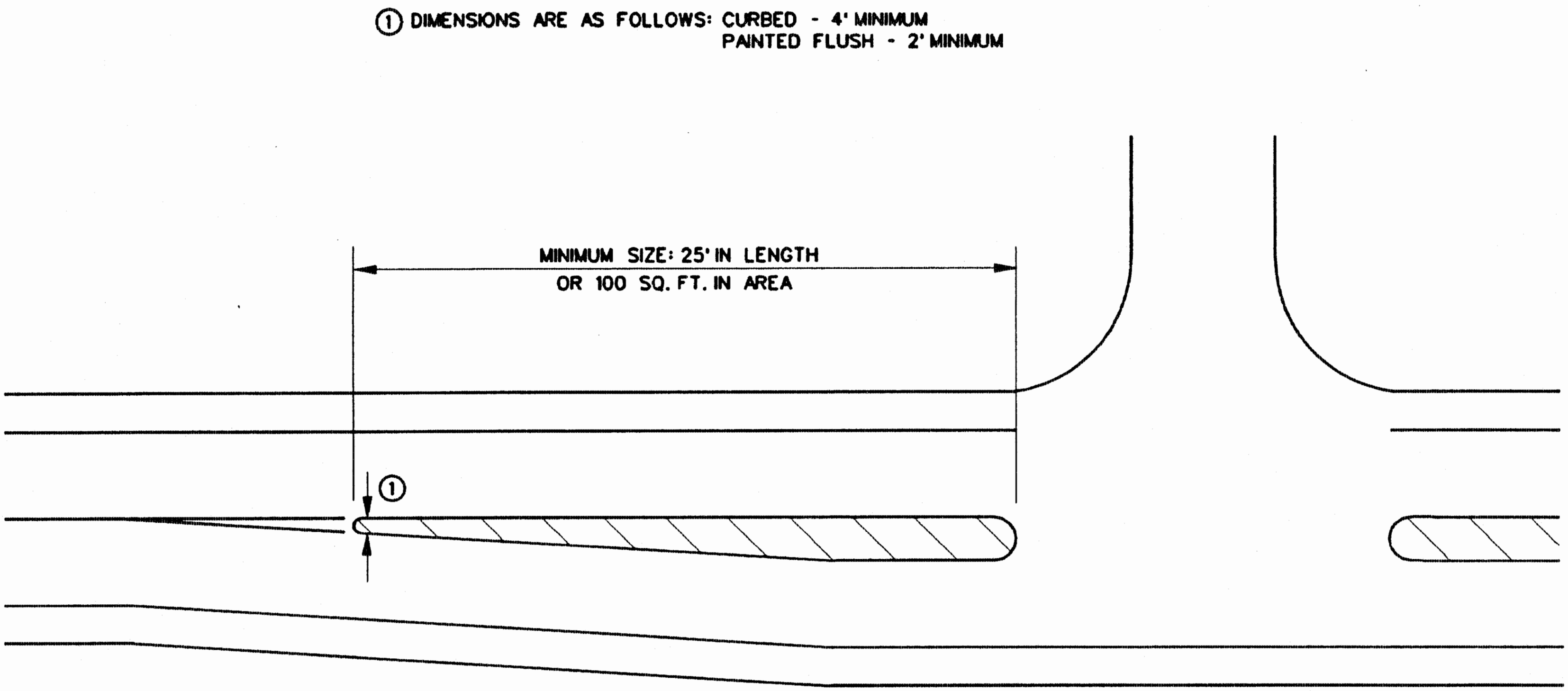
Notes:

① Where shoulders are present on the minor road or major road, the curbed island should be offset from the travel lane a distance equal to the shoulder width. Where a right-turn lane is present on the minor road or major road, the curbed island should be offset a minimum distance of 2′ from the outside edge of the right-turn lane.

② The turning roadway width (W) is based on the radius of the inner edge of pavement (R) and the turning path of the selected design vehicle. See Table 9.6A and apply the turning template of the design vehicle.

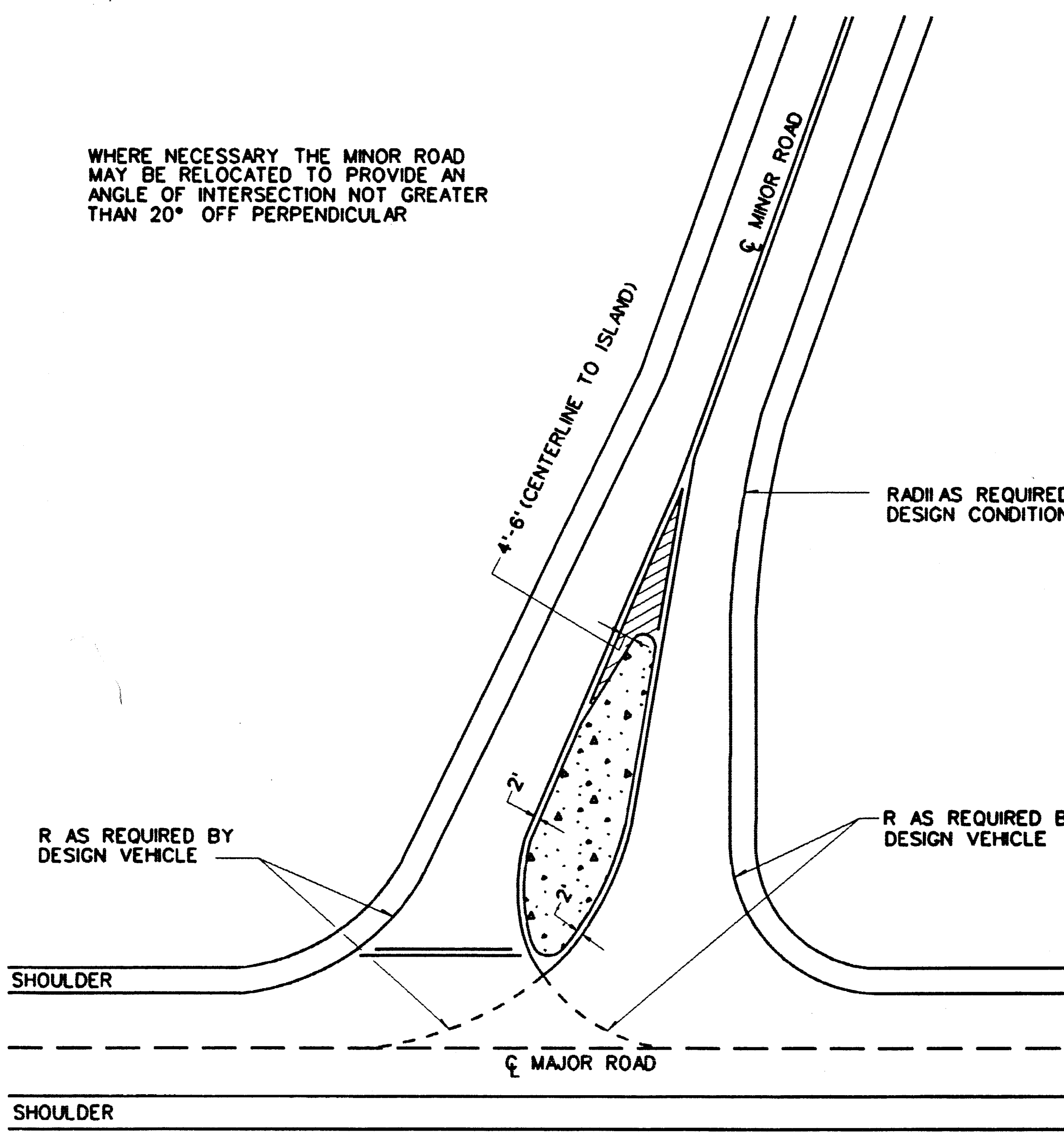
CURBED TRIANGULAR ISLAND
(Roadway With Shoulders)

Figure 9.7B



TYPICAL DIVISIONAL ISLAND

Figure 9.7C



EXAMPLE OF A CHANNELIZED INTERSECTION
(Rural)

Figure 9.7D

9.8 MEDIAN OPENINGS

9.8.1 Non-Freeways

9.8.1.1 Location/Spacing

Desirably, median openings will be provided on divided non-freeways at all public roads and major traffic generators. However, this may result in close intersection spacing which may impair the operation of the facility. The following recommended minimum spacings should be evaluated when determining the location for a median opening:

1. Rural Facilities. Median openings should be at least ½ mile apart and, desirably, 1 mile apart, subject to public service requirements as determined by an engineering study.

2. Urban Facilities. The desirable minimum spacing between median openings should be approximately ¼ mile. In addition, the spacing of median openings should also be long enough to allow the development of an exclusive left-turn lane with the proper length.

For both rural and urban facilities, the available sight distance in the vicinity of the median opening is a factor in the determination of its location. In addition, on some facilities, commercial establishments with heavy truck traffic may dictate the location of median openings.

9.8.1.2 Design

Figure 9.8A presents a general figure for the design of a median opening. Table 9.8A presents specific criteria for median opening design elements (length of opening, nose radii) based on median width and the selected design vehicle.

The following will apply to the design of a median opening:

1. Design Vehicle. The largest vehicle that will be making the turn with some frequency should be used as the design vehicle for median openings. The process for the selection of the design vehicle is the same as for a right-turning vehicle (see Sections 9.1 and 9.5).

2. Encroachment. The desirable design will be for the design vehicle to make the left turn and remain entirely within the through inside lane of the divided facility, and to come no closer than 2 ft to the inside curb or inside edge of pavement; i.e., there will be no encroachment into the through lane adjacent to the inside travel lane. However, it is acceptable for the design vehicle to occupy both travel lanes and the shoulder in its turn (see Figure 9.8A), if the shoulder paving is adequately strengthened.

3. Length of Opening. The length of a median opening should properly accommodate the turning path of the design vehicle. The minimum length is the largest of the following:

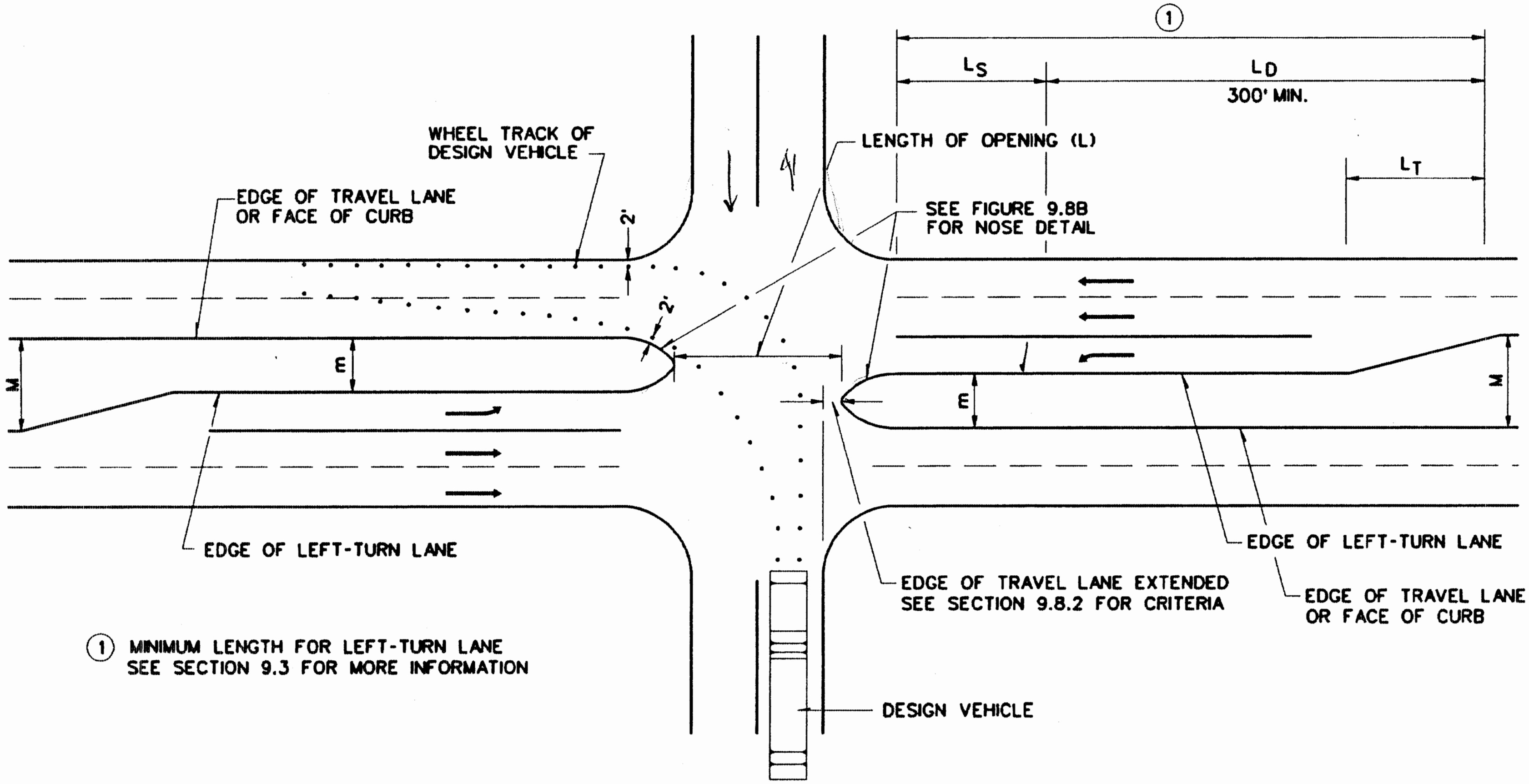
   a. approaching travelway width plus 8 ft,

   b. approaching travelway width plus the shoulder widths,

   c. the length based on the selected design vehicle from Table 9.8A, or

   d. 40 ft.

   Each median opening should be evaluated individually to determine the proper length of opening. The designer should consider the following factors in the evaluation:



Note: See Table 9.8A for criteria for L, and see discussion in Section 9.8.2 for minimum L criteria.

MEDIAN OPENING DESIGN

Figure 9.8A

Table 9.8A

MEDIAN OPENING DESIGN

Median Width (M) (ft)	Design Vehicle	Design Elements *			
		L	R_1	R_2	P
40′	WB-67 WB-114				
46′	WB-67 WB-114	TO BE PREPARED			
64′	WB-67 WB-114				

* See Figure 9.8A for application of L. See Figure 9.8B for application of R_1, R_2 and P.

Key:
- L = length of opening
- R_1 = control radius for bullet-nose design
- R_2 = nose radius for bullet-nose design
- P = see Figure 9.8B

Notes:

1. R_1, R_2 and P values in table are predicated on the presence of a left-turn bay as shown in Figure 9.8B.

2. See discussion in Section 9.8.2 for minimum L values.

e. Turning Templates. The designer should check the proposed design with the turning template for the selected design vehicle. Consideration should be given to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

f. 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 of the through travel lane (extended) and the median nose should be at least 4 ft.

g. Lane Alignment. The designer should ensure that lanes line up properly for crossing traffic.

h. Location of Crosswalks. Desirably, pedestrian crosswalks will intersect the median nose to provide some refuge for pedestrians. Therefore, median opening design should be coordinated with the location of crosswalks.

i. Traffic Control. The designer should coordinate with the Traffic Engineering Division for the design of the signing, striping and traffic control.

4. Median Nose Design. The shape of the nose at median openings is determined by the width of the median (M) or (m). The two basic types of median nose designs are the semicircular design and bullet-nose design. The following summarizes their usage:

   a. For medians up to 4 ft in width, there is little operational difference between the two designs.

   b. The semicircular design is generally acceptable for median widths up to 10 ft.

   c. For medians wider than 10 ft, the bullet-nose design should be used.

   d. As medians become successively wider, the minimum length of the median opening becomes the governing design control.

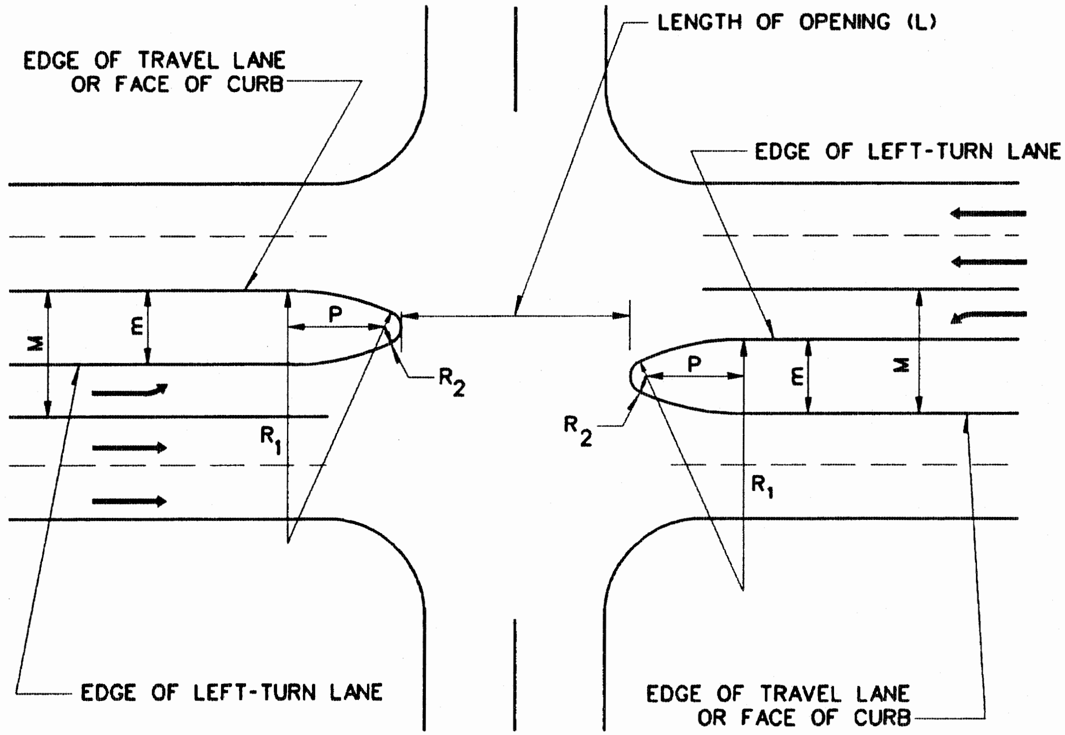
   For the bullet-nose design, a compound curvature arrangement should be used. Figure 9.8B provides the typical details for a median opening with a bullet-nose design.

5. U-turns. Median openings are sometimes used to accommodate U-turns on divided non-freeways. 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. However, if the needed median width is not available, a turning design vehicle may swing wide to avoid tracking beyond the available pavement (including a paved shoulder) on the opposite roadway. Figure 9.8C provides the minimum recommended median widths for U-turn maneuvers for various design vehicles and various levels of encroachment. On highways with heavy truck traffic (≥15% of ADT), the designer should desirably design for the WB-67 design vehicle.

6. Sight Distance. All median openings should be checked for applicable sight distance criteria. See Section 9.2.

9.8.2 Emergency Crossovers

On fully access-controlled freeways, median crossings are denied to the public. However,



M = median width measured between the two edges of the inside travel lanes

m = width of divider (raised or flush) remaining after the width of the left-turn lane (if present) has been subtracted from the median width (M)

R_1 = variable, based on design vehicle and median width (M). See Table 9.8A

R_2 = (m)/5, where a left-turn lane is present.

R_2 = M/5, where a left-turn lane is not present.

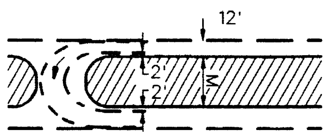
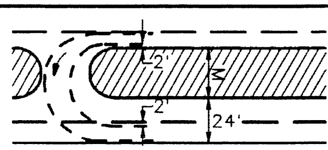
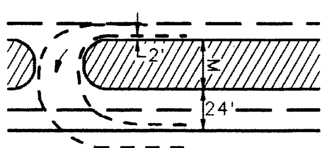
R_2 is typically rounded up to the next highest whole number.

L = length of median opening. See Table 9.8A and discussion in Section 9.8.2 for minimum L values.

P = as shown in figure. See Table 9.8A.

MEDIAN NOSE DESIGN

Figure 9.8B

Type of Maneuver		M - Min. width of median - ft for design vehicle						
		P	WB-40	SU	BUS	WB-50	WB-67	WB-114
		Length of design vehicle						
		19′	50′	30′	40′	55′	74′	118′
Inner Lane to Inner Lane		30	61	63	63	71	71	101
Inner Lane to Outer Lane		18	49	51	51	59	59	89
Inner Lane to Shoulder		8	39	41	41	49	49	79

Source: (1) Revised

Note: This figure assumes no left-turn lane and a 10′ surfaced shoulder (outside).

RECOMMENDED MINIMUM DESIGNS FOR U-TURNS

Figure 9.8C

occasional emergency crossovers are required to accommodate maintenance and emergency vehicles. The following should be considered:

1. Warrants and Location. Emergency crossovers on freeways are required to facilitate maintenance operations such as snow plowing and emergency vehicles. These crossovers should be placed well away from any mainline conflicts, such as interchanges. As a general guide, crossovers should be provided where the interchange spacing exceeds 5 miles. Between interchanges, emergency crossovers are spaced at 3-mile to 4-mile intervals. In addition, crossovers may be appropriate specifically at State lines and at boundary lines between maintenance districts.

2. Sight Distance. Because of the unexpected U-turn maneuver, good sight distance should be available when vehicles make U-turns on freeways. At a minimum, decision sight distance should be provided in both directions at a crossover. This would favor, for example, placing a crossover in a sag vertical curve.

3. Median Barriers. Emergency crossovers should be avoided where a median barrier is present. If a crossover must be provided, the barrier should be terminated as described in Chapter Eleven. The width of the opening should be approximately 25 ft - 30 ft.

4. Design. Figure 9.8C provides design details for U-turns on non-freeways which are also applicable to freeways. The median should be wide enough to allow for a SU design vehicle to turn from inner lane to inner lane.

9.9 DRIVEWAYS

9.9.1 General

The following sections provide general guidelines for the designer to use when constructing driveways on new or major reconstruction projects. For driveways on 3R projects or for driveway permits on existing highways, the user is referred to ODOT's *Policy on Driveway Regulations for Oklahoma Highways* and the *ODOT Standard Drawings*.

9.9.1.1 Driveway Types (Definitions)

The following provides definitions for the various driveway types:

1. Residential. Drives providing access to a:

   a. single family residence,

   b. duplex, or

   c. an apartment building containing not more than four dwelling units.

2. Commercial. Drives providing access to an:

   a. office, retail or institutional building;

   b. an apartment building having five or more dwelling units; or

   c. industrial plant, whose primary function it is to serve administrative or employee parking lots.

3. Industrial. Drives directly serving substantial numbers of truck movements and drives accessing to and from loading docks of an industrial facility, warehouse or truck terminal. Also, drives serving a centralized retail development, such as a community or regional shopping center, may have one or more driveways especially designed, signed and located to provide access for trucks. These may also be classified as industrial driveways.

9.9.1.2 Driveway Spacing and Corner Clearance

The following criteria will apply to driveway spacing and corner clearance:

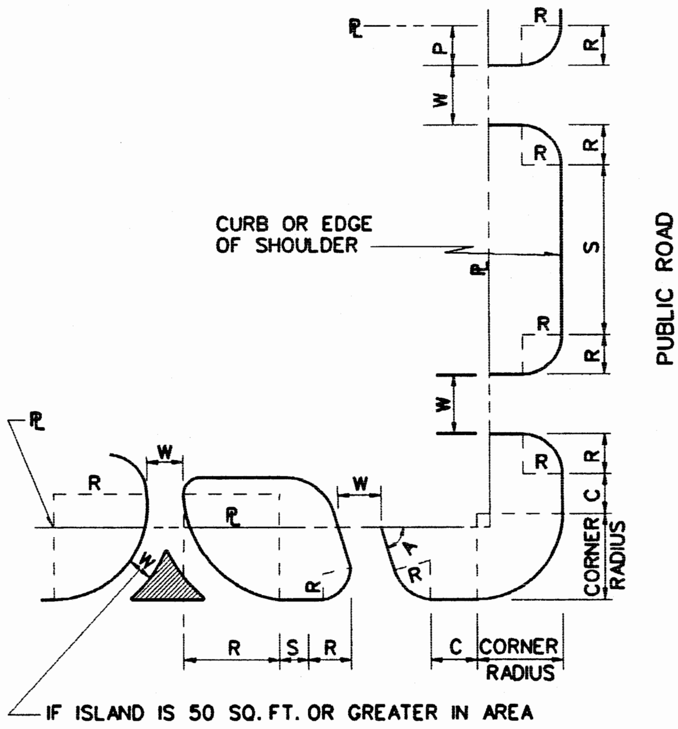
1. Guidelines. Table 9.9A presents criteria for driveway spacing and corner clearance. See Figure 9.9A for a definition of terms. These distances are measured along the curb or edge of pavement from the roadway end of the curb radius or flare. Desirable corner clearance is 40′ for rural areas and 20′ for urban areas.

Table 9.9A

DRIVEWAY SPACING AND CORNER CLEARANCE

Dimension	Term*	Type of Driveway		
		Residential	Commercial	Industrial
From Property Line	P	5′	15′	R
From Street Corner	C	5′	10′	10′
Between Driveways	S	3′	3′	10′

* See Figure 9.9A.



Source: (11)

Note: Driveway radius should be within the property line.

Key:

- R = Driveway radius
- W = Driveway width
- PL = Property line
- C = Corner clearance
- A = Driveway angle of intersection
- S = Spacing between two driveway radius points
- P = Spacing between driveway and property line radius point

DRIVEWAY DIMENSIONS

Figure 9.9A

2. Minimum. A driveway, including its entrance radius, should not be located within the radius of an intersection.

3. Future Control Accommodation. If these criteria cannot be met for properties in intersection corners, one possible solution is to relocate the driveway entrance from the major road to the minor road, if applicable.

4. Multiple Driveways. The number, arrangement and width of driveways are governed in part by the roadway frontage of abutting private property. The number of driveways provided should be the minimum number required to adequately serve the needs of the adjacent property. A frontage of 50 ft or less is generally limited to one driveway. Normally, not more than two driveway accesses are provided to any single property tract or business from any single roadway. Exceptions may be made where the frontage exceeds 600 to 1000 ft. In some cases, a single driveway serving two adjacent properties may best accommodate access to the properties. Joint-use driveways must be agreed to by both property owners.

   Where there are several commercial or residential properties, each with relatively limited frontage, or where there is the probability of such development, the designer may consider providing a frontage road for the several driveways. This would reduce the number of access points to the major roadway.

9.9.1.3 Driveway Sight Distance

Section 9.2 discusses intersection sight distance (ISD) criteria for intersections with public roads. Desirably, these criteria will also apply to sight distance at driveways. However, for driveways with low volumes, it may not be cost-effective to provide the desirable ISD values. Where the minimum ISD criteria cannot be met, directional driveways may be a solution. Another potential countermeasure is to prohibit turns for re-entry to the main road or to provide islands to discourage turns which may be a safety problem.

In general, the designer should check for sight obstructions in the vicinity of the driveway entrance (e.g., large trees, hedgerows) which may cause problems. To perform the check, it is reasonable to assume an eye location of approximately 10 ft from the edge of travel lane.

9.9.1.4 Auxiliary Lanes

Deceleration and acceleration lanes should be considered at high-volume driveway entrances, especially on high-speed, high-volume arterials. Section 9.3 further discusses the design and warrants for auxiliary lanes, and these also apply to high-volume driveways. In addition to traffic-volume considerations, it may be warranted to provide a right-turn lane into the driveway if the change in grade is abrupt at the driveway entrance.

9.9.2 Driveway Design Criteria

1. Tables. Table 9.9B presents design criteria for driveways. These apply to both urban and rural driveways.

2. Typical Driveway Profile Figures. Figure 9.9B presents design criteria for driveways where no curbs exist; Figure 9.9C applies to driveways where curbs are present.

3. Typical Driveway Plan Views. Typical plan views of driveway entrances are illustrated in ODOT's *Policy on Driveway Regulations for Oklahoma Highways* and in the *ODOT Standard Drawings*.

Table 9.9B

RECOMMENDED DESIGN CRITERIA FOR DRIVEWAYS

Driveway Design Element		Driveway Type	Functional Classification of Intersecting Road		
			Arterial	Collector	Local Road
Design Vehicle (1)		Residential	P	P	P
		Commercial	WB-50	SU	SU
		Industrial	WB-67	WB-50	WB-50
Turning Radii (R) (2)		Residential	15′	10′-15′	10′-15′
		Commercial/ Industrial	30′-50′	20′-30′	15′-30′
Width (W) (2)		Residential	12′-20′	12′-20′	12′-20′
		Commercial/ Industrial	35′ Maximum	35′ Maximum	35′ Maximum
Recommended Grades on Driveway Proper (G)		Residential	Desirable: 0 - 10% Maximum: 15%		
		Commercial/ Industrial	Desirable: 0 - 5% Maximum: 8%		
Change in Grade Without Vertical Curve (ΔG) (3)		All	Recommended: 8% or less	Recommended: 9% or less	Recommended: 15% or less
Driveway Side Slopes (without curbs)	Within Clear Zone (4)	All	See Chapter Eleven	See Chapter Eleven	See Chapter Eleven
	Outside Clear Zone	All	Maximum: 3:1	Maximum: 3:1	Maximum: 3:1

Note: See Figure 9.9A for application in plan view. See Figures 9.9B and 9.9C for application in profile view.

Table 9.9B

DRIVEWAY DESIGN

Footnotes

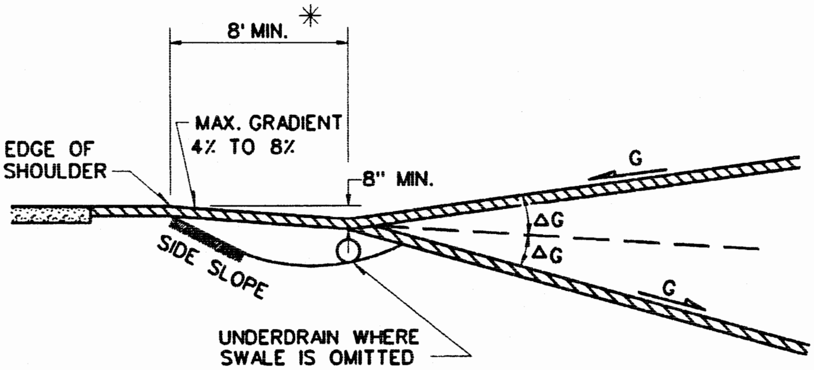
1. Design Vehicle. The design vehicle will be selected on the basis of the property served and not on the predominate vehicle in the traffic stream of the main road.

2. Turning Radii/Width. The criteria in the table are intended to provide a typical design for the driveway. In special cases or where the WB-67 or WB-114 is the selected design vehicle, driveway width may have to exceed 35 ft. The design should be based on the procedure and criteria discussed in Section 9.5 for turning radii designs at intersections. The application of the turning template of the design vehicle will be the determining factor for turning radii and driveway width. See also the *ODOT Standard Drawings* for application of turning radii and width criteria.

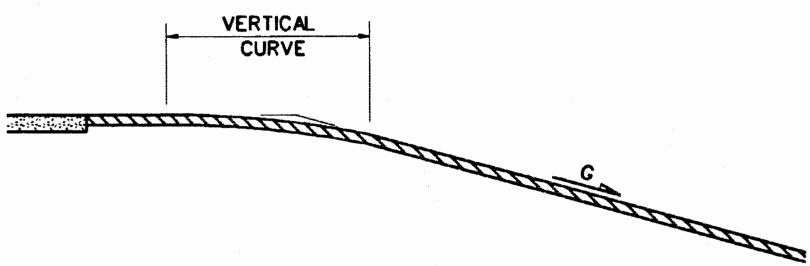
   The designer may need to increase the driveway width for two-way operation, depending upon the likelihood that vehicles will be entering and exiting from the driveway simultaneously. See ODOT's *Policy on Driveway Regulations for Oklahoma Highways* for recommendations on angle driveways.

3. Change in Grade Without Vertical Curve (ΔG). Desirably, vertical curves will be used to connect slopes of different grades. Where the ΔG criteria in the table are exceeded, the designer should insert a vertical curve to prevent the vehicle from bottoming out. To prevent drag, vertical curves should avoid a hump or dip greater than 6 inches within a wheel base length of 10 ft. To prevent center or overhang drag, with some allowance for load and bounce, crest vertical curves should not exceed a 3¼-inch hump in a 10-ft chord, and sag vertical curves should not exceed a 2-inch depression in a 10-ft chord.

4. Side Slopes. Chapter Eleven presents criteria for transverse slopes. In summary, these are 1) 10:1 for high-volume, multilane divided highways with a design speed $V \geq 50$ mph, and 2) 6:1 on other facilities. However, on urban facilities ($V \leq 45$ mph) and on rural facilities ($V \leq 40$ mph), transverse slopes steeper than 6:1 may be considered. Where a barrier is shielding the slope, transverse slopes steeper than 3:1 are acceptable. Note that clear zones are determined based on the direction of travel which is exposed to the drainage pipe and 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.



DRIVEWAY WITHOUT VERTICAL CURVE



DRIVEWAY WITH VERTICAL CURVE

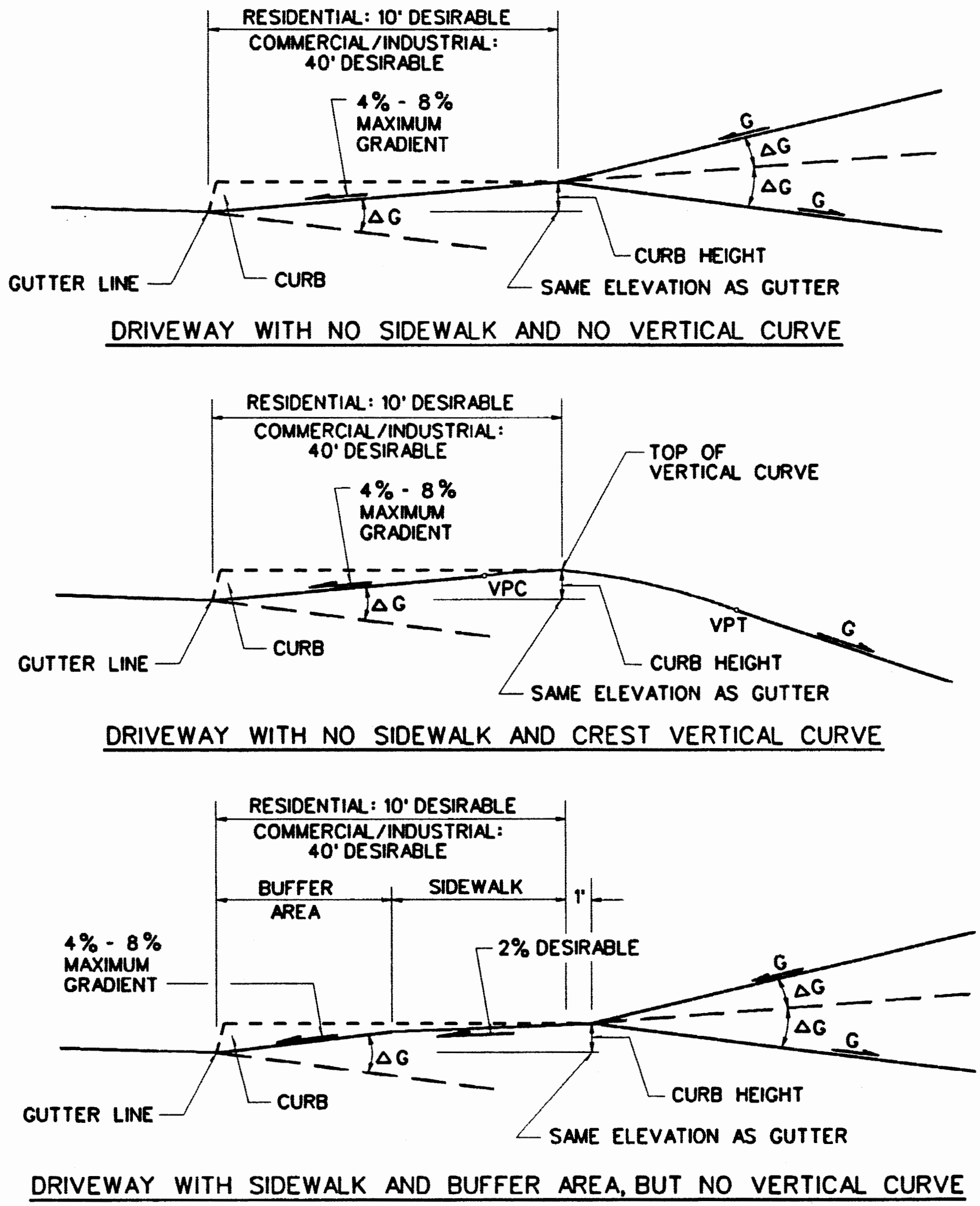
Notes:

1. See Table 9.9B for criteria on G, ΔG and vertical curves.

2. See *ODOT Standard Drawings* for more detail on driveway design and References 10 and 11.

TYPICAL DRIVEWAY PROFILES
(No Curbs)

Figure 9.9B



Notes:

1. See Table 9.9B for criteria on G, ΔG and vertical curves.
2. See *ODOT Standard Drawings* for more detail on driveway design and References 10 and 11.
3. Where practical, maintain the typical sidewalk cross slope through the driveway section. If necessary, the sidewalk will be warped to fit the profile of the driveway, unless the sidewalk is on an accessible route. See Chapter Seventeen for criteria for handicapped accessibility.

TYPICAL DRIVEWAY PROFILE (With Curbs)

Figure 9.9C

9.10 RAILROAD/HIGHWAY GRADE CROSSINGS

The following presents geometric design criteria for highway crossings at-grade with railroads. For other design considerations (e.g., traffic control devices, railroad crossing surfaces), the designer should coordinate with the Traffic Engineering Division. In addition, the designer should refer to Reference (12) for more information.

9.10.1 Sight Distance

Two sight distance applications must be addressed at railroads:

1. Case I — Allowing an approaching vehicle to safely cross or stop.
2. Case II — Allowing a stopped vehicle to safely cross the railroad tracks.

9.10.1.1 Case I

Figure 9.10A presents the Case I sight distance criteria for railroad/highway grade crossings. These criteria are important at crossings with only passive control devices (crossbucks, pavement markings, advance warning signs). Where practical, these criteria should also be met at crossings with active control devices (flashing lights, automatic gates). The Case I sight distances allow an approaching vehicle to either:

1. decide it is safe to cross the tracks without stopping, or
2. safely stop the vehicle before reaching the tracks.

Figure 9.10A presents the equations for calculating the Case I sight distance criteria. Table 9.10A presents the Case I criteria based on anticipated maximum train speed and the highway design speed. These criteria are appropriate for railroad/highway intersections at 90 degrees; adjustments should be made for skewed intersections. The distances in Table 9.10A.

are also based on an approaching level grade (i.e., flatter than 3%). For grades 3% or steeper, the expression $V_v^2/30f$ in the equation should be replaced with the following:

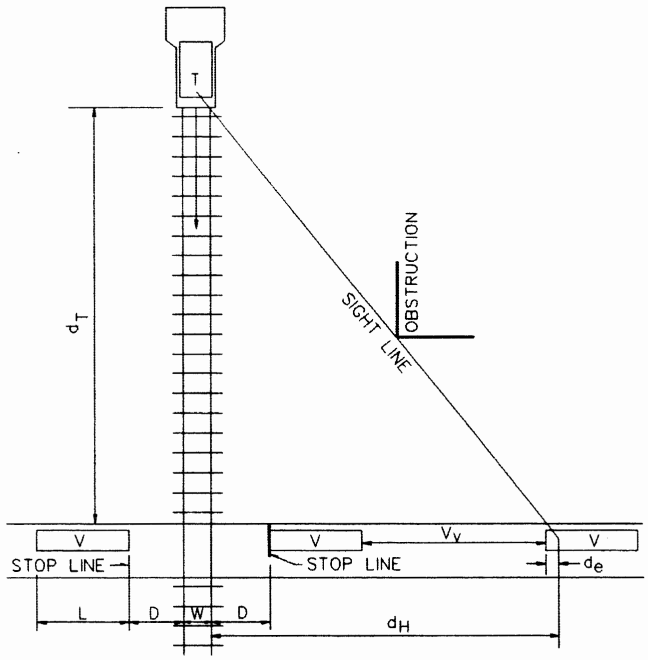
$V_v^2/30\ (f + G)$,

where G = the highway grade in percent, and the d_H and d_T values recalculated. Note that corrections are only made for downgrades approaching the railroad tracks; for level grades or upgrades, use Table 9.10A.

The designer should note that the Case I sight distances may be difficult to attain the field. Where these distances are not available, the following presents possible 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.

Any of these countermeasures requires coordination with the Traffic Engineering Division.



Source: (1) Revised

$$d_H = 1.47\ V_v t + \frac{V_V^2}{30f} + D + d_e$$

$$d_T = \frac{V_T}{V_v}\left[(1.47)\ V_v t\ \frac{V_V^2}{30f} + 2D + L + W)\right]$$

where:

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, ft

d_T = sight distance leg along the railroad tracks to permit the maneuvers described for d_H, ft

V_v = velocity of the vehicle, mph

V_T = velocity of the train, mph

t = perception/reaction time (assumed to be 2.5 sec)

f = coefficient of friction (assumed to be same values used for stopping sight distance (Chapter Five)

D = distance from the stop line or front of the vehicle to the nearest rail (assumed to be 15 ft)

d_e = distance from the driver to the front of the vehicle (assumed to be 10 ft)

L = length of vehicle (assumed to be 65 ft)

W = distance between outer rails; for a single track, this value is 5 ft

CASE I:
APPROACHING VEHICLE TO SAFELY CROSS OR STOP AT RAILROAD CROSSING

Figure 9.10A

Table 9.10A

CASE I
(Approaching Vehicle)

Train Speed (mph)	Design Speed (mph)						
	10	20	30	40	50	60	70
	Distance (d_T) Along Railroad From Crossing						
10	145	103	99	103	112	122	134
20	290	207	197	207	224	245	269
30	435	310	296	310	337	367	403
40	580	413	394	413	449	489	537
50	725	517	493	517	561	611	671
60	870	620	591	620	673	734	806
70	1,015	723	690	723	786	856	940
80	1,160	827	789	827	898	978	1,074
90	1,305	930	887	930	1,010	1,101	1,209
	Distance Along Highway from Crossing, d_H(ft)						
	69	132	221	338	486	659	865

Source: (12) Revised

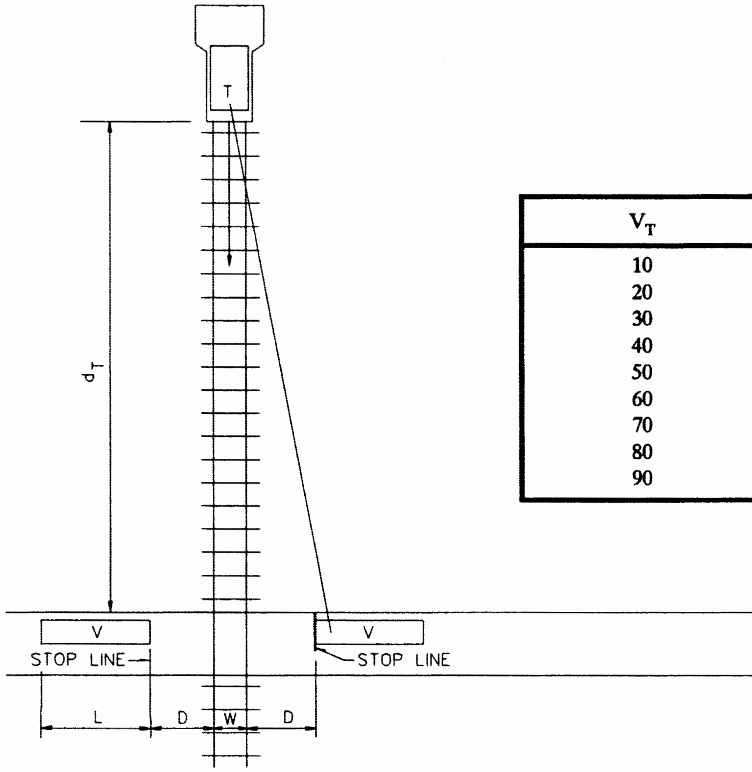
9.10.1.2 Case II

Figure 9.10B presents the Case II sight distance criteria and their derivation at railroad/highway grade crossings. The figure also presents the numerical values which apply for the anticipated maximum train speed. These criteria should be available at all crossings.

The designer should note that the Case II criteria are predicated on a 90-degree intersection between the railroad and highway; adjustments for skewed intersections are necessary. These distances are also predicated on the time needed for the design vehicle to cross a single set of tracks (i.e., W = 5 ft). Where more than one set of tracks is present, the designer should use the applicable W value and recalculate the Case II distances.

9.10.2 Horizontal Alignment

Where practical, the alignment of the highway and railroad crossing should intersect at an angle of 90 degrees, 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 Six for ODOT criteria on horizontal alignment for highways.



V_T	d_T(ft)
10	240
20	480
30	719
40	959
50	1,200
60	1,439
70	1,679
80	1,918
90	2,158

Source: (1) Revised

$$d_T = 1.47\ V_T \left[\frac{V_G}{a_1} + \frac{L + 2D + W + -da + J}{V_G} \right]$$

where:

d_T = sight distance along railroad tracks, ft

V_T = velocity of train, mph

V_G = maximum speed of vehicle in first gear (assumed to be 8.8 fps)

a_1 = acceleration of vehicle in first gear (assumed to be 1.47 ft/sec^2)

L = length of vehicle (assumed to be 65 ft)

D = distance from the stop line to the nearest rail (assumed to be 15 ft)

J = perception/reaction time (assumed to be 2.0 seconds)

W = distance between outer rails; for a single track, this value is 5 ft

$d_a = \frac{V_G^2}{2a_1}$, or distance vehicle travels while accelerating to maximum speed in first gear

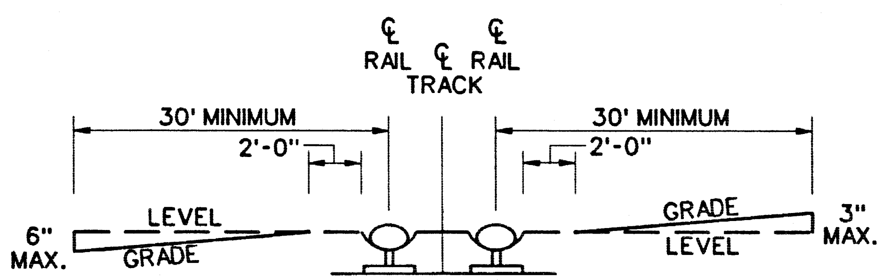
CASE II: DEPARTURE OF VEHICLE FROM STOPPED POSITION TO CROSS SINGLE RAILROAD TRACK

Figure 9.10B

9.10.3 Vertical Alignment

Desirably, the highway will be relatively level where it crosses the railroad. Where vertical curves are provided, they should meet the ODOT criteria for vertical alignment presented in Chapter Seven.

Figure 9.10C 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.



Source: (12)

PROFILE AT RAILROAD/HIGHWAY GRADE CROSSINGS

Figure 9.10C

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Chapter Ten

Interchanges

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Chapter Ten

INTERCHANGES

10.1 GENERAL

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways on different levels.

10.1.1 Warrants

Although an interchange is a high-level compromise to intersection problems, its high cost and environmental impact require that interchanges be used only after careful consideration of its benefits. The following discusses procedures and warrants used by ODOT.

10.1.1.1 ODOT Procedures

In general, the ODOT Planning Division determines the need for and location of interchanges. The assessment is based on a consideration of the factors discussed in Section 10.1.1.2 and of the socio-economic-political realities. Once the Planning Division has determined the interchange location, the Geometric Design Branch is responsible for selecting the interchange type. The Rural or Urban Design Division and the Geometric Design Branch work together to develop the detailed interchange design.

10.1.1.2 Guidelines

Because of the great variance in specific site conditions, ODOT has not adopted specific interchange warrants. Following are some general guidelines for interchanges or grade separations:

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 warrant 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, intersection treatments with public roads will be with 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 Comment #'s 2 through 10.

2. Functional Classification. Interchanges should 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 should be provided if practical.

3. Congestion. An interchange may be warranted 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.

4. Safety. A consideration of strictly the accident reduction benefits of an interchange may warrant its selection at an at-grade intersection. Section 10.1.2 provides additional information on various safety considerations relative to interchange selection.

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 significantly reduce the travel time when compared to at-grade intersections. Therefore, if an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then the interchange may be warranted.

7. Traffic Volumes. A traffic volume warrant is the most tangible of any interchange warrant. Although ODOT has not adopted specific numbers which, when exceeded, would demand an interchange, it is still an important factor. For example, the point at which volumes for an at-grade intersection exceed capacity may warrant an interchange, if the at-grade intersection cannot be practically upgraded. In addition, other factors, such as costs, right-of-way and environmental concerns, need to be considered.

8. Interchange Spacing. When interchanges are spaced farther apart, freeway operations are improved. Spacing of urban interchanges should not be less than 1 mile. This should allow for 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 for adequate advance and turnoff signing. In urban areas, a spacing of less than 1 mile may be developed by grade-separated ramps or by collector-distributor roads.

   In rural areas, interchanges should be spaced not less than 3 miles apart. Desirably, the average spacing between interchanges throughout a highway corridor in suburban areas should be not less than 4 miles and, in rural areas, not less than 8 miles.

9. Access. Interchanges may be warranted in areas where there is limited access available from other sources, and the freeway is the only facility that can practically serve the area.

10. Existing Facilities. In general, ODOT and FHWA discourage providing new interchanges on existing access-controlled facilities. New access should be based upon the overall system needs and site limitations. In general, a new interchange on an existing facility will only be considered if its introduction will not violate ODOT criteria for the design of interchanges as presented in Chapter Ten. This includes:

    a. level of service criteria;

    b. interchange spacing guidelines;

    c. sufficient road-user benefits based on traffic volumes, accident history, etc.;

d. sufficient space available to meet specific geometric design criteria (e.g., length of acceleration/deceleration, ramp design speed, weaving distances);

e. access control around interchanges; and

f. reasonable construction costs.

Typically, access to serve a single user (e.g., a shopping center) is not allowed.

10.1.2 Safety Considerations

Safety is an important consideration in the selection and design of an interchange. After many years of operating experience and safety evaluations, certain practices are considered less desirable at interchanges nationwide. The following summarizes several major safety considerations:

1. Exit Points. Many interchanges have been built with exit points which could not clearly be seen by approaching drivers. Decision sight distance should be provided where practical at freeway exits, and the pavement surface should desirably be used for the height of object (0.0 ft). A 6-inch height of object is acceptable. See Section 10.3 for the application of decision sight distance to freeway exits. Proper advance signing of exits is also essential.

2. Exit Speed Changes. Freeway exits should provide sufficient distance for a safe deceleration from the freeway design speed to the design speed of the first governing geometric feature on the ramp.

3. Merges. Rear-end collisions on entrance merges onto a freeway may result from a driver attempting the complicated maneuver of simultaneously searching for a gap in the mainline traffic stream and watching for vehicles in front. An acceleration distance of sufficient length should be provided to allow a merging vehicle to attain speed and find a sufficient gap to merge into.

4. Driver Expectancy. To avoid driver confusion, interchanges should be designed to conform to the principles of driver expectation. Left-hand merges are less desirable. It is difficult for a driver entering from a ramp to safely merge with the high-speed left lane on the mainline. Thus, use of left exits and entrances are discouraged, because they are not consistent with the concept of driver expectancy when they are mixed with right-hand entrances and exits. Other driver expectancy issues include avoiding the placement of exits beyond structures and avoiding the placement of exits in line with a freeway tangent section.

5. Fixed-Objects. Because of traffic operations at interchanges, a number of fixed objects may be located within interchanges, such as signs at exit gores or bridge piers and rails. These should be removed, where practical, made breakaway or shielded with barriers or crash cushions.

6. Wrong-Way Entrances. In almost all cases, wrong-way maneuvers originate at interchanges. Some simply cannot be avoided, but many result from driver confusion due to poor visibility, confusing ramp arrangement or inadequate signing. The interchange design must attempt to minimize wrong-way possibilities.

7. Weaving. Areas of vehicular weaving may create a high demand on driver skills and attentiveness. Where practical, interchanges should be designed with few

weaving areas or, as an alternative, with weaving areas removed from the highway mainline (e.g., with collector-distributor roads).

Figure 10.1A and Table 10.1A summarize the results of accident experience at interchanges from the *Interstate System Accident Research Study II* by FHWA. A general observation is that, the higher the level of confusion associated with the particular maneuver or the interchange as a whole, the higher the accident rate. It also illustrates the safety advantages for specific designs, such as the value of collector-distributor (C-D) roads. This data can be used when selecting and designing a new interchange or reconstructing an existing one. For example, the designer can estimate that, if an existing cloverleaf without C-D roads is reconstructed to include them, the ramps may realize approximately a 38% reduction in accidents and the loops approximately a 51% reduction. This alone may justify the inclusion of a C-D road at the reconstructed interchange.

10.1.3 Traffic Operational Factors

Several traffic operational factors are important in the design of an interchange.

10.1.3.1 Basic Number of Lanes

The basic number of lanes is the minimum number of lanes exclusive of auxiliary lanes designated and maintained over a significant length of a route based on the overall operational needs of that section. The basic number of lanes should remain constant over significant distances. For example, a lane should not be dropped at the exit of a diamond interchange and then added at the downstream entrance simply because the traffic volume between the exit and entrance drops significantly. Likewise, a basic lane should not be dropped between closely spaced interchanges simply because the estimated traffic volume in that short section of highway does not warrant the higher number of lanes.

10.1.3.2 Lane Balance

Lane balance refers to certain principles which apply at freeway exits and entrances:

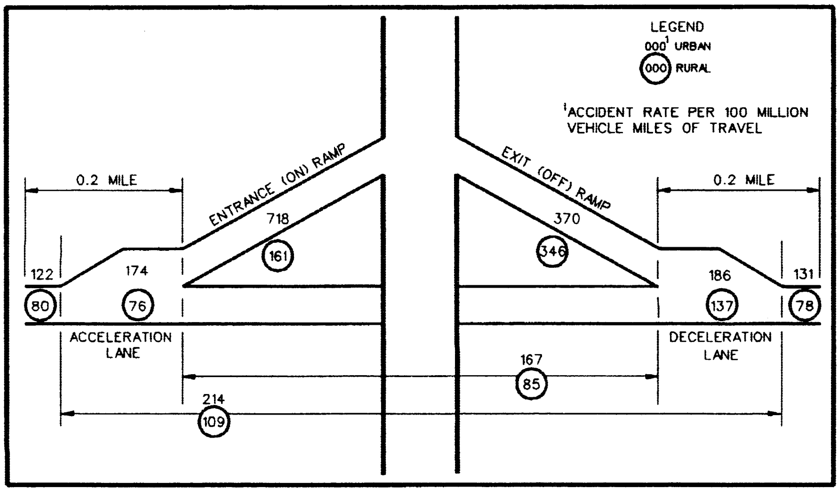
1. Exits. At exits the number of approach lanes on the highway should equal the sum of the number of mainline lanes beyond the exit plus the number of exiting lanes minus one.

2. Entrances. At entrances the number of lanes beyond the merging of the two traffic streams should be not less than the sum of the approaching lanes minus one.

For example, dropping two lanes at a 2-lane exit ramp would violate the principle of lane balance. One lane should provide the option of remaining on the freeway. Lane balance would also prohibit immediately merging both lanes of a 2-lane entrance ramp into a highway mainline without the addition of at least one additional lane beyond the entrance ramp. Figure 10.1B illustrates how to coordinate lane balance and the basic number of lanes at an interchange. Figure 10.1B also illustrates how to achieve lane balance at the merging and diverging points of branch connections.

10.1.3.3 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principle of lane balance, to increase capacity, to accommodate weaving or to accommodate entering and exiting vehicles. An auxiliary lane may be dropped at an exit if properly signed and designed. The following state-



ACCIDENT RATES AT INTERCHANGE EXITS AND ENTRANCES

Figure 10.1A

Table 10.1A

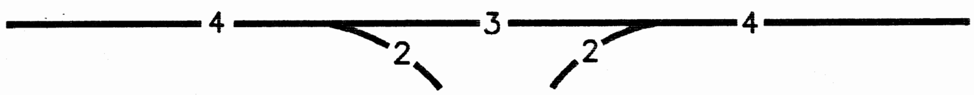
FREEWAY RAMP ACCIDENT RATES

RAMP TYPE	ACCIDENT RATE[1]		
	ON	OFF	ON & OFF
1. DIAMOND RAMPS	40	67	53
2. CLOVERLEAF RAMPS WITH COLLECTOR-DISTRIBUTOR ROADS[2]	45	62	61
3. DIRECT CONNECTIONS	50	91	67
4. CLOVERLEAF LOOPS WITH COLLECTOR-DISTRIBUTOR ROADS[2]	38	40	69
5. BUTTONHOOK RAMPS	64	96	80
6. LOOPS WITHOUT COLLECTOR-DISTRIBUTOR ROADS	78	88	83
7. CLOVERLEAF RAMPS WITHOUT COLLECTOR-DISTRIBUTOR ROADS	72	95	84
8. TRUMPET RAMPS	84	85	85
9. SCISSORS RAMPS	88	148	128
10. LEFT SIDE RAMPS	93	219	191
AVERAGE	59	95	79

Source: (2)

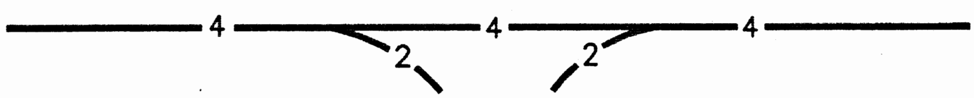
[1] ACCIDENTS PER HUNDRED MILLION VEHICLES

[2] ONLY THE (ON & OFF) RATE INCLUDES THE ACCIDENTS OCCURRING ON THE COLLECTOR-DISTRIBUTOR ROADS



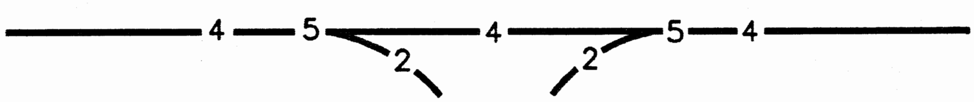
LANE BALANCE BUT NO COMPLIANCE WITH BASIC NUMBER OF LANES

-A-



NO LANE BALANCE BUT COMPLIANCE WITH BASIC NUMBER OF LANES

-B-



COMPLIANCE WITH BOTH LANE BALANCE AND BASIC NUMBER OF LANES

-C-

Source: (1)

COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES

Figure 10.1B

ments apply to the use of an auxiliary lane within or near interchanges:

1. Within Interchange. Figure 10.1C provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design will depend upon traffic volumes for the exiting, entering and through movements.

2. Between Interchanges. Where interchanges are closely spaced and an auxiliary lane is warranted at an entrance or exit, the designer should consider connecting the lane to the exit of the downstream interchange or entrance of the upstream interchange.

Design details for exits and entrances are provided in Section 10.3, and design details for lane drops are provided in Section 10.5.

10.1.3.4 Route Continuity

All highways with interchanges are designated by route number. The major route should desirably flow continuously through the interchange. The driver should not be required to change lanes nor to exit to remain on the major route. Route continuity is consistent with driver expectancy, simplifies signing and reduces the decision demands on the driver.

Interchange configurations should not necessarily favor the heavier traffic movement. There may occasionally be sites where it is advisable to design the interchange to provide route continuity despite the traffic volume movements.

10.1.3.5 Signing and Pavement Marking

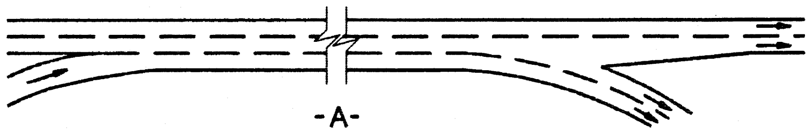
Proper interchange operations depend partially on the compatibility between its geometric design and the traffic control devices at the interchange. The proper application of signs and pavement markings increases the clarity of paths to be followed, and the safety and operational efficiency. The logistics of signing along a highway segment also impacts the minimum acceptable spacing between adjacent interchanges. The Traffic Engineering Division will determine the use of traffic control devices at interchanges.

10.1.3.6 Distance Between Successive Freeway/Ramp Junctions

Frequently, successive freeway/ramp junctions may need to be placed relatively close to each other, especially in urban areas. The distance between the junction should provide for vehicular maneuvering, signing and capacity. Figure 10.1D provides guidelines for recommended distances for spacings for various freeway/ramp junctions. The ramp-pair combinations are entrance followed by entrance (EN-EN), exit followed by exit (EX-EX), exit followed by entrance (EX-EN), entrance followed by exit (EN-EX) (except for cloverleaf loop ramps) and turning roadways. The criteria in Figure 10.1D are appropriate for the initial planning stages of interchange location. The final decision on the spacing between freeway/ ramp junctions will be based on the level-of-service criteria and on the detailed capacity methodology in the *Highway Capacity Manual*.

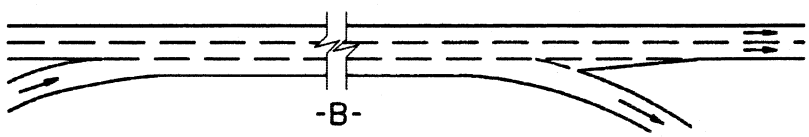
10.1.4 General Design Considerations

The following lists several basic design issues the designer should consider during the interchange analysis:



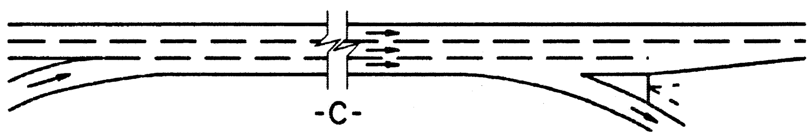
-A-

AUXILIARY LANE DROPPED ON EXIT RAMP



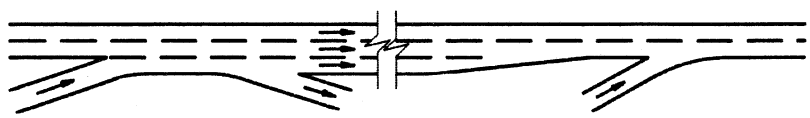
-B-

AUXILIARY LANE BETWEEN CLOVERLEAF LOOPS OR CLOSELY SPACED INTERCHANGES DROPPED ON SINGLE LANE



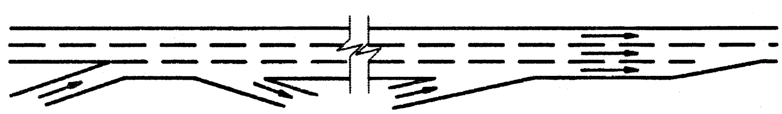
-C-

AUXILIARY LANE DROPPED AT PHYSICAL NOSE



-D-

AUXILIARY LANE DROPPED WITHIN AN INTERCHANGE



-E-

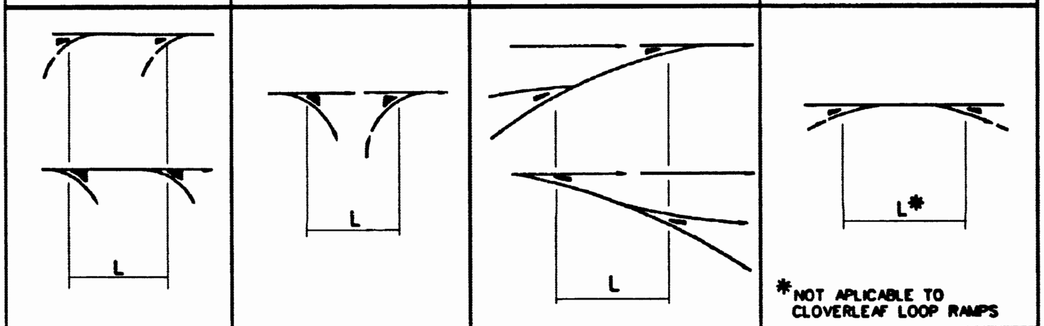
AUXILIARY LANE DROPPED BEYOND AN INTERCHANGE

Source: (1)

AUXILIARY LANES

Figure 10.1C

EN-EN OR EX-EX		EX-EN		TURNING ROADWAYS		EN-EX (WEAVING)			



						SYSTEM TO SERVICE INTERCHANGE		SERVICE TO SERVICE INTERCHANGE	
FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR	SYSTEM INTER-CHANGE	SERVICE INTER-CHANGE	FULL FWY.	CDR OR FDR	FULL FWY.	CDR OR FDR
MINIMUM LENGTHS MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS									
1000	800	500	400	800	600	2000	1600	1600	1000

NOTE: FDR - FREEWAY DISTRIBUTOR ROAD
CDR - COLLECTOR-DISTRIBUTOR ROAD
EN - ENTRANCE
EX - EXIT

The recommendations are based on operational experience and need for flexibility and adequate signing. They should be checked in accordance with the procedure outlined in the *Highway Capacity Manual*. The larger of the values is suggested for use. Also, a procedure for measuring the length of the weaving section is given in Chapter 4 of the *Highway Capacity Manual.*

Source: (1)

RECOMMENDED MINIMUM RAMP TERMINAL SPACING

Figure 10.1D

1. Design Year. The design year for the minor road intersecting the freeway should be the same as used for the freeway. The termination of other roads and streets in the area may generate a significant increase of traffic on the crossing facility.

2. Over versus Under. The decision on whether the freeway should go over or under the cross road is normally dictated by topography. If the topography does not favor one over the other, the following factors can be used as a guide to determine which highway should cross over the other:

   a. The designer needs to consider which alternative will be more cost effective to construct. Some elements to consider are the amount of fill, grading, span lengths, angle of skew, gradients, sight distances, geometrics, constructability and traffic control.

   b. One benefit of the cross road going over the freeway is that this may improve the ramp gradients. As drivers exit the freeway, they will normally tend to slow down going up an exit ramp and speed up going down an entrance ramp.

   c. The alternative which provides the highest design level for the major road should be selected. Typically, the crossing road has a lower design speed; therefore, the designer can advantageously consider that the minor road usually has steeper gradients, lesser widths, reduced vertical clearance requirements, etc.

   d. If any crossings and/or structures are planned for a future date, the mainline should go under these future crossings. Overpasses are easier to install and will be less disruptive to the major road when they are constructed in the future.

3. Horizontal Distance. Figure 10.1E can be used in the preliminary design phase to quickly determine whether or not a grade separation is feasible for a given set of conditions, what gradients may be involved, and what profile adjustments may be necessary on the cross street. The designer should also carefully study sight distance impacts because these often dictate the horizontal distance.

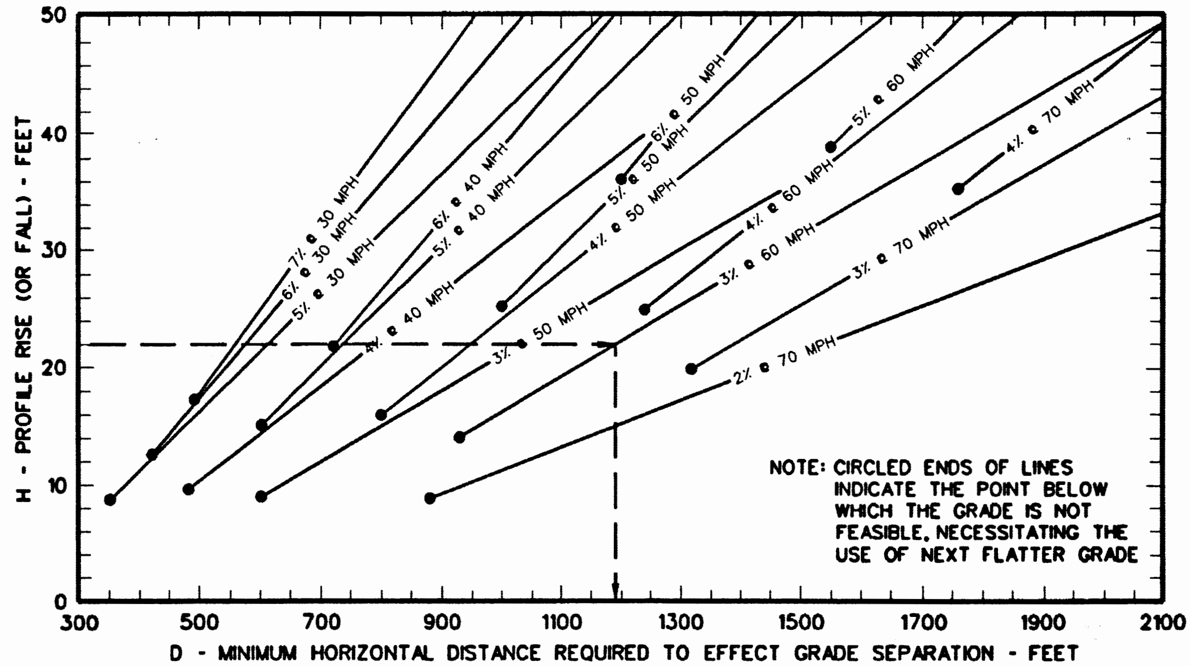
   The required horizontal distances "D" in Figure 10.1E for a grade separation are equal to the length of the initial vertical curve, plus one-half the central vertical curve, plus the length of tangent between the curves. Longer curves are desirable. The lower terminal of the gradient lines on the chart, marked by a small circle, indicates the point where the tangent between the curves is zero and below which a design for the given grade is not feasible; i.e., this results in a profile condition where the minimum central and end curves for the gradient would overlap. The typical difference in elevation for highways for preliminary design computations is 23 ft, including structure depth. For highway bridges over railroads, this distance is usually 28 ft.

* * * * * * * * * * *

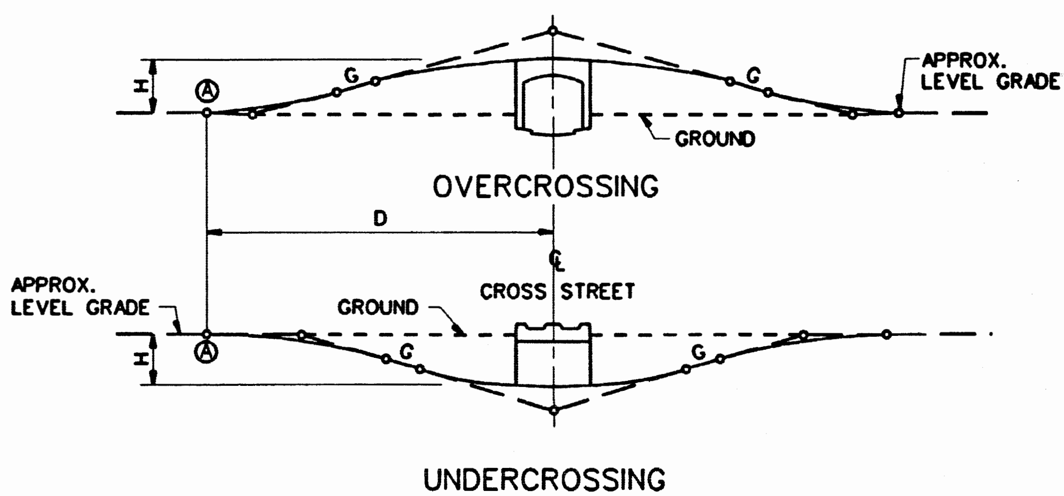
Example 10-1:

Given: Freeway over a cross road
H = 22 ft
Design speed = 60 mph
G = 3%

Problem: Determine necessary horizontal distance for freeway to overpass the cross road.



NOTE: MINIMUM VERTICAL CLEARANCE SHOULD BE CHECKED UNDER THE OUTSIDE EDGE OF THE OVERCROSSING STRUCTURE



Source: (1)

GRADE SEPARATION DETERMINATION

Figure 10.1E

Solution: From Figure 10.1E, read on the vertical axis H = 22′ over to the line labeled "3% @ 60 mph." From this intersection, read down to the horizontal axis and find D = 1180′. Therefore, it will require approximately 1200′ of horizontal distance for the freeway to attain a 22′ elevation above the crossing road without exceeding a grade of 3%. The solution to the example is illustrated on Figure 10.1E.

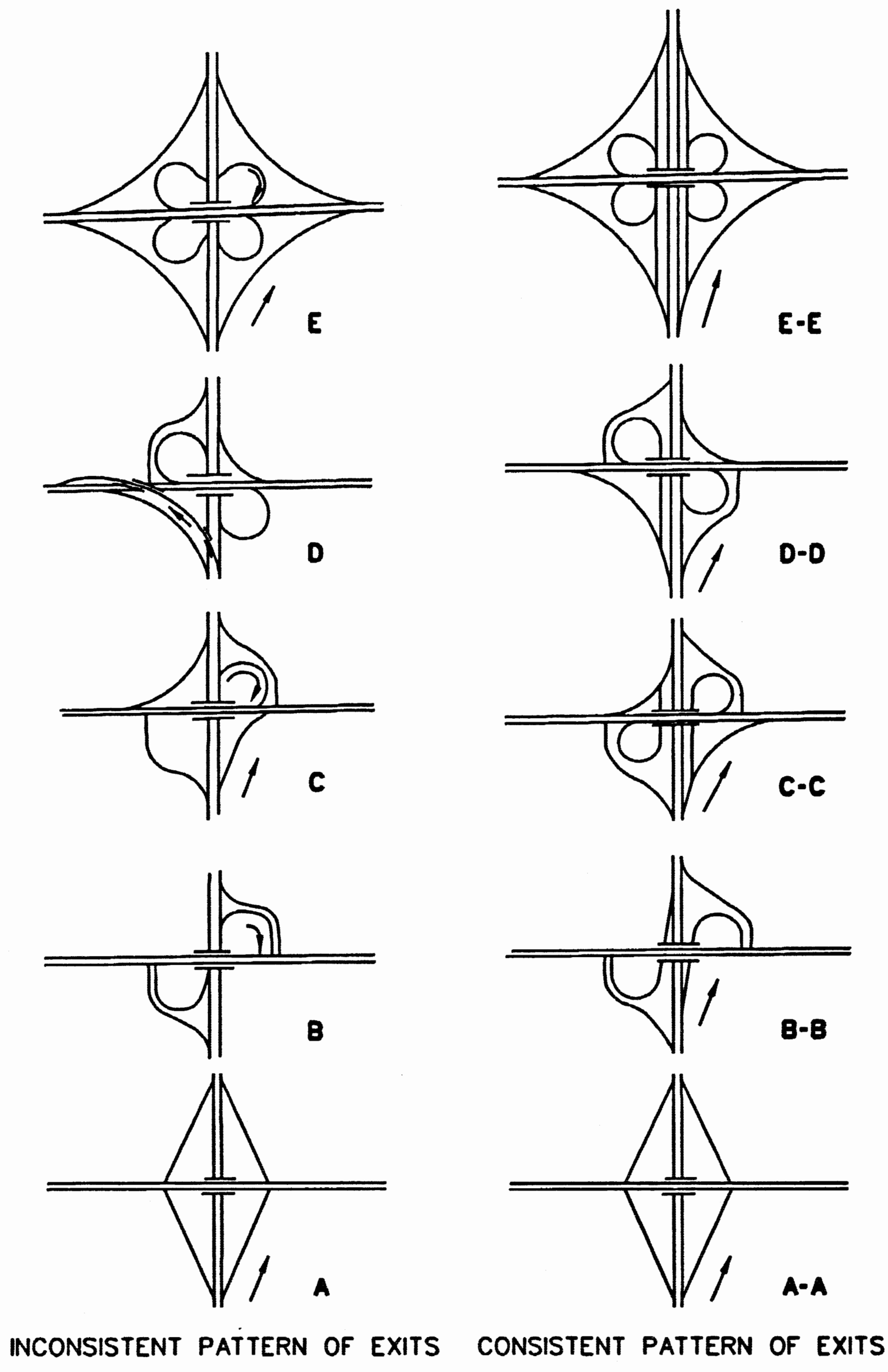
* * * * * * * * * * *

4. Underpass Width. The approach cross section, desirably including clear zones, should be carried through the underpass. Including the clear zone allows for possible expansion in the future with minimal disruption to the overhead structure. In addition, wider underpasses also provide greater sight distance for at-grade ramp terminals near the structure

5. Uniformity. Interchange patterns should be uniform from one interchange to another. All ramps should desirably exit and enter on the right, and all exits should desirably be before the structure. Figure 10.1F illustrates an inconsistent pattern and a consistent pattern.

6. Testing for Ease of Operation. The designer should review the proposed design from the driver's perspective. This involves tracing all possible movements that an unfamiliar motorist would drive through the interchange. The designer should review the plans for areas of possible confusion, proper signing and ease of operation, and to determine if sufficient weaving distances and sight distances are available. The designer should have available the peak-hour volumes, number of traffic lanes, etc., to determine the type of traffic the driver will encounter.

7. Grading. The designer should consider the grading around an interchange early in design. Properly graded interchanges allow the overpass structures to naturally blend into the terrain. In addition, the designer needs to ensure that the slopes are not too steep to support the bridge and roadways and that they can support plantings which prevent erosion and enhance the appearance of the area. Flatter slopes also allow for easier maintenance.



Source: (1)

ARRANGEMENTS OF SUCCESSIVE EXITS

Figure 10.1F

10.2 INTERCHANGE TYPE SELECTION

10.2.1 General Evaluation

Section 10.2.2 presents the interchange types which may be used at a given site. The Geometric Design Branch selects the type of interchange for the site. Typically, the Branch will evaluate several types for potential application. Each type should be evaluated considering:

1. compatibility with the surrounding highway system;
2. route continuity;
3. uniformity of exit and entrance patterns;
4. level of service for each interchange element (e.g., freeway/ramp junction, ramp proper);
5. operational characteristics (single versus double exits, weaving, signing);
6. road user impacts (travel distance and time, safety, convenience and comfort);
7. driver expectancy;
8. geometric design;
9. construction and maintenance costs;
10. potential for stage construction;
11. right-of-way impacts and availability; and
12. environmental impacts.

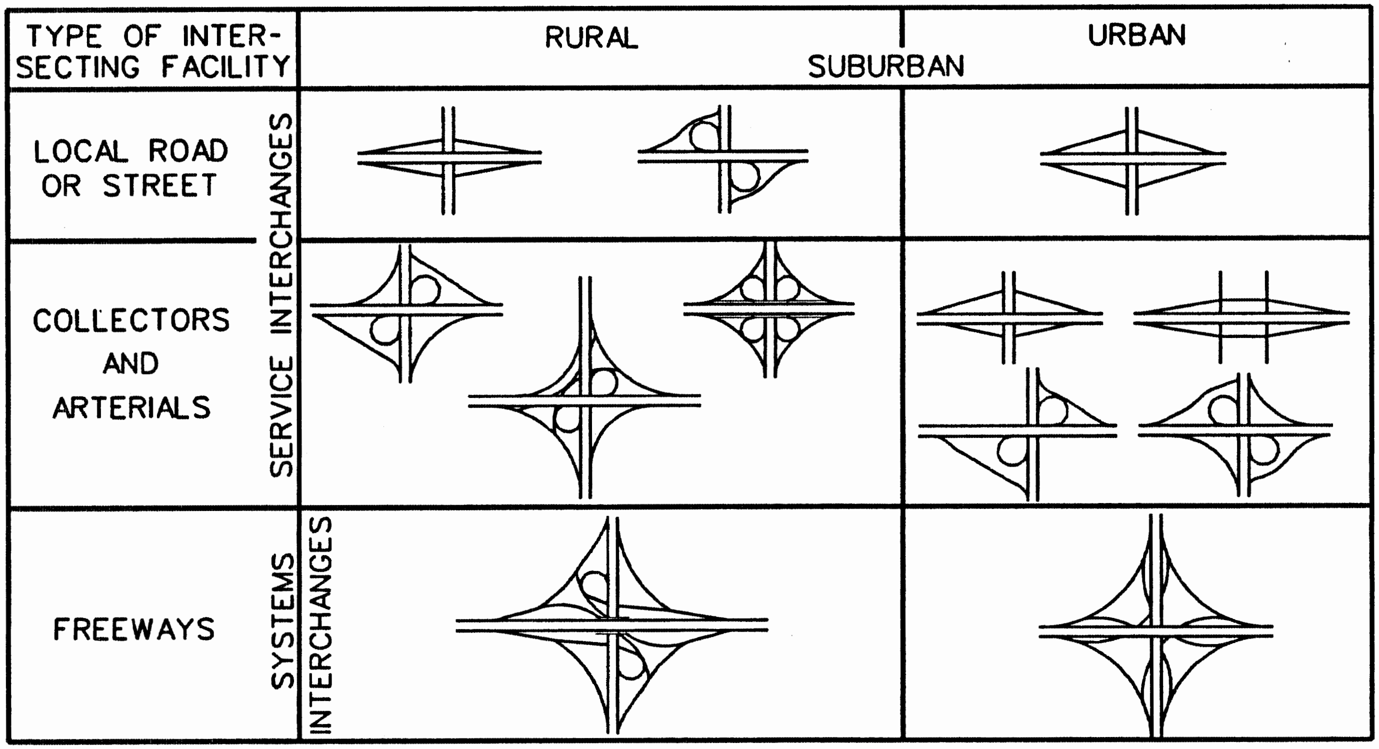
In addition, three other overall factors also influence the selection of an interchange type:

1. Basic Types. A freeway interchange will be one of two basic types. A "systems" interchange will connect freeway to freeway; a "service" interchange will connect freeway to a lesser facility.
2. Urban/Rural. In rural areas where interchanges occur relatively infrequently, the design can be selected strictly on the basis of service demand and analyzed as a separate unit. In urban areas where restricted right-of-way and close spacing of interchanges are common, the type selection and design of the interchange may be severely limited. The operational characteristics of the intersecting road and nearby interchanges will be major influences on the design of an urban interchange.
3. Movements. 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 the maneuver.

Figure 10.2A presents general guidance for interchanges that are adaptable on freeways based on the classification of the intersecting facilities in rural, suburban or urban environments.

10.2.2 Types

This section describes the basic types of interchanges. Each interchange must be custom-designed to fit the individual site considerations. The final design may be a minor or major modification of one of the basic types or may be a combination of two or more basic types.



Source: (1)

FREEWAY INTERCHANGES
(Based on Type of Intersecting Facilities)

Figure 10.2A

10.2.2.1 Three-Leg

Three-leg interchanges, also known as T- or Y-interchanges, are provided at intersections with three legs. Figure 10.2B illustrates examples of 3-leg interchanges with several methods of providing the turning movement s. The trumpet type is shown in (A) where three of the turning movements are accommodated with direct or semi-direct ramps and one movement by a loop ramp. In general, the semi-direct ramp should favor the heavier left-turn movement and the loop the lighter volume. Where both left-turning movements are fairly heavy, the design in (B) may be suitable. A fully directional interchange (C) is appropriate when all turning volumes are heavy, or the intersection is between two access-controlled highways. This would be the most costly type because of the necessary multiple structure.

10.2.2.2 Diamond

The diamond is the simplest and perhaps the 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. Figure 10.2C illustrates schematics of typical diamond interchanges with and without frontage roads. Some of its advantages and disadvantages include:

<u>Advantages</u>

1. All traffic can enter and exit the mainline at relatively high speeds. Adequate sight distance can usually be provided, and the operational maneuvers are normally uncomplicated.
2. Relatively little right-of-way is required.
3. All exits from the mainline are made before reaching the structure. This conforms to driver expectancy and therefore minimizes confusion.
4. Left-turning maneuvers require little extra travel distance.
5. The diamond configuration easily allows modifications to provide greater ramp capacity, if needed in the future.
6. Their common usage has resulted in a high degree of driver familiarity.

<u>Disadvantages</u>

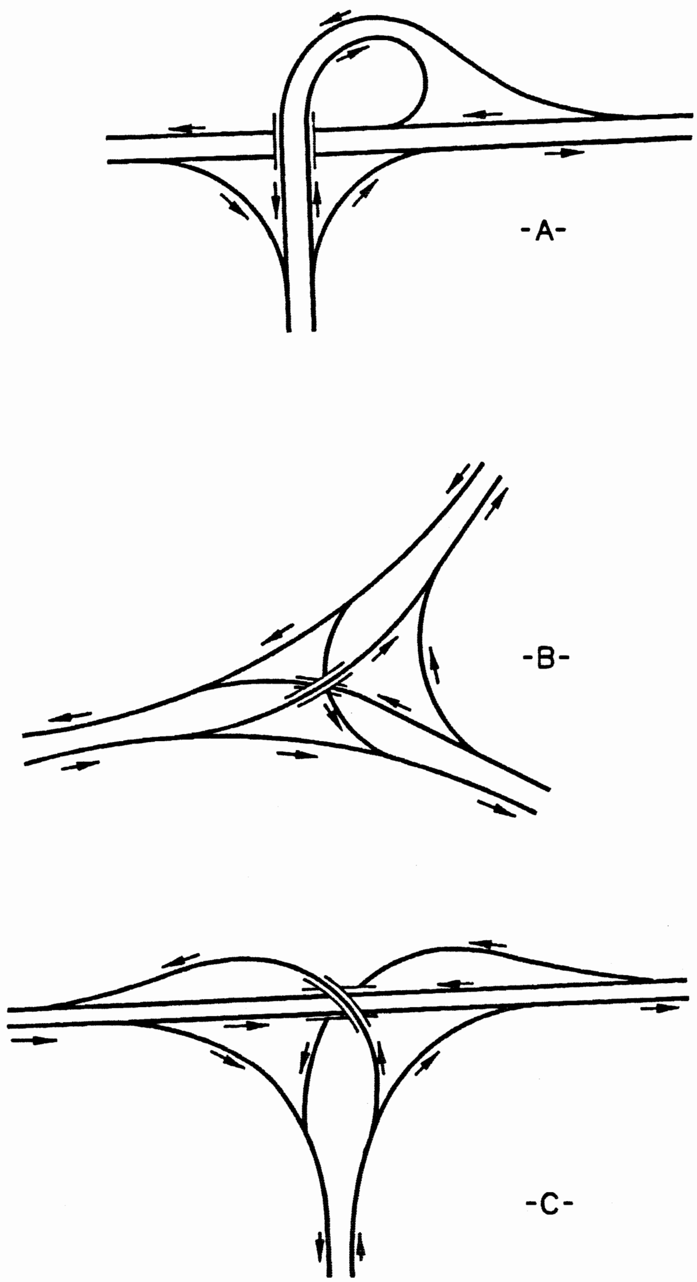
1. There are potential operational problems with the two at-grade intersections at the minor road.
2. There is greater potential for wrong-way entry onto the ramps.
3. Sufficient intersection sight distance needs to be provided at the minor roads.

10.2.2.3 Compressed Diamond

Figure 10.2D illustrates a special type of diamond interchange — a "compressed" diamond. With this interchange, all legs of the interchange meet at a single point. Some of the advantages and disadvantages of this interchange are as follows:

<u>Advantages</u>

1. It can significantly increase the interchange capacity. This arrangement can alleviate the operational problems of having two closely spaced at-grade



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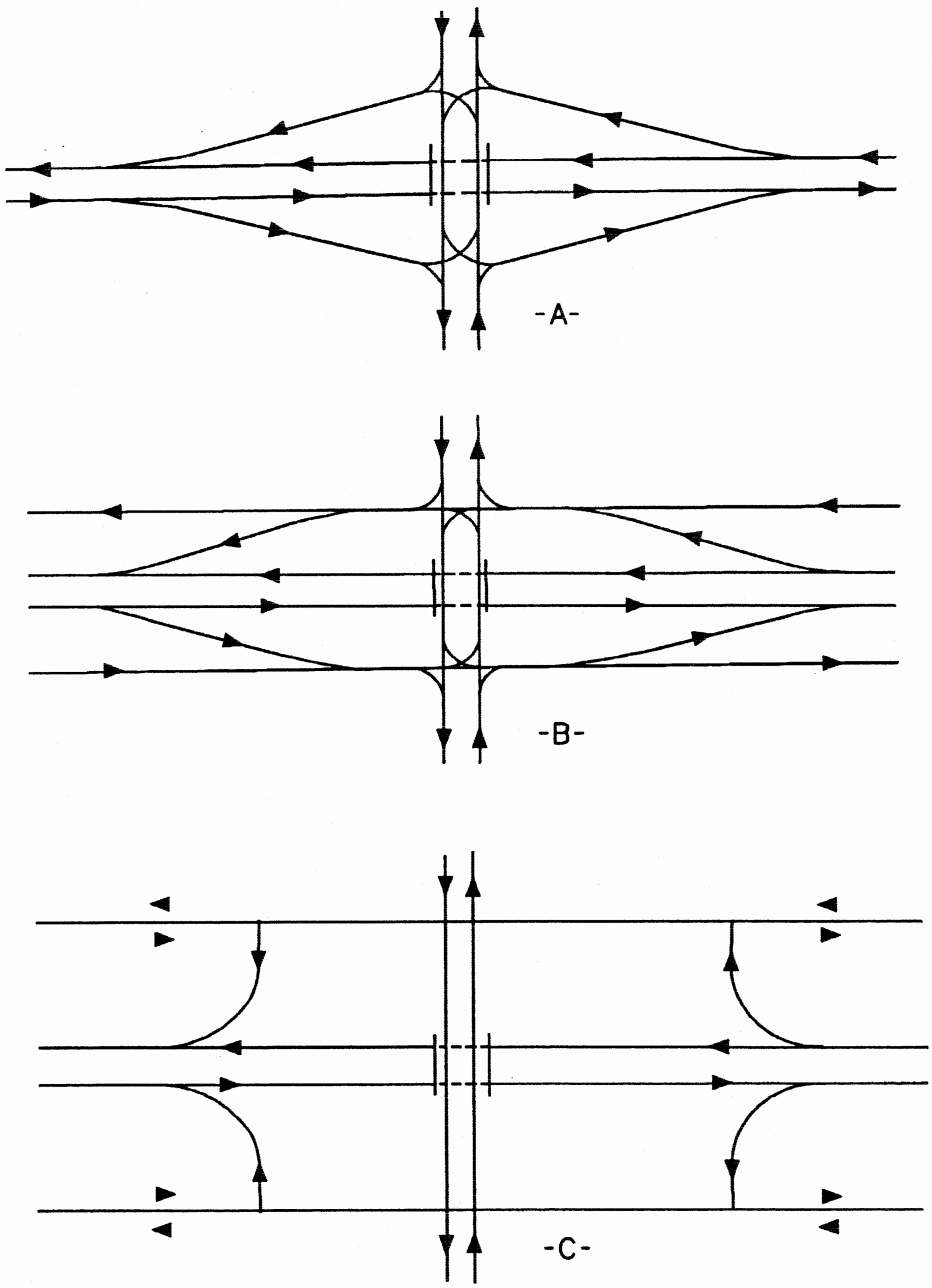
THREE-LEG INTERCHANGES

Figure 10.2B



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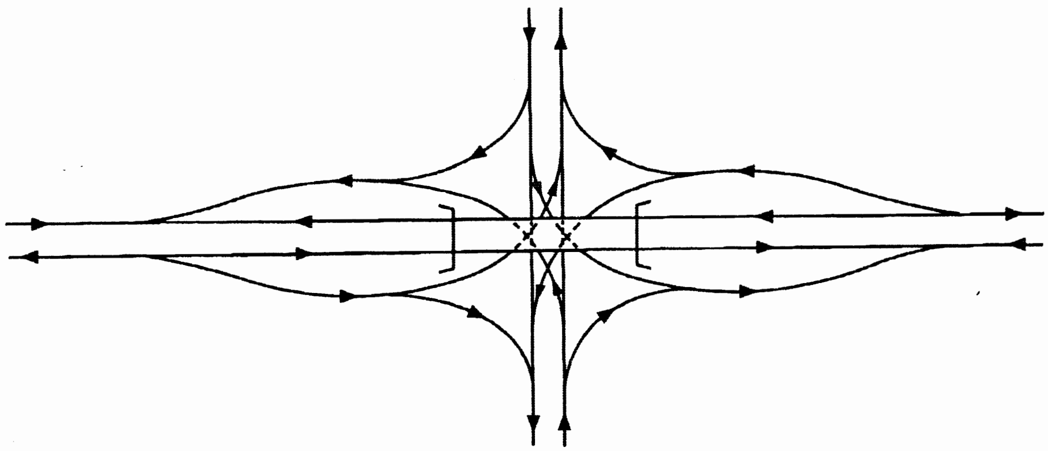
DIAMOND INTERCHANGES

Figure 10.2C



"COMPRESSED" DIAMOND INTERCHANGE

Figure 10.2D

intersections on the minor road. In particular, it overcomes the left-turning lane storage problem for drivers wishing to enter the freeway.

2. It only requires one signal instead of two at the typical diamond.

Disadvantages

1. It has a higher cost than the typical diamond because of the need for a larger structure.

2. Careful consideration needs to be given to the design of channelization to minimize driver confusion and the likelihood of wrong-way maneuvers.

3. There is a significantly wider pavement area for pedestrians to cross.

4. Because of wide pavement areas, it requires longer signal clearance intervals.

5. It is difficult to accommodate frontage roads.

10.2.2.4 Three-Level Diamond

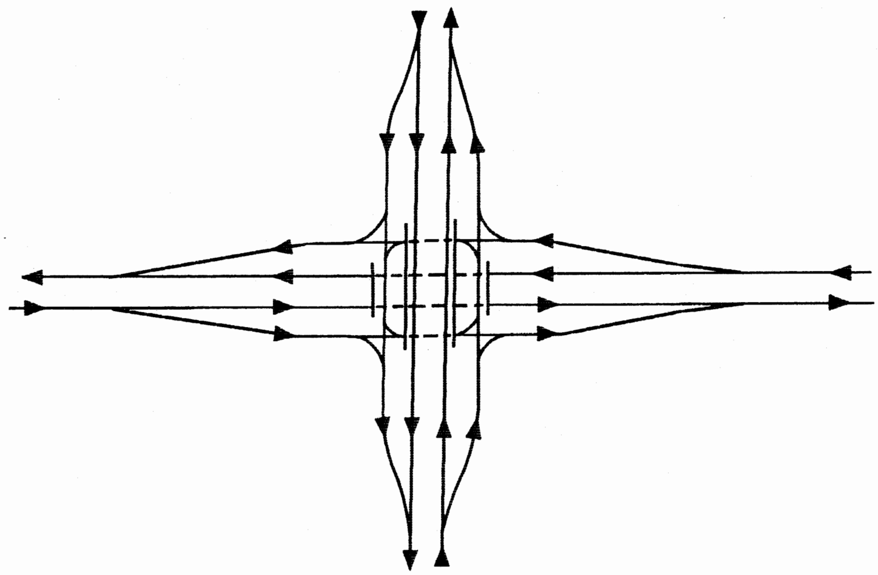
Figure 10.2E illustrates a special type of diamond interchange called a "three-level" diamond. With this interchange, all of the at-grade intersections are on a separate level than the two mainlines. Some advantages and disadvantages of this type of interchange are as follows:

Advantages

1. It can handle high traffic volumes.

2. It uses less right-of-way than loop ramps.

3. At-grade intersections are removed from both mainlines, thereby significantly increasing the capacity of the intersection.



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THREE-LEVEL DIAMOND INTERCHANGE

Figure 10.2E

4. One-way frontage roads can be easily incorporated into the interchange configuration.

Disadvantages

1. To make a left turn, a driver needs to pass through three at-grade intersections and/or traffic signals.

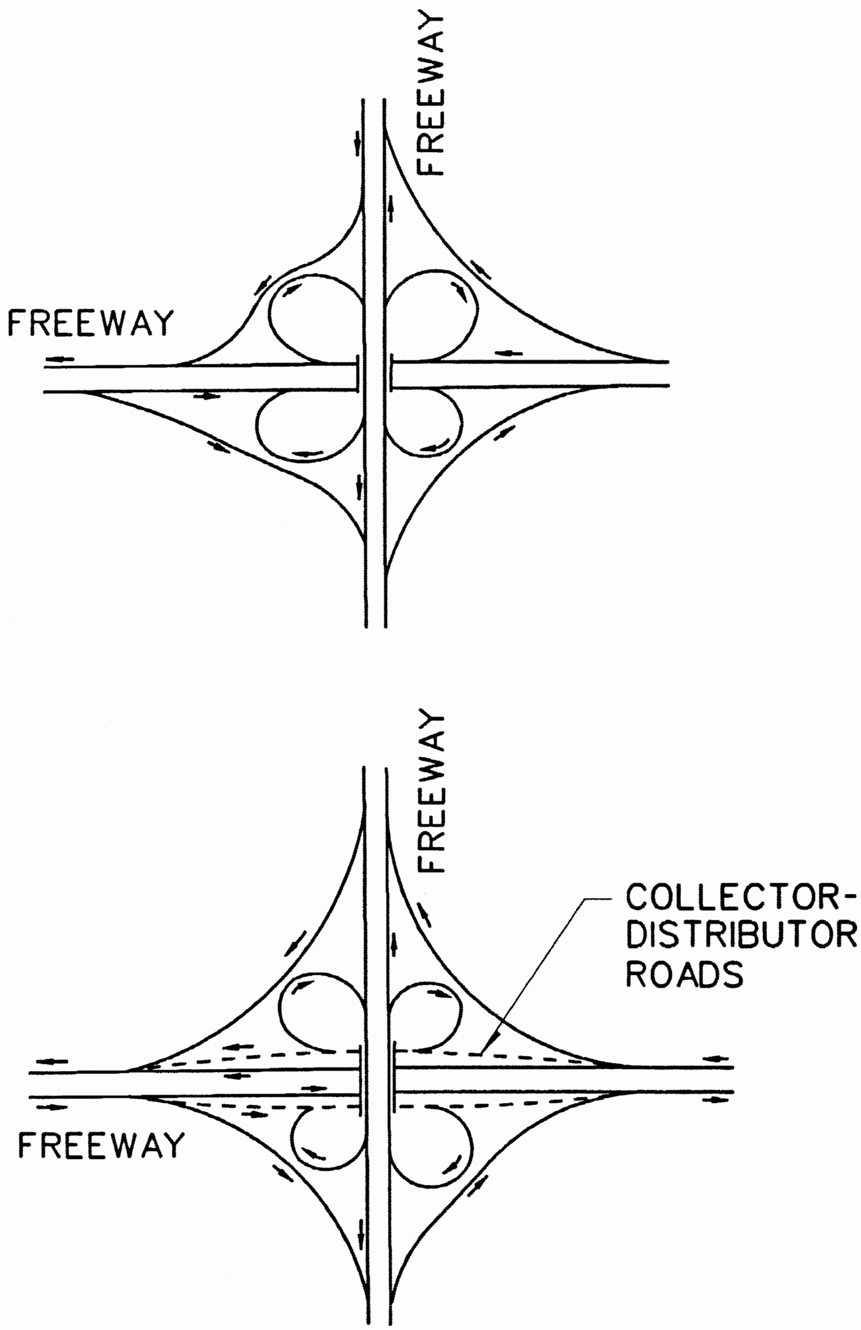
2. The additional structures result in higher construction costs.

10.2.2.5 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.

Where two access-controlled highways intersect, a full cloverleaf is the minimum type design interchange that will suffice. However, cloverleafs introduce several undesirable operational features such as the double exits and entrances from the mainline, the weaving between entering and exiting vehicles with the mainline traffic and, when compared to directional interchanges, the additional travel time and distance for left-turning vehicles. Therefore, at freeway-to-freeway interchanges a collector-distributor (C-D) road should be considered with a full cloverleaf, or a fully directional interchange should be provided. Figure 10.2F provides typical examples of full cloverleafs with and without C-D roads. See Section 10.5 for more discussion on C-D roads.

Because of the undesirable operational features and the amount of right-of-way necessary for full cloverleafs, ODOT rarely builds new full cloverleaf interchanges; however, reconstruction of existing full cloverleaf interchanges may be necessary.



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FULL CLOVERLEAFS

Figure 10.2F

Operational experience with full-cloverleaf interchanges has yielded several conclusions on their design. Subject to a detailed analysis on a site-by-site basis, the following generally pertain to the design of cloverleafs:

1. Loop Radii. Considering all factors, loops can be practically designed for approximate radii of 100 to 150 ft for minor movements on highways with design speeds of 50 mph or less and for 150 to 250 ft for more important movements on highways with higher design speeds.

2. Design Speed Impacts. For every 5-mph increase in design (e.g., 20 to 25), the following results (approximately):

   a. a 50% increase in travel distance,

   b. a 130% increase in required right-of-way, and

   c. a 20% - 30% increase in travel time.

3. Loop Capacity. A loop rarely operates with other than a single line of traffic and, therefore, its design capacity is between 800-1200 vph. The higher figure is only achievable where the design speed is 30 mph or higher and few trucks use the loop.

4. Weaving Volumes. An auxiliary lane may be provided between successive entrance/exit loops within the interior of a cloverleaf interchange. This produces a weaving section between the mainline and entering/exiting traffic. When the total volume on the two successive ramps reaches approximately 1000 vph, interference increases rapidly with a resulting reduction of the through traffic speed. At these weaving volume levels, a collector-distributor road should be considered.

Some of the advantages and disadvantages of full cloverleafs include:

Advantages

1. Full cloverleafs eliminate all at-grade intersections and, therefore, eliminate left turns.

2. Full cloverleafs are intended to eliminate all vehicular stops through the use of merges.

3. Where right-of-way is reasonably inexpensive, full cloverleafs may be a practical option.

Disadvantages

1. Full cloverleafs require more right-of-way and are more expensive than diamonds.

2. Full cloverleafs result in weaving sections.

3. The loops in cloverleafs result in a greater travel distance for left-turning vehicles than do diamonds, and the loops operate at lower speeds.

4. All exits and entrances at diamonds are conducted before reaching the structure, which conforms to driver expectancy. This is not true of full cloverleafs, and the double exits introduce signing problems.

5. Ramps at diamond interchanges can be widened to increase capacity. Loop ramps, regardless of width, almost always operate in a single line therefore limiting capacity.

6. Pedestrian movements along cross streets are difficult to safely accommodate at cloverleaf interchanges.

10.2.2.6 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 on the major road by a loop without the immediate presence of an exiting loop from the major road. Figure 10.2G illustrates eight examples of partial cloverleafs. In "A" and "C" both left-turn movements onto the major road are provided by loops, a distinct preference. In "B" and "D" left-turn movements are necessary across the major road, which can cause problems. Therefore, the partial cloverleaf arrangements in "A" and "C" should be used wherever practical. The other examples illustrate two loops in opposite quadrants and loops in three quadrants.

Several of the advantages and disadvantages listed for full cloverleafs also apply to partial cloverleafs (e.g., geometric restriction of loops). Some specific advantages of partial cloverleafs include:

1. Partial cloverleafs are often appropriate where one or more quadrants present serious right-of-way and/or terrain problems.

2. Partial cloverleafs may reduce the number of left-turn movements when compared to a diamond interchange.

3. Depending upon site conditions, partial cloverleafs may offer the opportunity to increase weaving distances.

10.2.2.7 Directional and Semi-Directional

The following definitions apply to directional and semi-directional interchanges:

1. Semi-Direct Connection -- A ramp that is indirect in alignment, yet more direct than loops (see Figure 10.2H).

2. Direct Connection -- A ramp that does not deviate greatly from the intended direction of travel (see Figure 10.2I).

3. Semi-Directional Interchange -- An interchange where one or more left-turn movements are provided by semi-direct connections, even if the minor left-turn movements are accommodated by loops (Figure 10.2H).

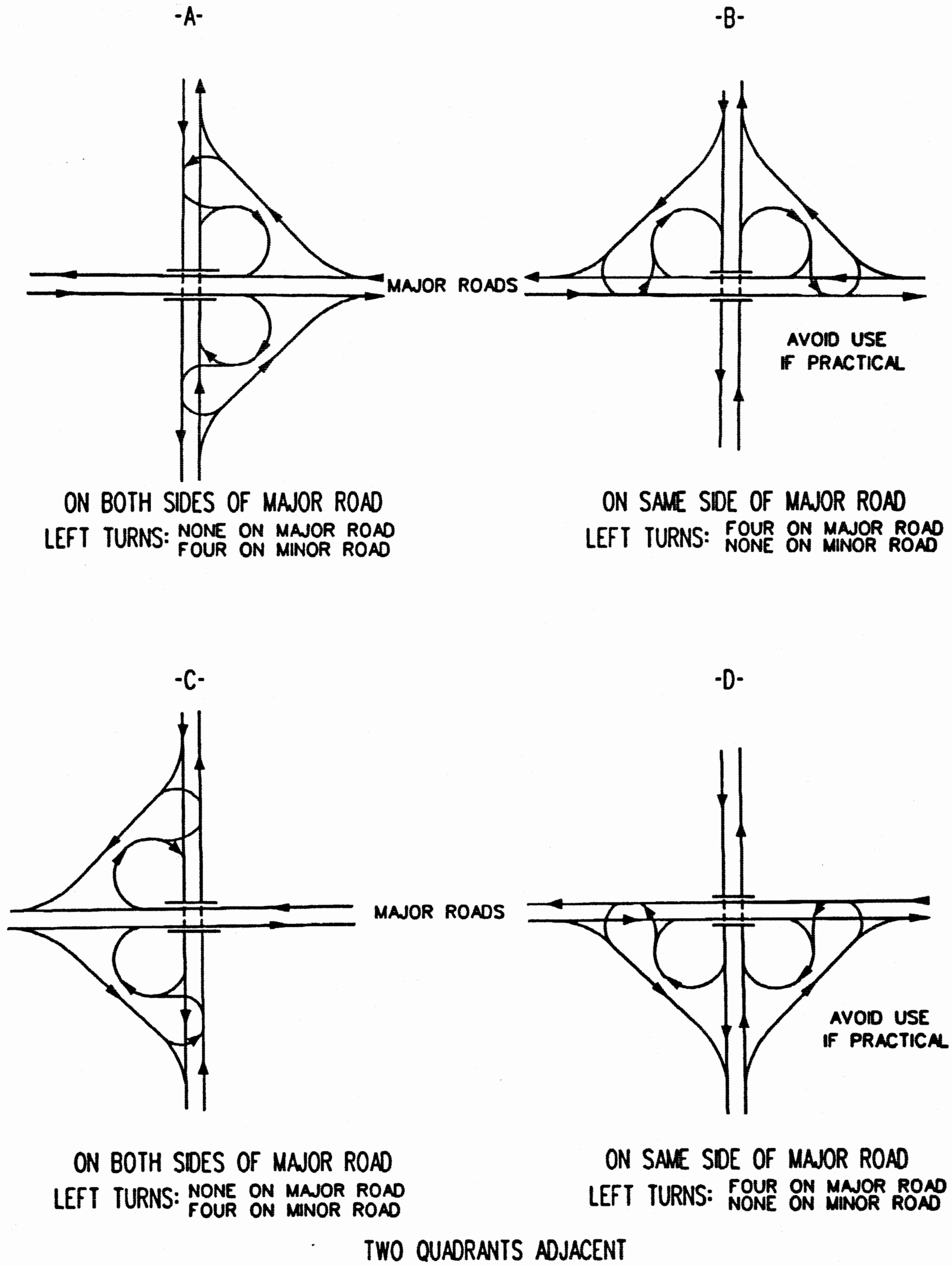
4. Directional Interchanges -- An interchange where one or more left-turn movements are provided by direct connection, even if the minor left-turn movements are accommodated by loops (Figure 10.2I).

5. Fully Directional Interchange -- An interchange where all left-turn movements are provided by direct connections (see "D" in Figure 10.2I).

Direct or semi-direct connections are used for heavy left-turn movements to reduce travel distance, increase speed and capacity and eliminate weaving. These type connections allow an interchange to operate at a better level of service than is possible with cloverleaf interchanges.

Directional or semi-directional interchanges are most often warranted in urban areas at freeway-to-freeway or freeway-to-arterial intersections. They require less right-of-way than cloverleaf interchanges. A fully directional interchange provides the highest possible capacity and level of service, but it is extremely costly to build because of the multiple-level structure required.

Figures 10.2H and 10.2I illustrate several semi-directional and directional interchanges.



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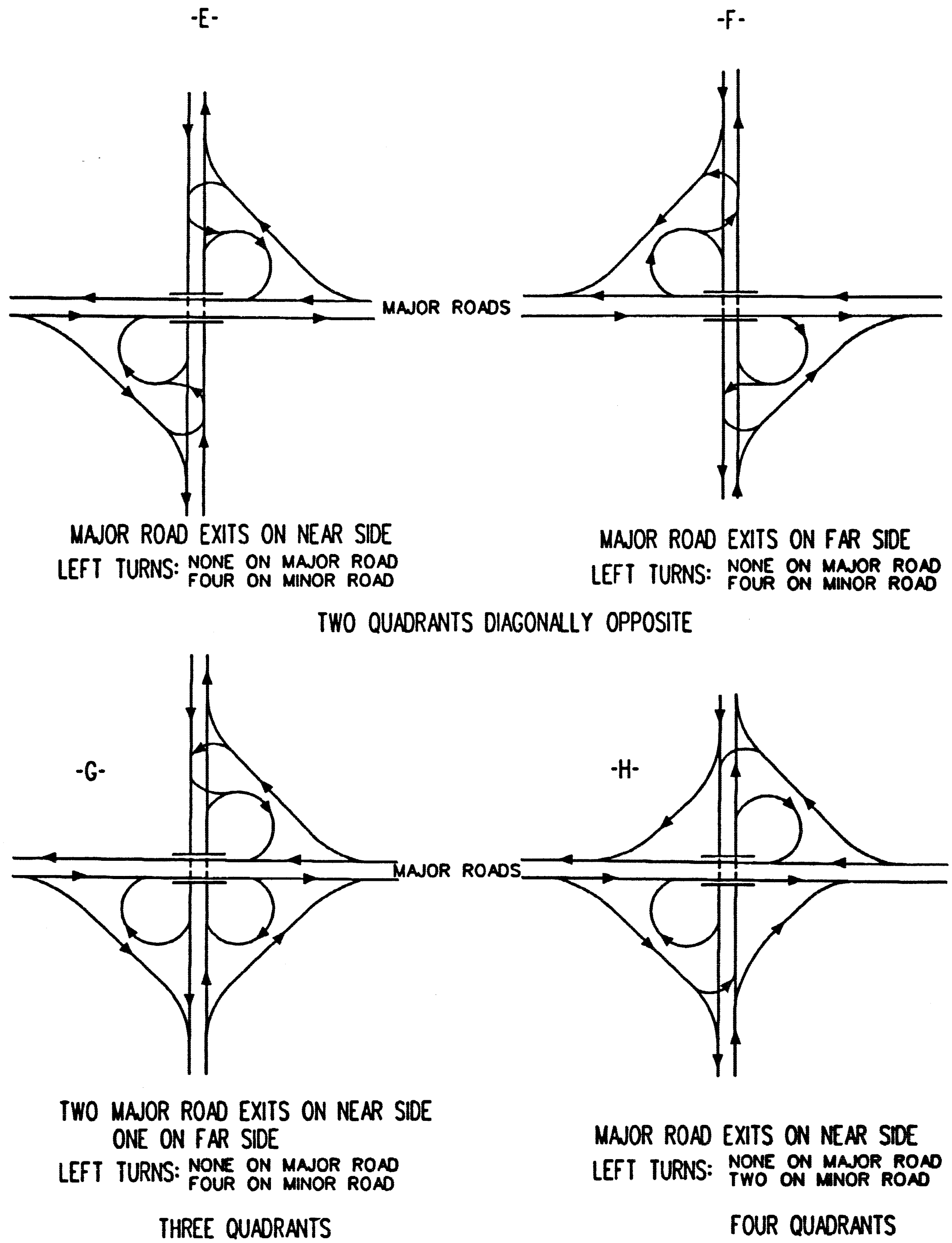
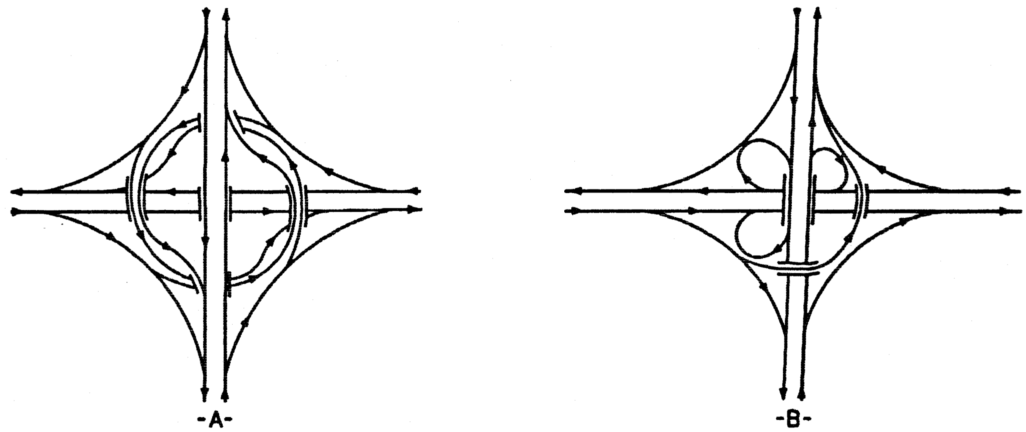
PARTIAL CLOVERLEAF ARRANGEMENTS

Figure 10.2G



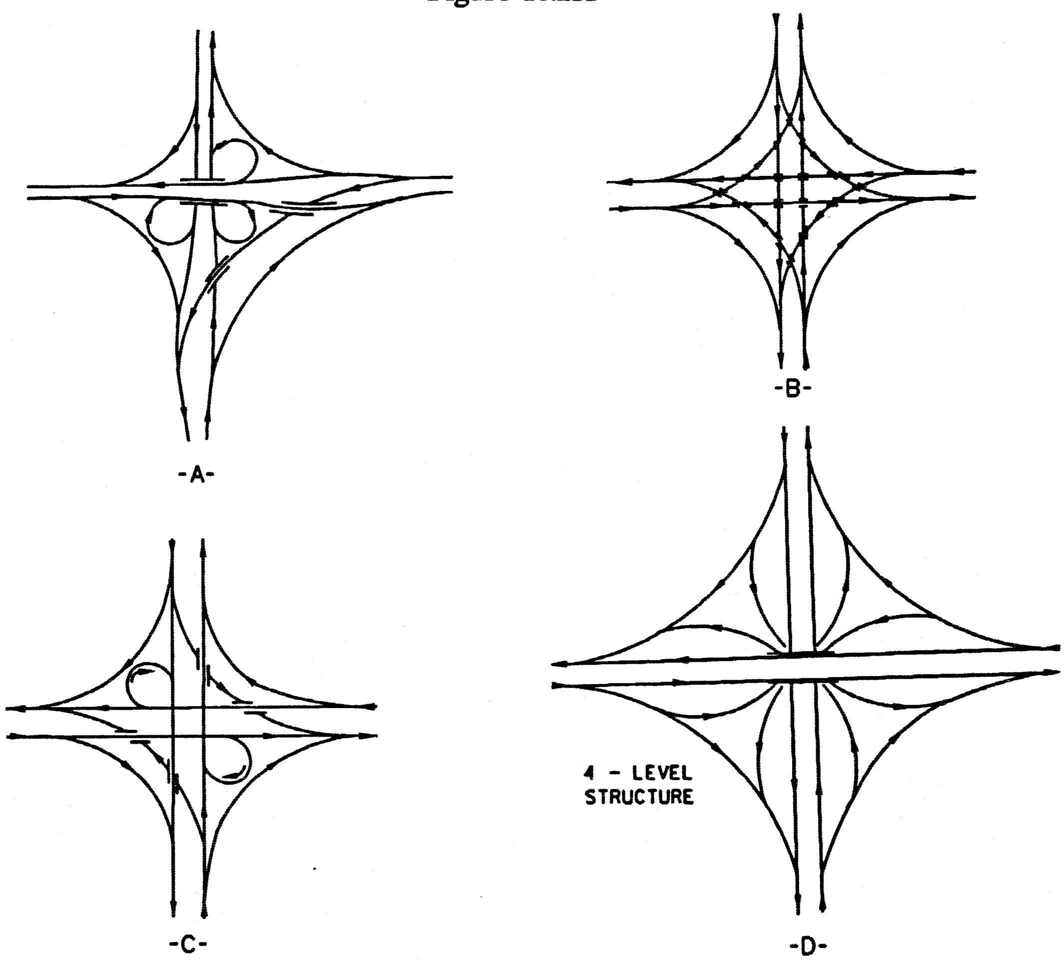
PARTIAL CLOVERLEAF ARRANGEMENTS
(Continued)

Figure 10.2G



SEMI-DIRECTIONAL INTERCHANGES

Figure 10.2H



DIRECTIONAL INTERCHANGES

Figure 10.2I

In comparing "D" in Figure 10.2I with the other directional interchanges, "D" is the only design with no left-hand exits or entrances from the mainline, which is very desirable.

10.3 FREEWAY/RAMP JUNCTIONS

10.3.1 Exit Ramps

10.3.1.1 Types of Exit Ramps

ODOT uses two basic types of exit ramps -- the taper design and the parallel design. ODOT policy is to provide taper exit ramps whenever practical. The taper exit is the design generally preferred by drivers because it provides a clear path of departure. However, there are several situations where the parallel design may be more appropriate:

1. where a taper design cannot provide the needed deceleration distance prior to a sharp curve on the ramp;

2. where a ramp exit is just beyond a structure and there is insufficient sight distance available to the ramp gore. The signing and sight distance restrictions often require the ramp terminus to begin prior to the structure; a taper design could require significant widening of the structure;

3. where the capacity of the at-grade ramp terminal is insufficient and ramp traffic may back up onto the freeway;

4. where the exit ramp departs from a horizontal curve on the mainline. The parallel design is less confusing to through traffic and will normally result in smoother operations. The parallel design may also be required to meet ODOT's cross slope rollover criteria; and

5. where there is a need for a continuous auxiliary lane (see Section 10.3.3).

The following figures illustrate ODOT's design details for exit ramps:

1. Figure 10.3A illustrates the ODOT typical taper exit ramp for uncurbed facilities. The design speed of the mainline is 60 mph, and the design speed of the exit curve at the freeway/ramp junction is 45 mph or higher. NOTE: 50 mph should be the minimum ramp speed for this design; 45 mph may be used in restricted locations.

2. Figure 10.3B illustrates the ODOT typical taper exit ramp for uncurbed facilities. The design speed of the mainline is 70 mph, and the design speed of the exit curve at the freeway/ramp junction is 50 mph or higher.

3. Figure 10.3C illustrates the ODOT typical parallel-lane exit ramp for uncurbed facilities.

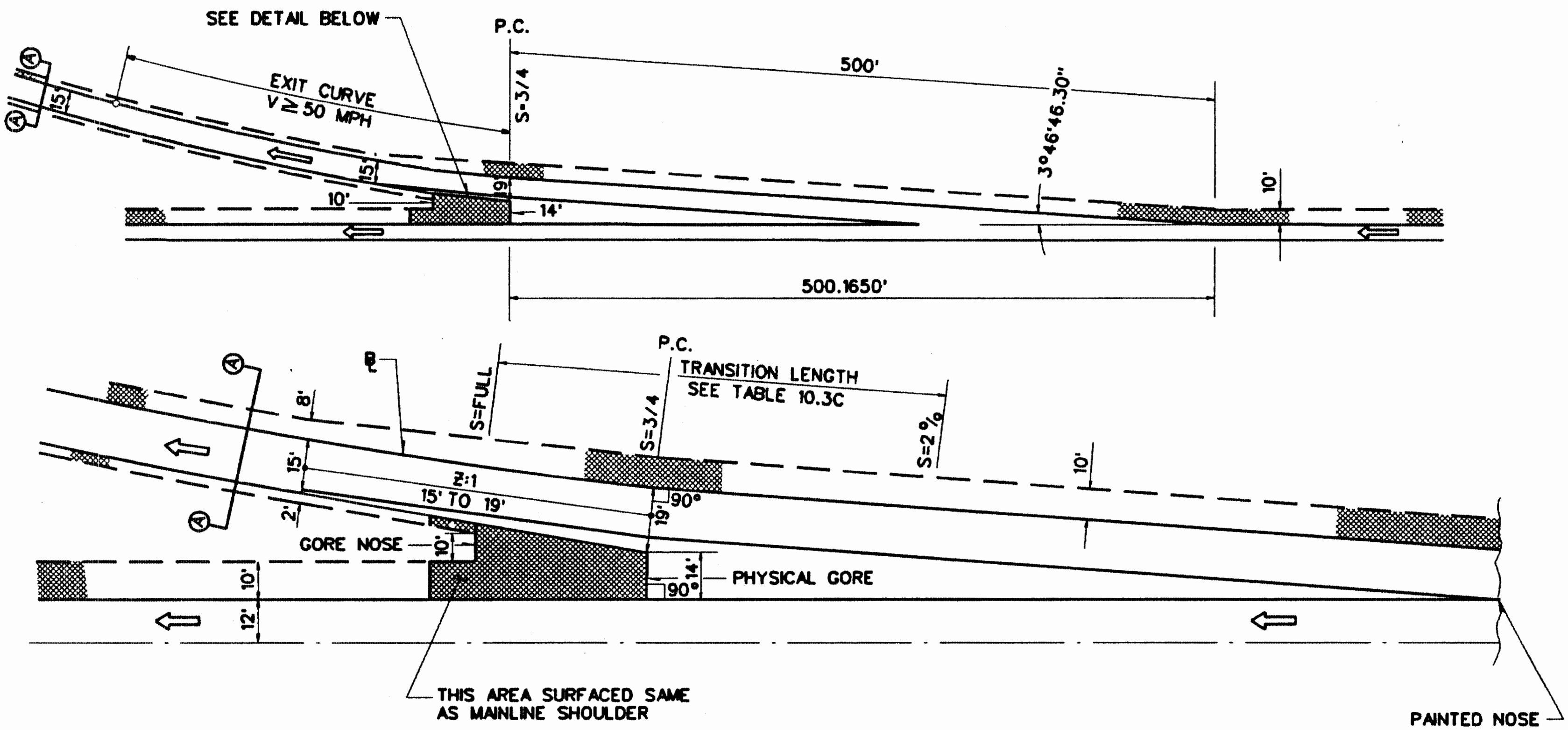
4. Figure 10.3D illustrates the typical exit gore design for a curbed ramp. See Section 10.4 for criteria on when to use the curbed ramp design.

10.3.1.2 Taper Rates

The taper rate is the angle upon which the ramp departs from the mainline. The taper length is typically 500 ft to 550 ft, depending upon design speeds (see Figures 10.3A and 10.3B). For a parallel-lane exit design, the taper rate applies to the beginning taper of the parallel lane. This distance is typically 200 ft - 300 ft (see Figure 10.3C).

10.3.1.3 Deceleration

Sufficient deceleration distance is needed to safely and comfortably allow an exiting vehicle to leave the freeway mainline. All deceleration should desirably occur within the full width of the exit lane. The length of deceleration will depend upon the design



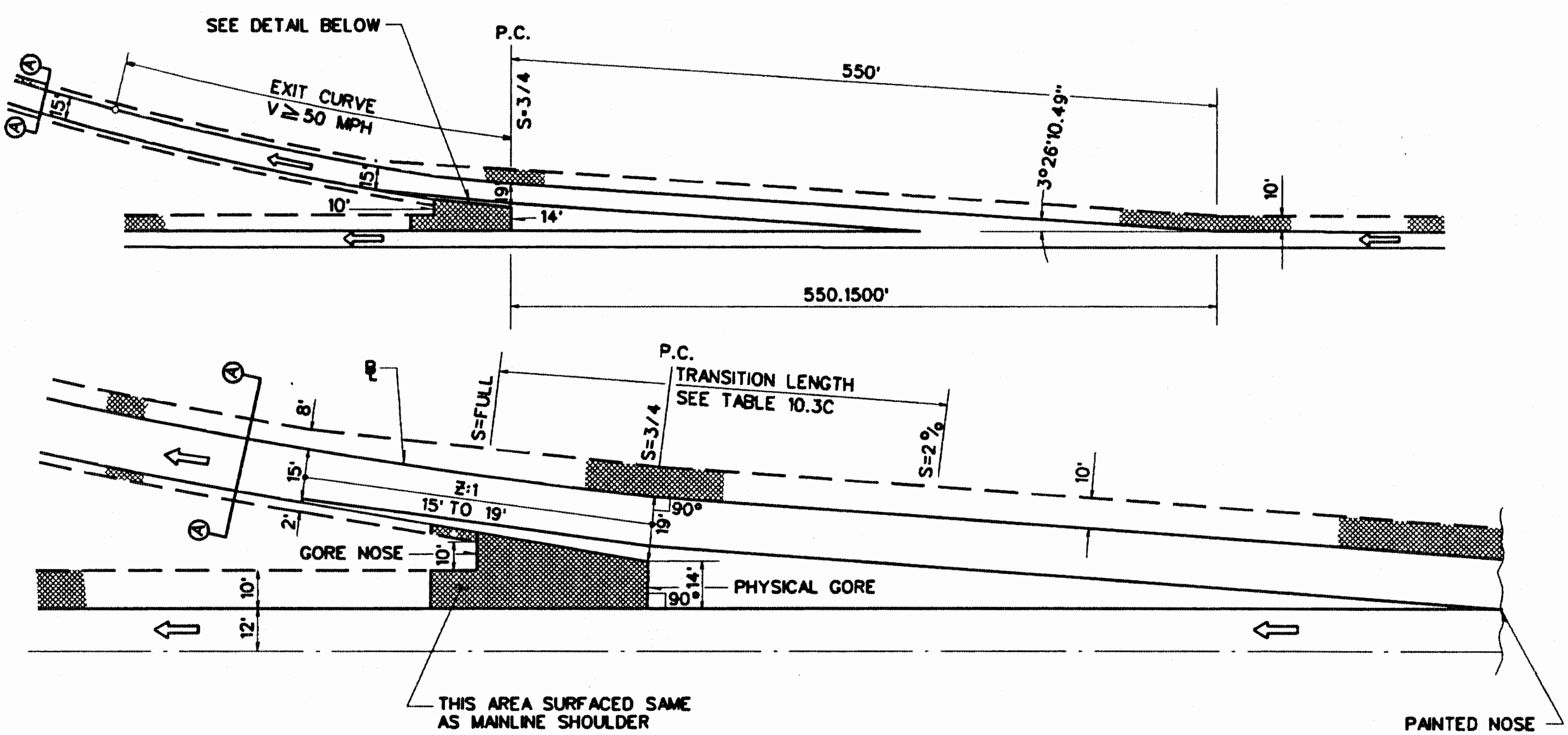
Design Speed of Ramp Curve (mph)	Z — Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where V=60 mph for mainline and V≥50 mph for exit curve. In restricted locations, the exit curve may be designed for V=45 mph.
2. See Table 10.3C for superelevation information.
3. See Figure 10.3D for typical curbed exit gore.
4. See Figure 10.4A for typical ramp section.
5. See Figures 10.3J and 10.3K for exits on curvilinear alignment.
6. A 300′ minimum length of exit curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL TAPER EXIT RAMP
(Uncurbed, V=60 on Mainline)

Figure 10.3A



Design Speed of Ramp Curve (mph)	Z — Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where V = 70 mph for mainline and V≥50 mph for exit curve.
2. See Table 10.3D for superelevation information.
3. See Figure 10.3D for typical curbed exit gore.
4. See Figure 10.4A for typical ramp section.
5. See Figures 10.3J and 10.3K for exits on curvilinear alignment.
6. A 300′ minimum length of exit curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

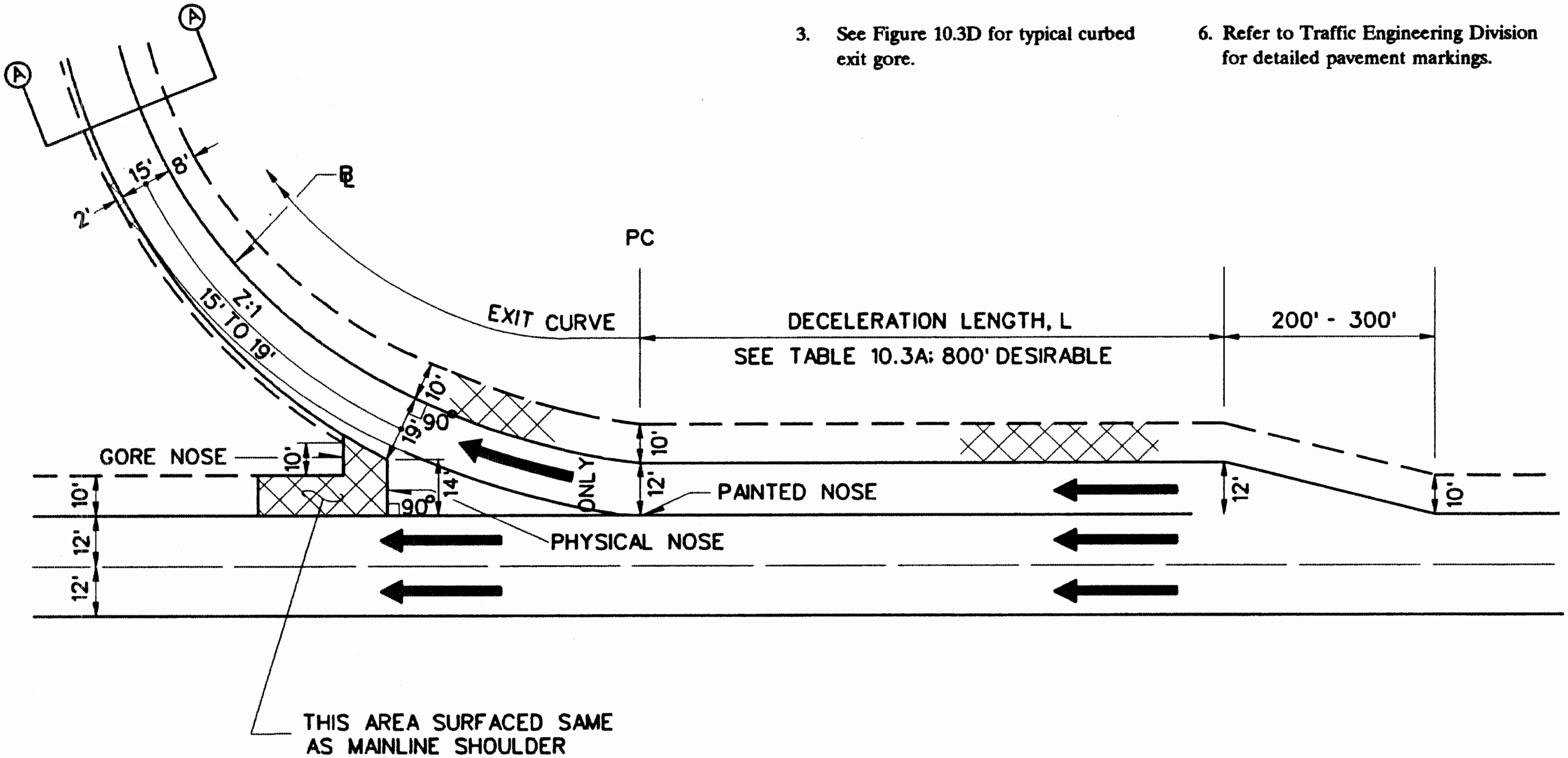
ODOT TYPICAL TAPER EXIT RAMP
(Uncurbed, V = 70 on Mainline)

Figure 10.3B

Design Speed of Ramp Curve (mph)	Z – Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

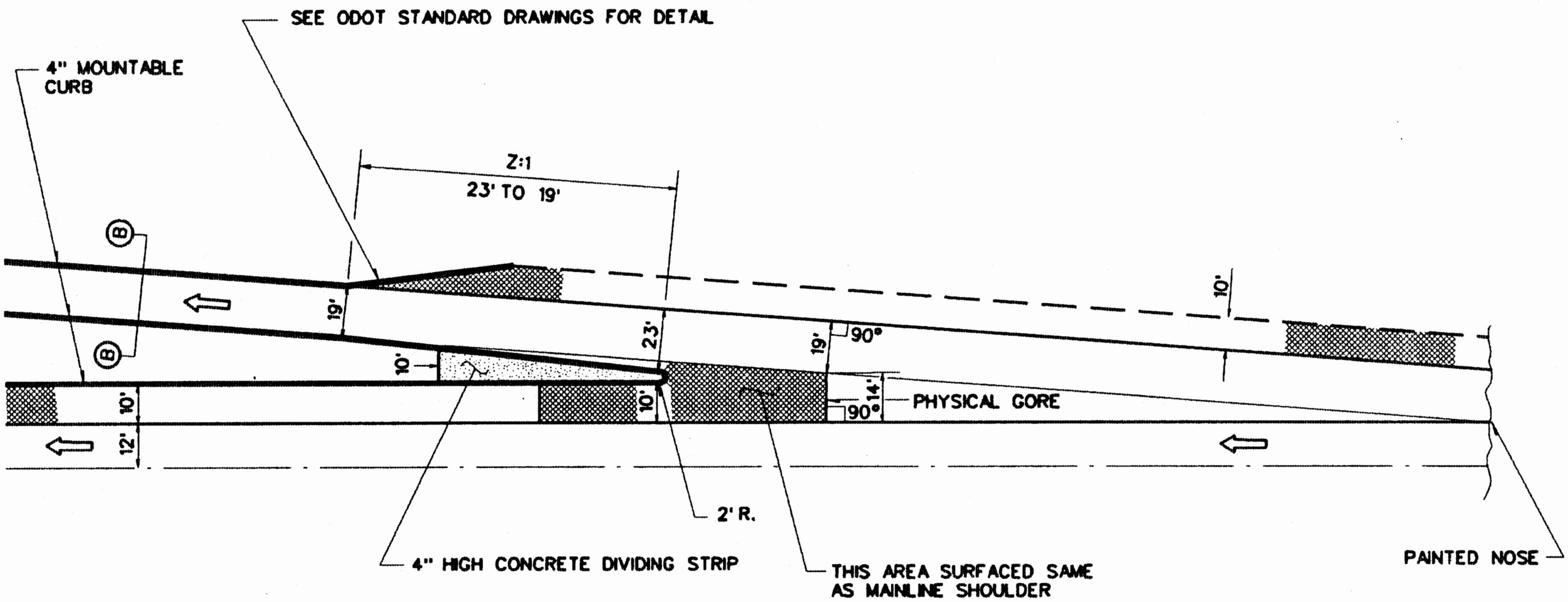
Notes:

1. Figure applies where taper design is inappropriate.
2. See Chapter Six for superelevation information.
3. See Figure 10.3D for typical curbed exit gore.
4. See Figure 10.4A for typical ramp section.
5. A 300′ minimum length of exit curve is desirable.
6. Refer to Traffic Engineering Division for detailed pavement markings.



ODOT TYPICAL PARALLEL-LANE EXIT RAMP
(Uncurbed)

Figure 10.3C



Design Speed of Ramp Curve (mph)	Z – Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where curb is warranted on ramp. See Section 10.4.
2. See Figure 10.4A for typical ramp section.
3. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL EXIT GORE (Curbed)

Figure 10.3D

speed of the mainline and the design speed of the first governing geometric control on the exit ramp. This will most often be a horizontal curve but could be, for example, stopping sight distance on a crest vertical curve. Table 10.3A provides the deceleration distance for various combinations of highway design speed and exit design speed. Greater distances should be provided if practical. Lengths of at least 800 ft are desirable for the parallel-lane design. If the deceleration will occur on a grade of 3% or more, the length of the lane should be adjusted according to the criteria in Table 10.3B.

The specific use of the deceleration criteria to horizontal curves warrants some elaboration. The following will apply:

1. The design speed of the first horizontal curve on the exit ramp will be determined by open-highway conditions (e.g., side-friction factors (f) and the distribution method between f and superelevation) where the radius > 500 ft (see Chapter Six). Where R ≤ 500′, turning roadway conditions may be assumed (see Chapter IX of Reference (1)).

2. Based on the design speeds of the highway and the first curve on the exit ramp, Table 10.3A will yield the required length of deceleration. This will apply from the point where the exit ramp becomes 12 ft to the PC of the horizontal curve.

3. Where ramp lengths are short, the deceleration length may be determined based on the distance required to decelerate from the mainline design speed to zero (a stop condition at an at-grade intersection).

10.3.1.4 Sight Distance

Decision sight distance should be provided for drivers approaching a freeway exit. This sight distance is particularly important for exit loops immediately beyond a structure. Vertical curvature or bridge piers can obstruct the exit points if not carefully designed. When measuring for adequate sight distance, the desirable height of object will be 0.0 ft (the roadway surface); it is acceptable to use 6 inches.

10.3.1.5 Superelevation

The superelevation for horizontal curves in the vicinity of the freeway/ramp junction must be developed to properly transition the driver from the mainline to the curvature at the exit. The principles of superelevation for open highways, as discussed in Chapter Six, should be applied to the freeway/ramp junction. Figures 10.3A and 10.3B illustrate superelevation development at the junction.

Tables 10.3C and 10.3D provide the detailed superelevation criteria for various combinations of mainline and ramp design speeds. These tables are used in conjunction with Figures 10.3A and 10.3B.

The following will apply to superelevation development at exit ramps:

1. $\underline{e_{max}}$. On the exit ramp portion of the freeway/ramp junction, the typical e_{max} is 0.06. An e_{max} = 0.08 may be used for loop ramps and/or where restrictive site conditions dictate the need for sharper curvature.

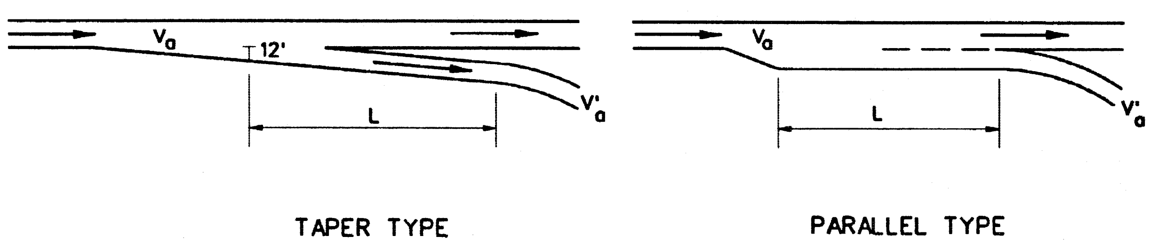
2. Superelevation Rate. As discussed in Chapter Six, Method 5 is used for open highways to distribute superelevation and side friction. Therefore, Table 10.3C or 10.3D (e_{max} = 0.06) or Table 6.2B (e_{max} =

Table 10.3A

LENGTHS FOR DECELERATION*
(Passenger Cars)

Highway Design Speed (mph) (V)	Average Running Speed (mph) (V_a)	L = Deceleration Length (ft)								
		For Design Speed Of First Governing Geometric Control (mph) (V′)								
		Stop	15	20	25	30	35	40	45	50
		For Average Running Speed (mph) (V'_a)								
		0	14	18	22	26	30	36	40	44
30	28	235	185	160	140	-	-	-	-	-
40	36	315	295	265	235	185	155	-	-	-
50	44	435	405	385	355	315	285	225	175	-
60	52	530	500	490	460	430	410	340	300	240
65	55	570	540	530	490	480	430	380	330	280
70	58	615	590	570	550	510	490	430	390	340

* These values are for grades less than 3%. See Table 10.3B for steeper upgrades or downgrades.



Note: The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the highway mainline to the first governing geometric control on the exit ramp. The basic assumptions within the AASHTO deceleration model are:

1. The vehicle is initially traveling at the average running speed of the highway mainline.

2. The vehicle decelerates in gear for 3 seconds of travel time.

3. The driver brakes the vehicle at a "comfortable" rate until it reaches the average running speed of the first governing geometric control.

The AASHTO deceleration model is discussed in detail on pp. 348-351 of Reference (4).

Table 10.3B

SUGGESTED 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

* Upgrade adjustment is typically only used in restricted locations.

Notes: 1. Table applies to all highway 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:

Highway Design Speed	-	70 mph
First Exit Curve Design Speed	-	40 mph
Average Grade	-	5% downgrade

Problem: Determine length of deceleration.

Solution: Table 10.3A yields a minimum deceleration length of 430 ft on the level. According to Table 10.3B, this should be increased by 1.35.

Therefore: L = 430 x 1.35
L = 581 ft

The needed 581-ft distance means that a parallel-lane exit would be used. Desirably, an 800-ft full-width parallel lane would be used.

Table 10.3C

SUPERELEVATION DEVELOPMENT
(Freeway/Ramp Exits, V=60 mph (Mainline))

D Exit Curve	V=45 mph Exit Curve		V=50 mph Exit Curve		V=55 mph Exit Curve		V=60 mph Exit Curve	
	S	Trans. Length (ft)	S	Trans. Length (ft)	S	Trans. Length (ft)	S	Trans. Length (ft)
3°00′	.040	65	.045	80	.050	100	.055	120
3°15′	.041	65	.046	85	.052	105	.057	125
3°30′	.043	75	.048	90	.054	115	.058	130
3°45′	.045	80	.050	95	.055	115	.059	130
4°00′	.046	80	.052	105	.057	125	.060	135
4°15′	.048	90	.053	105	.058	125	D_{max} = 4°15′	
4°30′	.049	90	.054	110	.059	130		
4°45′	.050	95	.055	115	.060	135		
5°00′	.051	100	.056	115	.060	135		
5°30′	.054	105	.058	125	D_{max} = 5°15′			
6°00′	.055	110	.059	125				
6°30′	.057	115	.060	130				
7°00′	.058	120	D_{max} = 6°45′					
7°30′	.059	120						
8°00′	.060	125						
8°30′	.060	125						
	D_{max} = 8°30′							

e_{max} = 0.06

Notes:

1. Superelevation rates are based on open-road conditions. See Chapter Six.

2. Transition lengths for freeway/ramp exits are calculated by :

$$L_{F/R} = (S - .02) \times 15 \times RS_{avg}$$

Where:

$L_{F/R}$ = superelevation transition length, ft
S = superelevation rate for exit curve
.02 = typical cross slope of mainline
15 = typical ramp width, ft
RS_{avg} = average relative longitudinal slope between mainline design speed and exit curve design speed. See Table 6.2F.

Desirably, $L_{F/R}$ will be based on two seconds of travel time using a travel speed equal to the average between the mainline design speed and the exit curve design speed. See Chapter Six for more information on superelevation transitions.

Table 10.3D

SUPERELEVATION DEVELOPMENT
(Freeway/Ramp Exits, V=70 mph (Mainline))

D Exit Curve	V=50 mph Exit Curve		V=55 mph Exit Curve		V=60 mph Exit Curve	
	S	Trans. Length (ft)	S	Trans. Length (ft)	S	Trans. Length (ft)
3°00′	.045	85	.050	105	.055	125
3°15′	.046	90	.052	115	.057	135
3°30′	.048	95	.054	120	.058	135
3°45′	.050	105	.055	125	.059	140
4°00′	.052	110	.057	130	.060	145
4°15′	.053	115	.058	135	D_{max} = 4°15′	
4°30′	.054	115	.059	140		
4°45′	.055	120	.060	140		
5°00′	.056	125	.060	140		
5°30′	.058	130	D_{max} = 5°15′			
6°00′	.059	135			e_{max} = 0.06	
6°30′	.060	135				
	D_{max} = 6°45′					

Notes:

1. Superelevation rates are based on open-road conditions. See Chapter Six.

2. Transition lengths for freeway/ramp exits are calculated by :

$$L_{F/R} = (S - .02) \times 15 \times RS_{avg}$$

Where:

$L_{F/R}$ = superelevation transition length, ft
S = superelevation rate for exit curve
.02 = typical cross slope of mainline
15 = typical ramp width, ft
RS_{avg} = average relative longitudinal slope between mainline design speed and exit curve design speed. See Table 6.2F.

Desirably, $L_{F/R}$ will be based on two seconds of travel time using a travel speed equal to the average between the mainline design speed and the exit curve design speed. See Chapter Six for more information on superelevation transitions.

0.08) will be used to determine the proper superelevation rate for horizontal curves. The designer will use the ramp design speed and the degree of curve to read into the tables to determine "S", subject to D_{max} for the ramp design speed.

3. Transition Length. Where the mainline is on tangent or curving in the same direction as the exit ramp, the superelevation transition lengths presented in Tables 10.3C and 10.3D will be used to transition the exit ramp from the mainline cross slope (for a taper exit) or from the parallel lane cross slope (for a parallel-lane exit) to the superelevation rate at the PC assuming the cross slope is 2%.

   Where the mainline is curving away from the exit ramp, a parallel-lane design will ease the superelevation transition development. Where a taper exit design is used, the superelevation transition length will transition the ramp from the mainline cross slope at the exit ramp departure to the superelevation rate at the PC. The designer should use a variation of the basic equation to determine the transition length:

$$L_{F/R} = (S_1 + S_2) \times W \times RS_{avg} \qquad (10\text{-}1)$$

where:

$L_{F/R}$ = superelevation transition length (from S on mainline to S on ramp), ft

S_1 = full superelevation rate of the exit ramp curve (use absolute value)

S_2 = superelevation of mainline at the exit ramp departure (use absolute value)

W = 15 ft

RS_{avg} = relative longitudinal gradient based on the average design speed between highway mainline and exit ramp curve (see Table 6.2F)

* * * * * * * * * *

Example 10-2

Given: Design speed highway mainline = 70 mph
Design speed exit ramp curve = 50 mph
Left-curving mainline
Right-side exit ramp
S_1 = .045
S_2 = .020 (mainline superelevation rate)

Problem: Determine superelevation transition length for a taper exit ramp.

Solution: RS_{avg} is calculated by averaging RS for 70 mph and 50 mph. From Table 6.2F, this yields 225. Using Equation 10-1:

$$L_{F/R} = (S_1 + S_2) \times W \times RS_{avg}$$
$$L_{F/R} = (.045 + .028) \times 15 \times 225$$
$$L_{F/R} = 247'$$

Desirably, $L_{F/R}$ will be based on 2 seconds of travel time using the average between the mainline design speed and the exit curve design speed. See Chapter Six for more information on superelevation transitions.

* * * * * * * * * *

3. Distribution. The superelevation transition length should be distributed

such that 75% of the length is in advance of the PC and 25% of the length is beyond the PC. However, at freeway/ramp junctions, field conditions may make this distribution impractical, and a different distribution may be necessary. The distribution may also be adjusted to provide smooth ramp edge profiles.

4. Axis of Rotation. The axis of rotation is typically about the baseline which is normally the right edge of the ramp travelway. For taper exits, the rotation will desirably begin where the taper wedge is 12-ft wide. It may begin where the wedge is 2-ft wide. For parallel-lane exits, the axis of rotation may be anywhere beyond the entrance taper.

10.3.1.6 Cross Slope Rollover

The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the exit lane and/or gore. The following will apply:

1. Up to Physical Gore. The cross slope rollover should not exceed a range of 4%-5%.

2. From Physical Gore to Gore Nose. The cross slope rollover should not exceed 8%.

3. Drainage Inlets. Where required, these are normally placed between the physical gore and gore nose. The presence of drainage inlets may require two breaks in the gore cross slope. These breaks should meet the criteria in Comment #'s 1 or 2, depending on the inlet location.

10.3.1.7 Shoulders

The right shoulder of the mainline will be transitioned to the narrower shoulder of the ramp. As illustrated in the typical exit ramp figures (uncurbed), the shoulder width along the mainline will be maintained until the physical nose. The shoulder width will then be transitioned to the ramp right shoulder width (typically 8 ft) at a rate of Z:1 based on the design speed of the horizontal curve at the freeway/ramp junction. Criteria for Z are presented in Figures 10.3A and 10.3B.

In restricted areas, it is acceptable to provide an 8-ft minimum right shoulder through the exit taper area.

10.3.1.8 Gore Area

The gore area is normally considered to be both the paved triangular area between the through lane and the exit ramp, plus the graded area which may extend a few hundred feet downstream beyond the gore nose. The following definitions will apply (see Figures 10.3A and 10.3B):

1. Painted Nose. This is the point (without width) where the pavement striping on the left side of the ramp converges with the stripe on the right side of the mainline travelway.

2. Physical Gore. This is the point where the ramp and mainline shoulders converge, and the point where the full-depth travelway pavement structure ends. As illustrated in Figures 10.3A and 10.3B, the physical gore has a dimensional width of 14 ft.

3. Gore Nose. This is the point where the paved shoulder ends and the sodded area begins as the ramp and mainline diverge from one another. As illustrated in

Figures 10.3A and 10.3B, the gore nose has a dimensioned width of 10 ft between paved shoulders.

The following should be considered when designing the gore:

4. Obstacles. If practical, the area beyond the gore nose should desirably be free of all obstacles (except the ramp exit sign) for approximately 100 ft beyond the gore nose. Any obstacles within 300 ft of the gore nose will be breakaway or shielded by a barrier. (See Chapter Eleven).

5. Side Slopes. The graded area beyond the gore nose should be as flat as practical. If the elevation between the exit ramp or loop and the mainline increases rapidly, this may not be practical. These areas will likely be non-traversable and the gore design must shield these areas from the motorist. At some sites, the vertical divergence of the ramp and mainline will warrant protection for both roadways beyond the gore (see Chapter Eleven).

6. Cross Slopes. The paved triangular gore area between the through lane and exit ramp should be safely traversable. The cross slope is the same as that of the mainline (typically 2% on tangent) from the painted nose up to where the gore width is 2-ft wide. Beyond this point, the gore 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 ramp. See Section 10.3.1.6 for criteria on breaks in cross slopes within the gore area.

7. Traffic Control Devices. Signing in advance of the exit and at the divergence should be according to the MUTCD. This also applies to the pavement markings in the triangular area upstream from the gore nose.

10.3.1.9 Escape Taper

When a lane is being dropped at the exit, such as a continuous auxiliary lane, special design considerations are necessary at the gore area of the exit. An area needs to be provided to allow a driver, who may have missed the signing, an opportunity to merge left into the through lanes. Figure 10.3E provides the typical details on how to properly design the gore area to provide these drivers with an escape taper.

10.3.2 Entrance Ramps

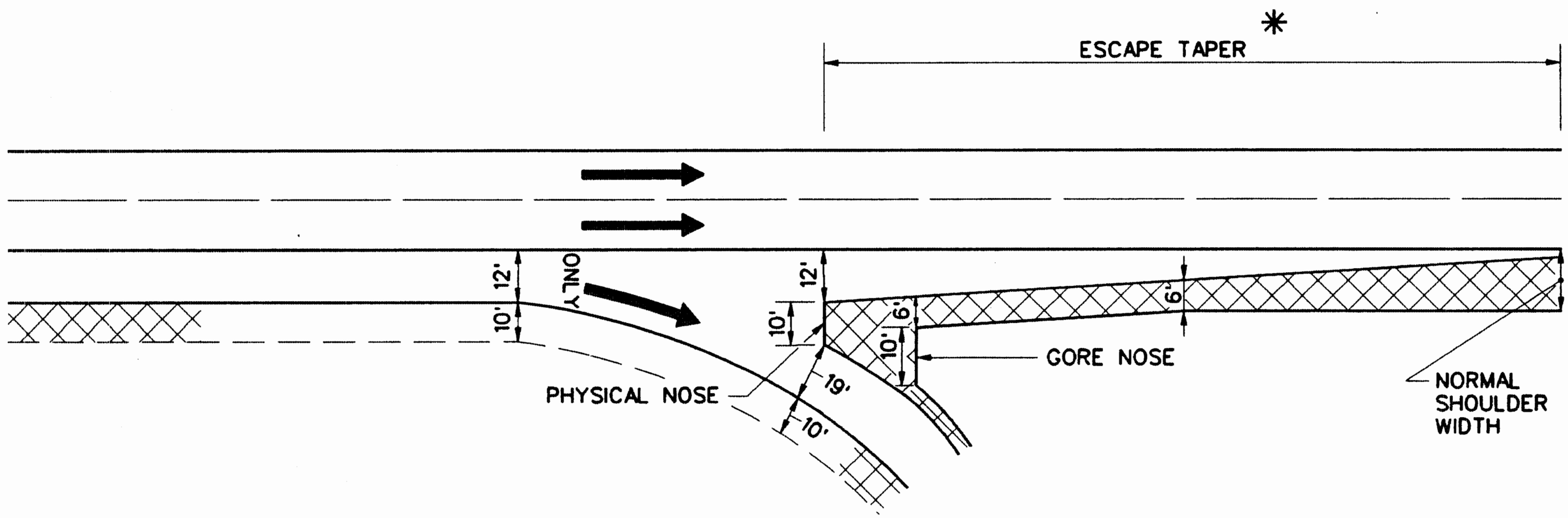
10.3.2.1 Types

ODOT uses two types of entrance freeway/ramp junctions -- the taper design and the parallel design. ODOT's general practice is that the taper design should be used whenever practical. However, there are special cases where a parallel design may be more practical. The following lists a few examples:

1. Where the LOS for the freeway/ramp merge approaches capacity, a parallel design may be used to allow the driver more time to merge into the through traffic.

2. Where the acceleration length needs to be lengthened for grades and/or trucks, the parallel design provides longer distances more easily than a taper design.
3. Where there is insufficient sight distance available for the driver to merge into the mainline (e.g., where there are sharp curves on the mainline), the parallel entrance ramp allows a driver to use the side-view and rear-view mirrors to locate gaps in the mainline traffic.

4. Where the need has been satisfied for a continuous auxiliary lane, a parallel-lane



✱ DESIRABLE LENGTH BASED ON DESIGN SPEED:1
MINIMUM LENGTH BASED ON 50:1

Notes:

1. Figure applies to an uncurbed gore section. See Figure 10.3D for details on a curbed gore.

2. Refer to Traffic Engineering Division for details on pavement markings.

ESCAPE TAPER
(Lane Drop at Exit Ramp)

Figure 10.3E

entrance is appropriate. See Section 10.3.3.

The following figures illustrate ODOT's design details for entrance ramps:

5. Figure 10.3F illustrates the ODOT typical taper entrance design for an uncurbed ramp. The design speed of the mainline is 60 mph, and the design speed of the approach curve on the entrance ramp is 45 mph or higher. NOTE: 50 mph should be the minimum ramp speed for this design; 45 mph may be used in restricted locations.

6. Figure 10.3G illustrates the ODOT typical taper entrance design for an uncurbed ramp. The design speed of the mainline is 70 mph, and the design speed of the approach curve on the entrance ramp is 50 mph or higher.

7. Figure 10.3H illustrates the ODOT typical parallel-lane entrance design for an uncurbed ramp.

8. Figure 10.3I illustrates the ODOT typical gore design for a curbed entrance ramp.

10.3.2.2 Taper Rates

The taper entrance design merges into the freeway with a long, uniform taper (see Figures 10.3F and G).

For parallel-lane entrance ramps, the taper at the merge point is 300 ft minimum (see Figure 10.3H). Where a parallel acceleration lane exceeds 1300 ft, uniform 50:1 to 70:1 tapers are recommended.

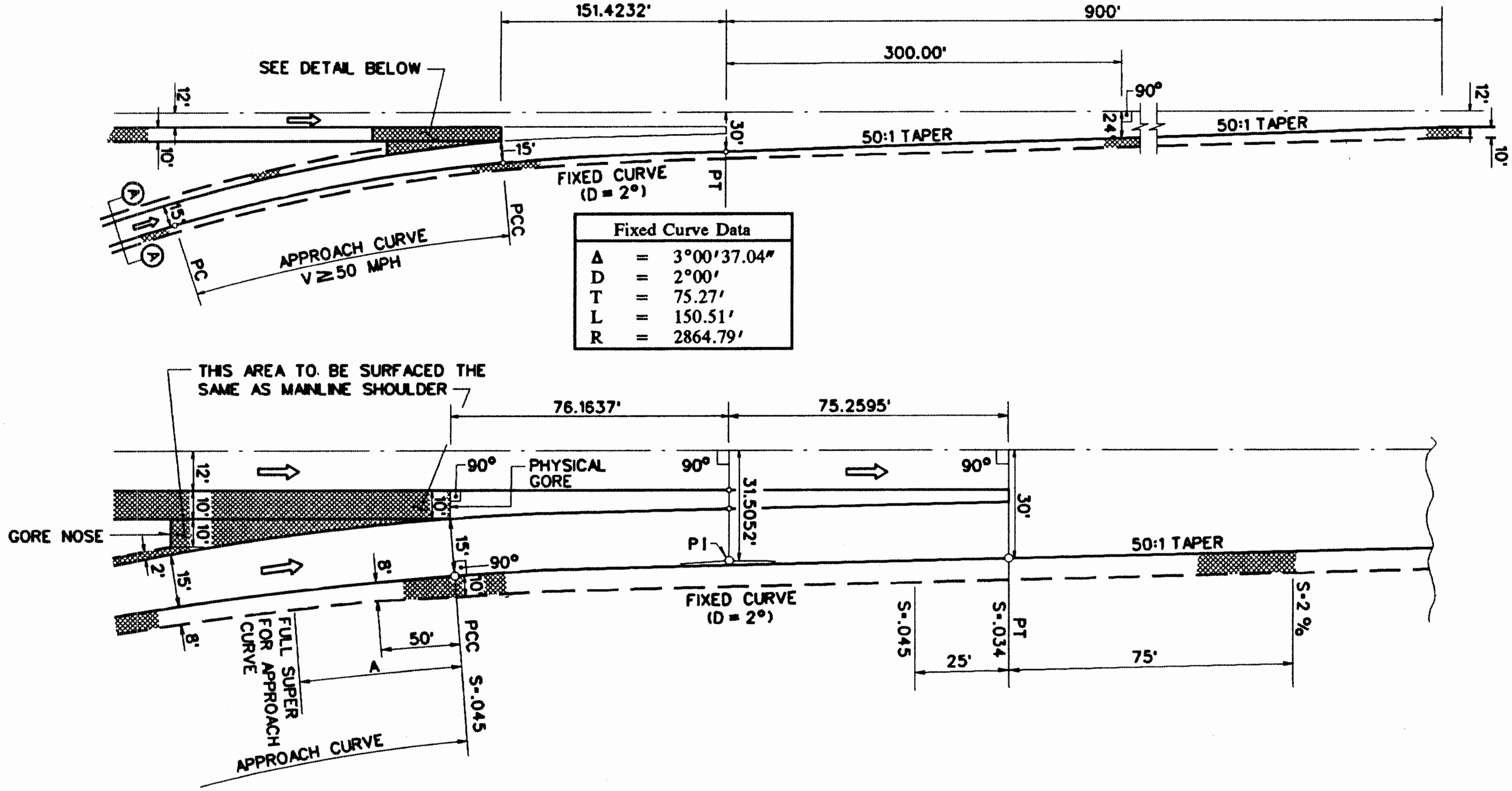
10.3.2.3 Acceleration

Driver comfort, traffic operations and safety will be improved if sufficient distance is available for acceleration. The length for acceleration will primarily depend upon the design speed of the last controlling horizontal curve on the entrance ramp and the design speed of the mainline. On the ODOT typical entrance design figures, this is called the approach curve.

Table 10.3E provides minimum lengths of acceleration for passenger cars. For a taper design, the acceleration distance is from the last controlling curve to the point where the lane narrows down to 12 ft (see Figures 10.3F and G). For a parallel-lane design, the acceleration distance is measured from the PC of the last controlling curve to the beginning of the taper (see Figure 10.3H). Where upgrades of 3% or more occur over the acceleration distance, adjustments should be considered in its length according to Table 10.3F.

The values in Table 10.3E provide sufficient distance for vehicular acceleration of passenger cars. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merging area, the available acceleration distance should desirably total 1200 ft, exclusive of the taper, to provide additional merging opportunities. This distance is measured from the PT of the ramp entrance curve.

Where there are a significant number of trucks to govern the design of the ramp, the truck acceleration distances provided in Table 10.3G should be considered. Typical areas where trucks might govern the ramp design will include weigh stations, truck stops, rest areas 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:



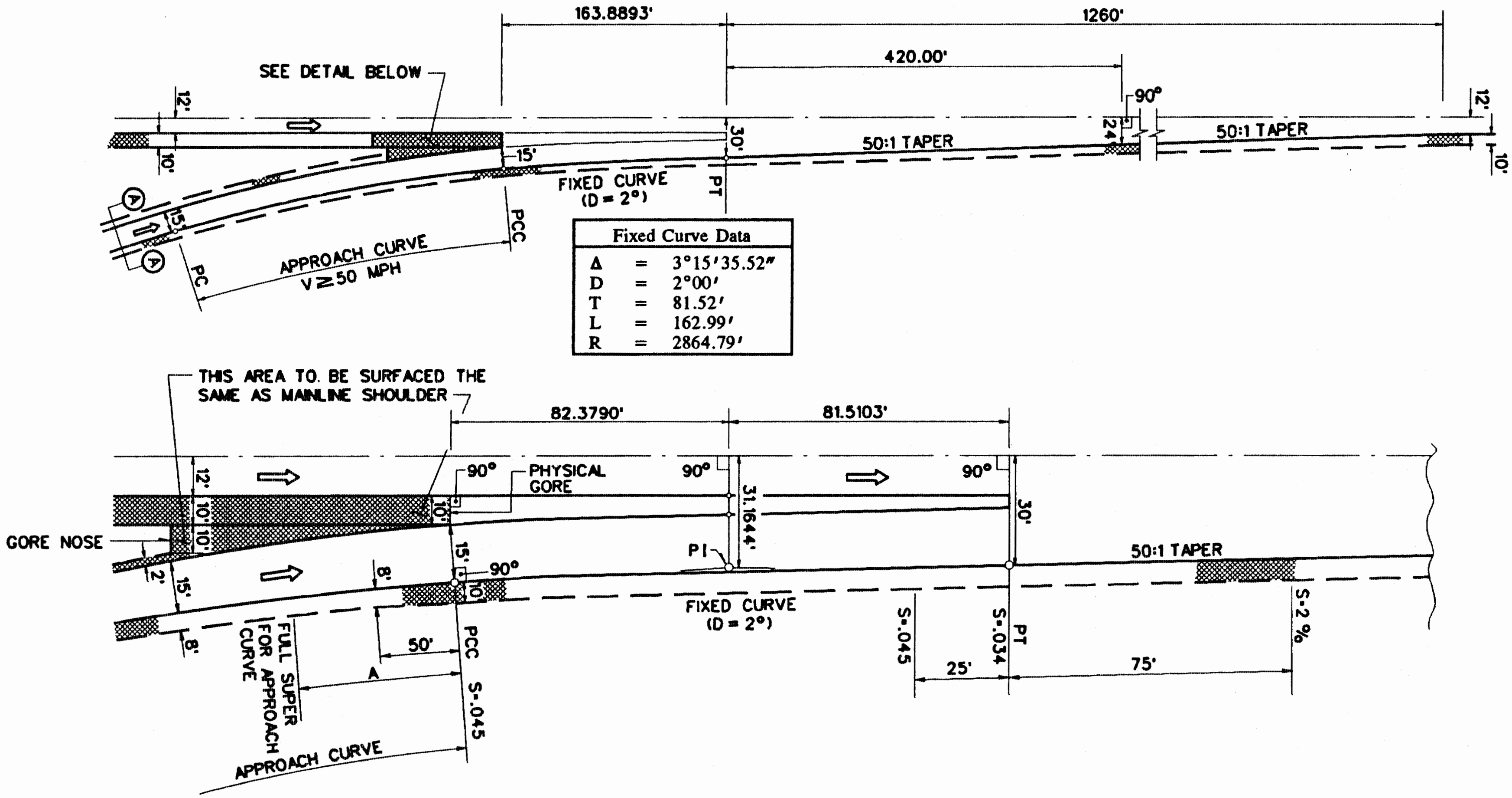
Fixed Curve Data		
Δ	=	3°00'37.04"
D	=	2°00'
T	=	75.27'
L	=	150.51'
R	=	2864.79'

Notes:

1. Figure applies where V=60 mph for mainline and V≥50 mph for approach curve. In restricted locations, the approach curve may be designed for V=45 mph.
2. See Table 10.3H for superelevation information on the approach curve, including the dimension "A".
3. See Figure 10.3I for typical curbed entrance gore.
4. See Figure 10.4A for typical ramp section.
5. See Figure 10.3L for entrances on curvilinear alignment.
6. A 300' minimum length of approach curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL TAPER ENTRANCE RAMP
(Uncurbed, V=60 on Mainline)

Figure 10.3F



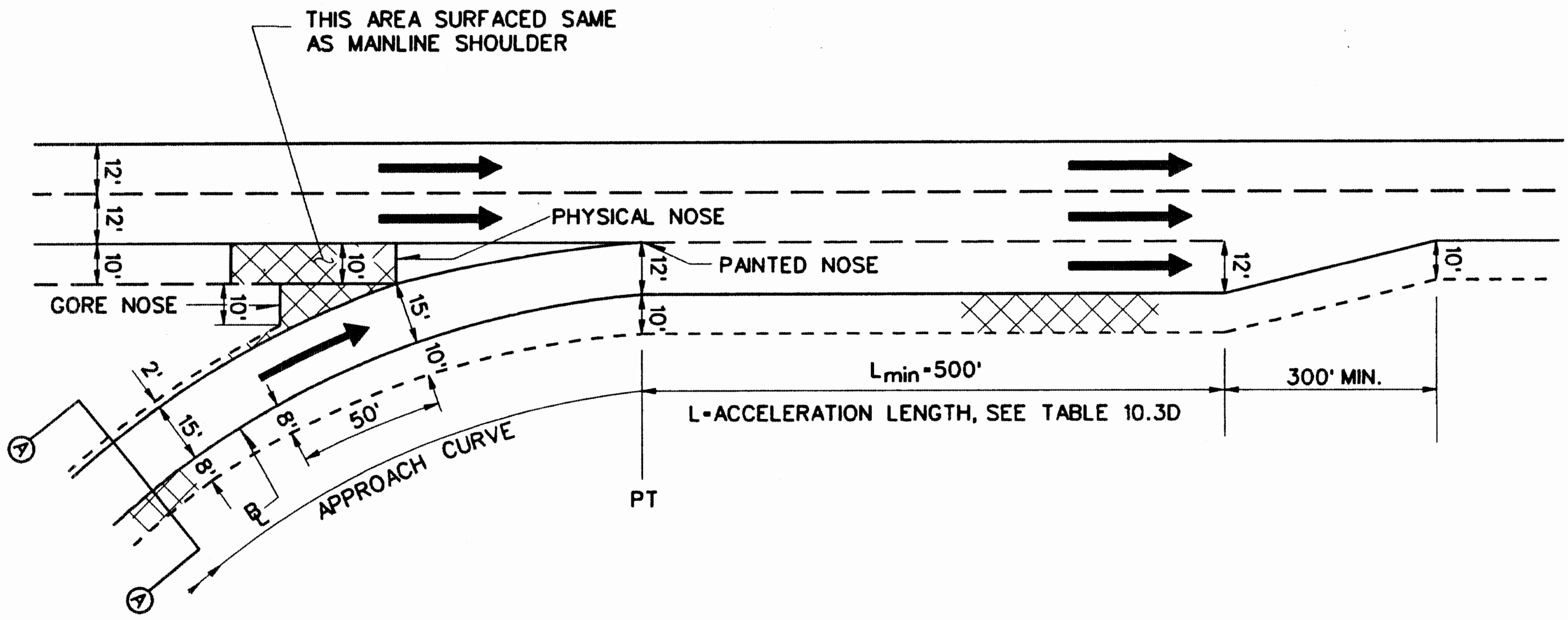
Fixed Curve Data		
Δ	=	3°15'35.52"
D	=	2°00'
T	=	81.52'
L	=	162.99'
R	=	2864.79'

Notes:

1. Figure applies where V=70 mph for mainline and V≥50 mph for approach curve.
2. See Table 10.3H for superelevation information on the approach curve, including the dimension "A".
3. See Figure 10.3I for typical curbed entrance gore.
4. See Figure 10.4A for typical ramp section.
5. See Figure 10.3L for entrances on curvilinear alignment.
6. A 300' minimum length of approach curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL TAPER ENTRANCE RAMP
(Uncurbed, V=70 on Mainline)

Figure 10.3G

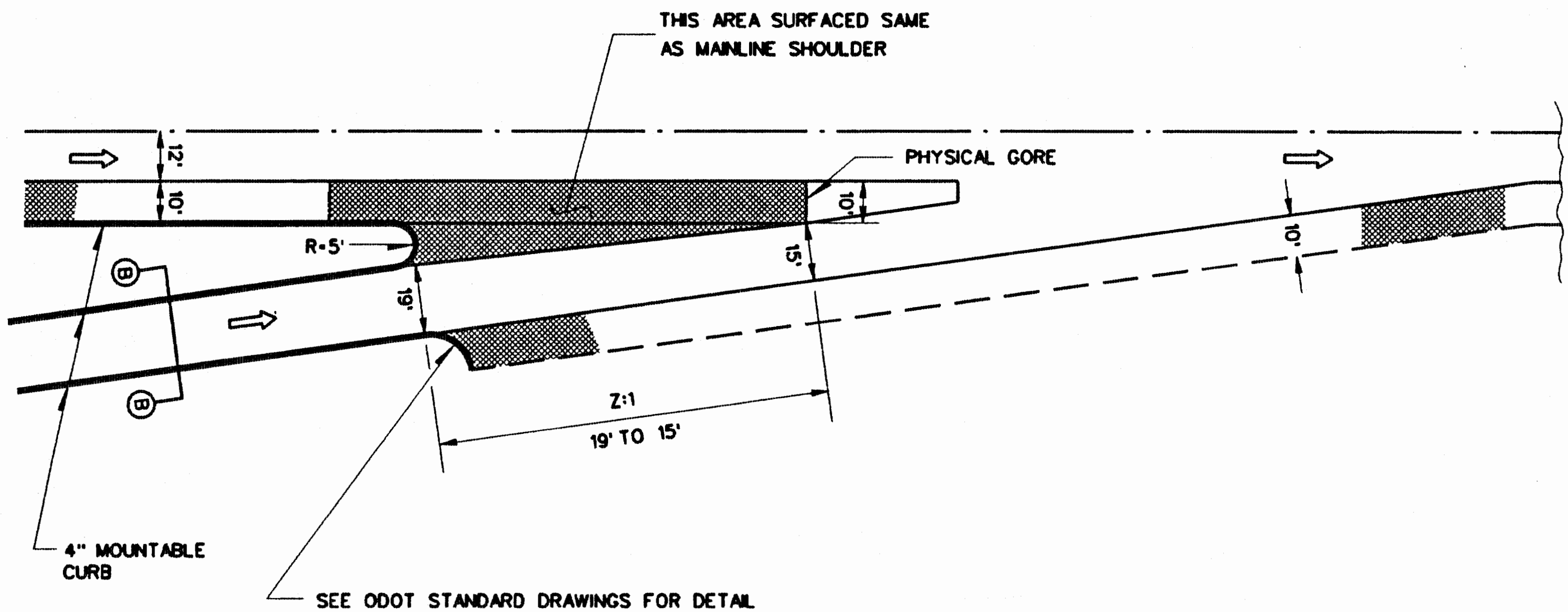


Notes:

1. Figure applies where taper design is inappropriate.
2. See Table 10.3C for superelevation information.
3. See Figure 10.3I for typical curbed entrance gore.
4. See Figure 10.4A for typical ramp section.
5. A 300′ minimum length of approach curve is desirable.
6. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL PARALLEL-LANE ENTRANCE RAMP
(Uncurbed)

Figure 10.3H



Design Speed of Ramp Curve (mph)	Z – Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where curb is warranted on ramp. See Section 10.4.
2. See Figure 10.4A for typical ramp section.
3. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL ENTRANCE GORE (Curbed)

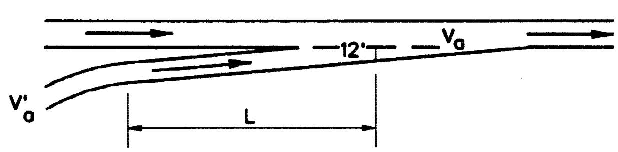
Figure 10.3I

Table 10.3E

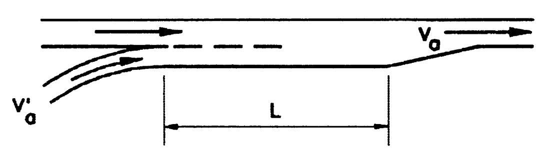
LENGTHS FOR ACCELERATION*
(Passenger Cars)

Highway Design Speed (mph) (V)	Speed Reached (mph) (V_a)	L = Acceleration Length (ft)								
		For Design Speed Of First Governing Geometric Control (mph) (V′)								
		Stop	15	20	25	30	35	40	45	50
		For Average Running Speed (mph) (V'_a)								
		0	14	18	22	26	30	36	40	44
30	23	190	-	-	-	-	-	-	-	-
40	31	380	320	250	220	140	-	-	-	-
50	39	760	700	630	580	500	380	160	-	-
60	47	1,170	1,120	1,070	1,000	910	800	590	400	170
70	53	1,590	1,540	1,500	1,410	1,330	1,230	1,010	830	580

* These values are for grades less than 3%. See Table 10.3F for steeper upgrades or downgrades.



TAPER TYPE



PARALLEL TYPE

Note: 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 5 mph below the average running speed on the mainline. The theoretical basis for the acceleration distances is presented on pages 351-355 of Reference (4).

Table 10.3F

SUGGESTED GRADE ADJUSTMENTS FOR ACCELERATION

Design Speed of Highway (mph)	Ratio of Length on Grade to Length on Level For Design Speed of Ramp Curve (mph)				
	20	30	40	50	All Speeds*
	$3\% \leq G \leq 4\%$ (Upgrade)				$3\% \leq G \leq 4\%$ (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
	$G > 4\%$ (Upgrade)				$G > 4\%$ (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

* The downgrade adjustment may only be used 1) where the truck acceleration distances in Table 10.G apply, or 2) for 3R freeway projects.

Notes:

1. No adjustment is needed on grades less than 3%.

2. The "grade" in the table is the <u>average</u> grade measured over the distance for which the acceleration length applies.

<u>Example</u>

Given:

Highway Design Speed	-	70 mph
Entrance Ramp Curve Design Speed	-	40 mph
Average Grade	-	5% upgrade

Problem: Determine length of acceleration.

Solution: Table 10.3F yields an acceleration length of 1010 ft on the level. According to Table 10.3F, this should be increased by 2.6.

Therefore: L = 1010 x 2.6
L = 2626 ft

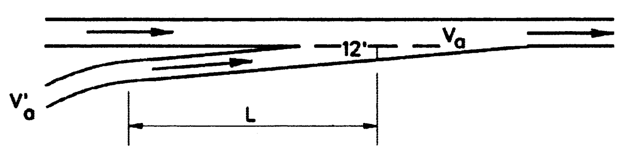
A 2626-ft acceleration length should desirably be provided from the PT of the entrance ramp curve to the last point at which the full width of the entrance ramp is available. However, the 1010-ft acceleration distance would be acceptable; the additional distance would be appropriate if judged to be practical and cost effective.

Table 10.3G

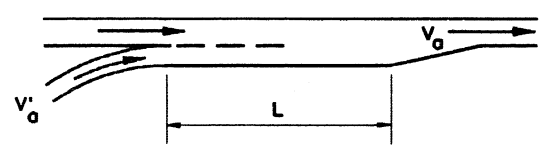
LENGTHS FOR ACCELERATION (300 lb/hp Trucks)

Highway Design Speed (mph) (V)	Speed Reached (mph) (V_a)	L = Acceleration Length (ft)								
		For Entrance Curve Design Speed (mph) (V′)								
		Stop	15	20	25	30	35	40	45	50
		And Initial Speed (mph) (V'_a)								
		0	14	18	22	26	30	36	40	44
30	18	150*	-	-	-	-	-	-	-	-
40	26	350*	300*	250*	150*	-*	-	-	-	-
50	34	900	850	800	700	600	450	-	-	-
60	42	2,200	2,150	2,100	2,000	1,900	1,800	1,000	900	-
70	48	3,700	3,700	3,700	3,600	3,400	3,300	2,700	1,950	1,100

* For these speeds, the minimum lengths for passenger cars in Table 10.3E will apply.



TAPER TYPE



PARALLEL TYPE

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 a speed of 10 mph below the average running speed on the mainline.

1. there is LOS D or worse at the junction,
2. there is a significant accident history involving trucks which can be attributed to an inadequate acceleration length, and/or
3. there is an undesirable level of vehicular delay at the junction attributed to an inadequate acceleration length.

Where the upgrades exceed 3%, the truck acceleration distances may be corrected for grades as provided in Table 10.3F. Before providing any additional acceleration lane length, the designer must consider the impacts of the added acceleration length (e.g., additional construction costs, wider structures, right-of-way impacts).

The specific application of the acceleration criteria to horizontal curves is as follows:

1. The design speed of the last horizontal curve on the ramp proper will be determined by open-highway conditions (e.g., side-friction factors (f) and the distribution method between f and superelevation). These are discussed in Chapter Six.
2. Based on the design speeds of the highway and the ramp curve, Table 10.3E or Table 10.3G will yield the required length of acceleration.
3. For relatively short entrance ramps, the acceleration distance may be determined by that distance needed to accelerate from zero (at the beginning of the ramp) to the mainline design speed. The designer should check to determine if this distance governs.

10.3.2.4 Sight Distance

Decision sight distance should desirably be provided for drivers on the mainline approaching an entrance terminal. They need sufficient distance to see the merging traffic so they can adjust their speed or change lanes to allow the merging traffic to enter the freeway. Likewise, drivers on the entrance ramp need to see a sufficient distance upstream from the entrance to locate gaps in the traffic stream for merging.

10.3.2.5 Superelevation

The entrance ramp superelevation should be gradually transitioned to meet the normal cross slope of the mainline. The principles of superelevation for open highways, as discussed in Chapter Six, should be applied to the entrance design. Figures 10.3F and G illustrate the superelevation application at freeway/ramp entrances. Table 10.3H presents detailed superelevation development criteria which apply to entrance ramps. Section 10.3.1 also discusses superelevation criteria at the freeway/ramp junction. This includes e_{max}, superelevation rate, transition lengths, the distribution of transition lengths between curve and tangent, and the axis of rotation.

10.3.2.6 Cross Slope Rollover

The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance lane, where these two are adjacent to each other. The maximum algebraic difference is 4% - 5% beyond the physical gore. Between the gore nose and physical gore, the maximum cross slope rollover is 8%.

Table 10.3H

SUPERELEVATION DEVELOPMENT
(Freeway/Ramp Entrance)

D Approach Curve	V=45 mph Approach Curve		V=50 mph Approach Curve		V=55 mph Approach Curve		V=60 mph Approach Curve	
	S	A (ft)	S	A (ft)	S	A (ft)	S	A (ft)
3°00′	.040	30	.045	-	.050	30	.055	35
3°15′	.041	30	.046	30	.052	30	.057	40
3°30′	.043	30	.048	30	.054	30	.058	45
3°45′	.045	-	.050	30	.055	35	.059	50
4°00′	.046	30	.052	30	.057	40	.060	50
4°15′	.048	30	.053	30	.058	45	D_{max} = 4°15′	
4°30′	.049	30	.054	30	.059	50		
4°45′	.050	30	.055	35	.060	50		
5°00′	.051	30	.056	40	.060	50		
5°30′	.054	30	.058	45	D_{max} = 5°15′			
6°00′	.055	35	.059	50				
6°30′	.057	40	.060	50				
7°00′	.058	45	D_{max} = 6°45′					
7°30′	.059	50						
8°00′	.060	50						
8°30′	.060	50						
	D_{max} = 8°30′							

e_{max} = 0.06

Notes:

1. Superelevation rates are based on open-road conditions. See Chapter Six.

2. Transition lengths "A" are calculated by :

$$A = (S - .045) \times 15 \times 222$$

Where:

A = distance needed to transition from superelevation rate of approach curve to superelevation rate of fixed curve (.045), ft

15 = typical ramp width, ft

222 = relative longitudinal slope for design speed of fixed curve (60 mph)

10.3.2.7 Shoulder Transitions

At entrance terminals, the right hand shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder. Figures 10.3F and 10.3G illustrate the typical shoulder transition, which occurs just before the PCC at the physical gore. In restricted areas, it is acceptable to maintain the 8-ft right shoulder width throughout the acceleration taper (or parallel lane) until the merge with the mainline.

10.3.3 Continuous Auxiliary Lane

For closely spaced interchanges, it may be warranted to provide a continuous auxiliary lane between the entrance ramp of one interchange and the exit ramp of the downstream interchange. A continuous auxiliary lane should be considered where:

1. the distance between the end of the entrance taper (without the connecting auxiliary lane) and the beginning of the downstream exit taper would be less than 1000 ft, or

2. a capacity and operational analysis indicates the need.

Where the continuous lane is used, both freeway/ramp junctions should be designed based on the parallel-lane type exit and entrance.

10.3.4 Freeway/Ramp Junctions on Curves

The previous discussions on freeway/ramp junctions assumed tangent alignments. Figures 10.3J, 10.3K and 10.3L illustrate schematics for both taper and parallel designs on curves which may be appropriate. The basic objective is to present a layout on curves which have approximately the same operational characteristics as the recommended layouts on tangent.

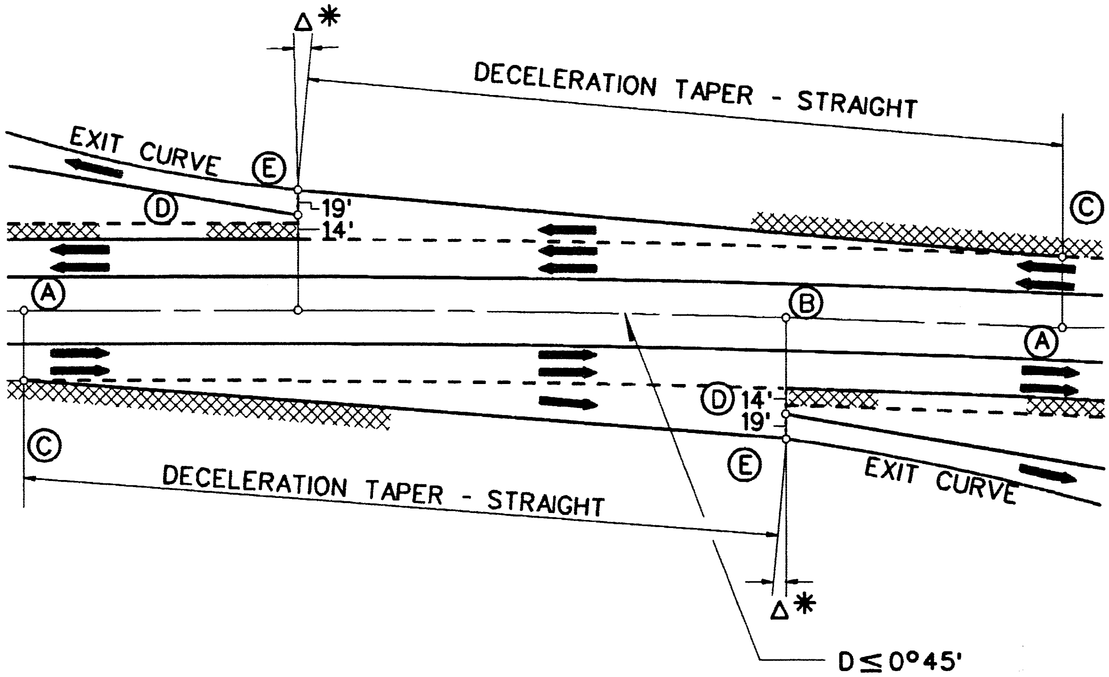
Left-hand exits on horizontal curves may be particularly troublesome. Through traffic in the outside lanes tend to follow the ramp exit causing confusion at the gore. Consequently, left-hand exits on curves should be avoided, if practical. For right-hand exits on left-turning curves, the ramp should not be a direct extension of the mainline tangent. This gives the driver an illusion that the freeway continues up the ramp instead of turning. To eliminate this problem, the exit ramp taper should begin a significant distance before or after the PC.

Superelevation development is another specific problem of freeway/ramp junctions on horizontal curves, especially where the ramp and mainline are curving in opposite directions. In these cases, it may be difficult to coordinate the mainline and ramp design to meet all applicable criteria (e.g., cross slope rollover, gore cross slopes, superelevation transition lengths). As discussed in Section 10.3.1, freeway/ramp junctions on horizontal curves are candidate locations where the parallel-lane design may be more adaptable to exit ramps than the taper design.

Wherever an exit or entrance taper falls on curved alignment, the entire length of the taper should desirably be within the limits of the curve. If the taper is introduced on tangent alignment just upstream from the beginning of the curve, the designer should be careful to minimize the appearance of a kink.

10.3.5 Multilane Terminals

Multilane terminals may be required when the capacity of the ramp is too great for single-lane operation. They may also be used to improve traffic operations (e.g., weaving) at the junction. The following lists several



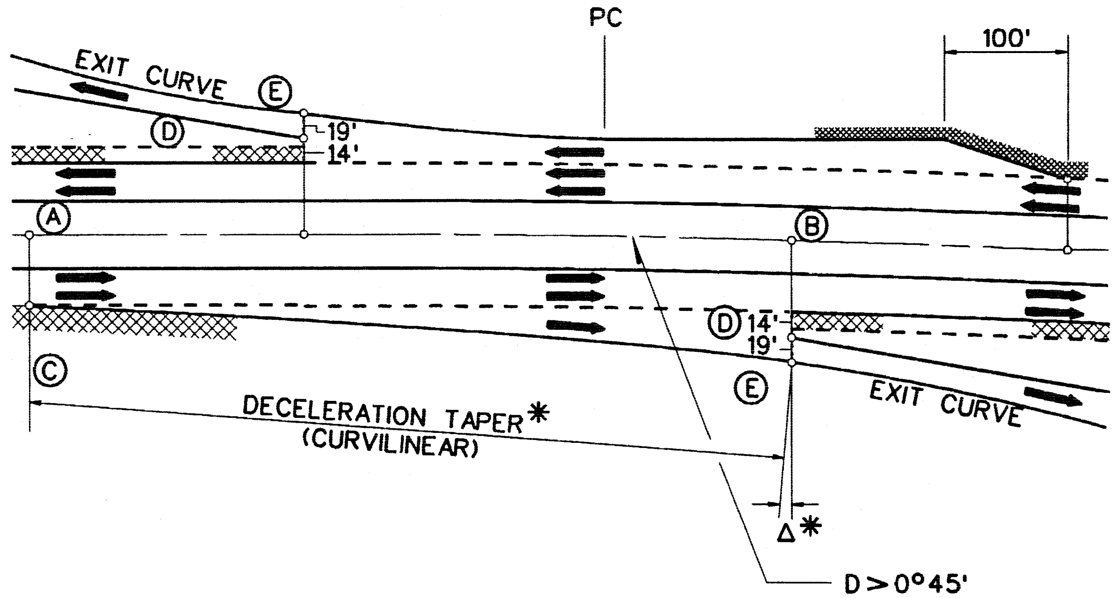
* FOR A DECELERATION TAPER = 500',Δ - 3°46'46.30"
FOR A DECELERATION TAPER = 550',Δ - 3°26' 10.49"

Note: When the main roadway alignment is on a curve whose degree does not exceed 00°45′, the following geometric controls will be required. Points "A" and "B" are located on the centerline of the median or, in the case of a median transition, on the base line of the lane such that the Arc "A" - "B" is 500′ in length for 60 mph and 550′ for 70 mph. Through Point "A", a radial line is passed to obtain Point "C," the beginning of the taper. Similarly, a radial line is passed through "B" to obtain Point "D" 14′ beyond the outside edge of pavement. From Point "D," deflect an angle of Δ and establish Point "E" 19′ beyond Point "D." The straight line connecting Point "C" and Point "E" establishes the bearing and length of the taper and the bearing of the tangent of the Exit Curve.

These geometric controls are to be used in conjunction with the criteria on exit ramps in Section 10.3. See the Geometric Design Branch for more information.

EXIT TERMINALS ON CURVILINEAR ALIGNMENT
(D=00°45′ max)

Figure 10.3J



* FOR A DECELERATION TAPER = 500',Δ = 3°46'46.30' AND OFFSET 6.6' PER STATION
FOR A DECELERATION TAPER = 550',Δ = 3°26'10.49" AND OFFSET 6.0' PER STATION

Note: When the main roadway is on a curve whose degree exceeds 00°45′ minimum, the following geometric controls will be required on the inside of the roadway curve. Points "A" and "B" are located 500′ (or 550′) apart along the arc on the centerline of median or, in the case of a median transition, on the base line of the lane. A radial line is passed through Point "A" to establish Point "C," the beginning of the taper. A radial line is also passed through Point "B" to establish Point "D" 14′ from the edge of pavement. From Point "D," deflect an angle of Δ and establish Point "E" 19′ beyond Point "D."

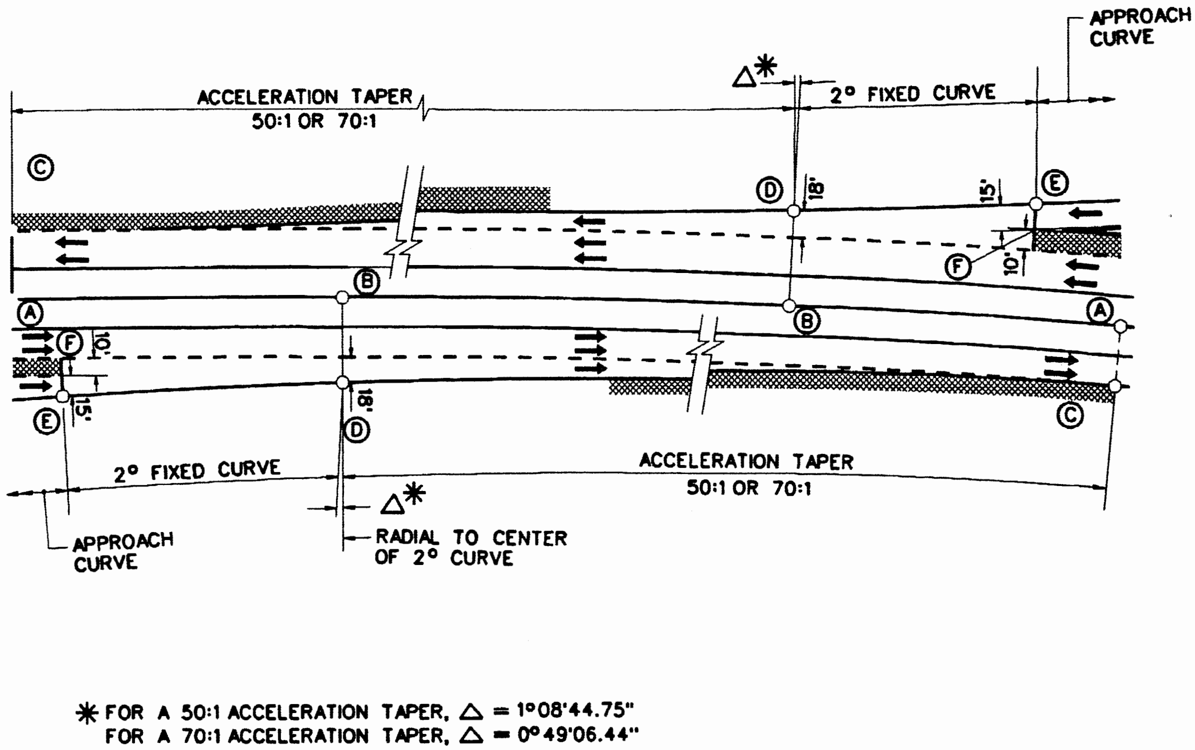
The bearing of the line from Point "D" to Point "E" is the bearing of the radius of the exit curve. The survey data for the ramp will begin at Point "E." The edge line on the inside of the ramp from Point "C" to Point "E" shall be identified as "Offset 6.6 Feet per Station for 60 mph" or "Offset 6.0 Feet per Station for 70 mph."

For the design of exit terminals on the outside of the roadway curve, consult with the Geometric Design Branch. For D>00°45′ and the exit ramp on the outside of the curve, the use of a parallel-lane exit ramp should be considered.

These geometric controls are to be used in conjunction with the criteria on exit ramps in Section 10.3. See the Geometric Design Branch for more information.

EXIT TERMINALS ON CURVILINEAR ALIGNMENT
(D=00°45′ Minimum)

Figure 10.3K



Note: When the main roadway alignment is on a curve, the following geometric controls will be required. Points "A" and "B" are located on the centerline of survey or baseline of construction, such that the arc length "A" to "B" will be either a 900′ arc length for a 50:1 taper or a 1260′ arc length for a 70:1 taper rate. A radial line is passed through "A" to establish Point "C," on outside edge of pavement; also, a radial is passed through "B" to produce Point "D" 18′ beyond the outside edge of main line pavement. Center of 2° curve is established by projecting a deflection angle of Δ from a radial line on ℄ curve passed through Point "D" as shown on detail. Point "E" is established on a radial line on 2° curve passed through F, a point of intersection of an offset curve concentric with curve on base line and an offset curve concentric with 2° curve on ramp.

Above design is applicable where design speed of approach curve is 50 mph or more. Where $V < 50$ mph, a parallel-lane entrance design should be considered.

These controls are to be used in conjunction with the criteria on entrance ramps in Section 10.3. See the Geometric Design Branch for more information.

ENTRANCE TERMINALS ON CURVILINEAR ALIGNMENT

Figure 10.3L

elements the designer needs to consider when a multilane terminal is required:

1. Lane Balance. Lane balance at the freeway/ramp junction should be maintained. See Section 10.1.

2. Entrances. ODOT practice is that, for multilane entrance ramps, a parallel-lane design is typically used. Figure 10.3M illustrates a schematic of a typical multilane entrance ramp.

3. Exits. For a two-lane exit ramp, the additional lane should be added at least 1500 ft prior to the terminal. Either a taper or a parallel-lane design is acceptable (see Figure 10.3N).

4. Signing. The geometric layout of multilane exits must be coordinated with the Traffic Engineering Division because of the complicated signing which may be required in advance of the exit.

10.3.6 Major Fork/Branch Connections

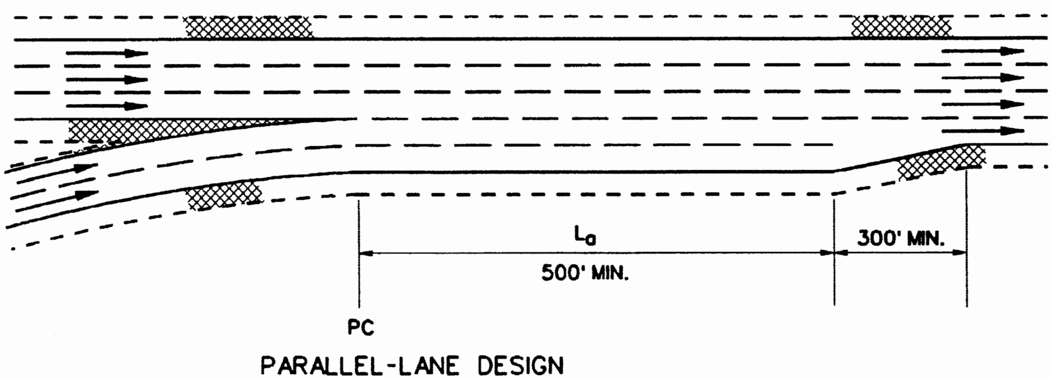
Figures 10.3O and 10.3P illustrate typical design details for a major fork or branch connection. The following lists a few geometric issues that the designer should consider when designing major divisions:

1. Lane Balance. The principle of lane balance should be maintained. See Section 10.1.

2. Divergence Point. Where the alignments of both roadways are on horizontal curves at a major fork, the nose of the split should be placed in direct alignment with the centerline of one of the interior lanes, so that a driver in the center lane has the option of going in either direction. See Schematics A, B and C in Figure 10.3O. Where one of the roadways is on a tangent at a major fork, the gore design should be the same as a freeway/ramp multilane exit. See Figure 10.3N.

3. Nose Width. At the painted nose of a major fork, the lane should be at least 24-ft wide but preferably not over 28 ft. The widening from 12 ft to 24 ft should take place within a distance of 1000 to 1800 ft. See Schematic A in Figure 10.3O.

4. Branch Connection. When merging, a full lane width should be carried for at least 1000 ft beyond the nose. See Schematic B in Figure 10.3P.



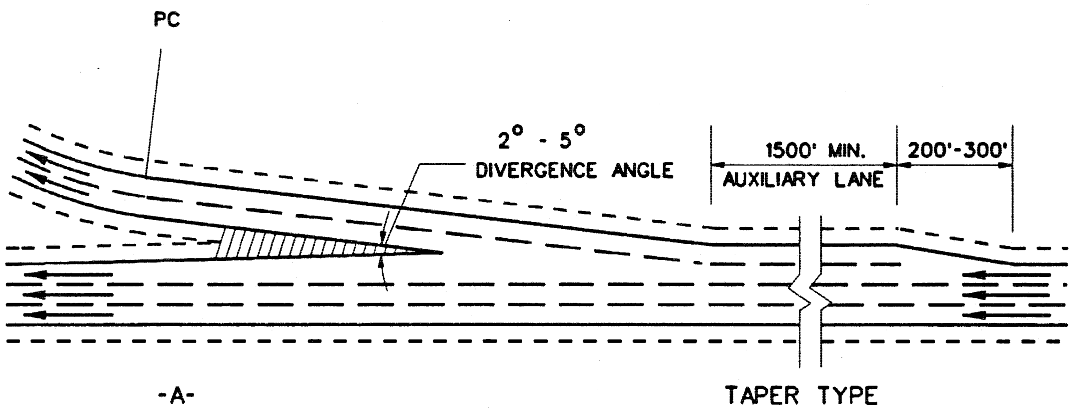
Notes:

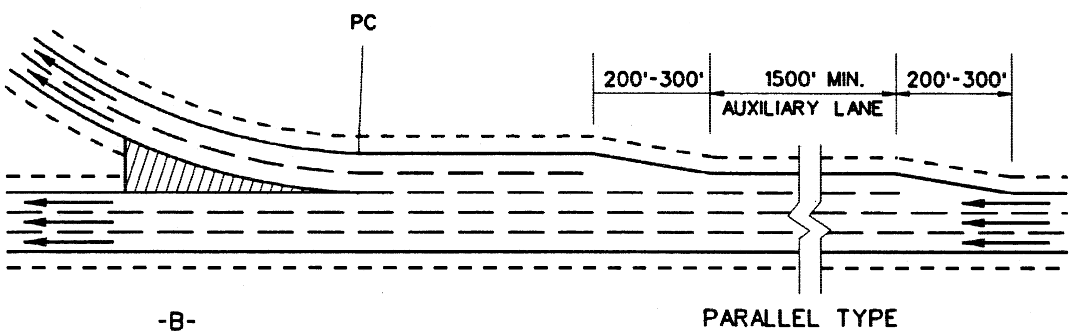
1. L_a is the required acceleration length as shown in Table 10.3E. L_a = 500′ minimum.

2. Typically, a parallel-lane design will be used at multilane entrances. The design details are similar to those for a single-lane entrance. See Figure 10.3H.

MULTILANE ENTRANCE RAMPS

Figure 10.3M





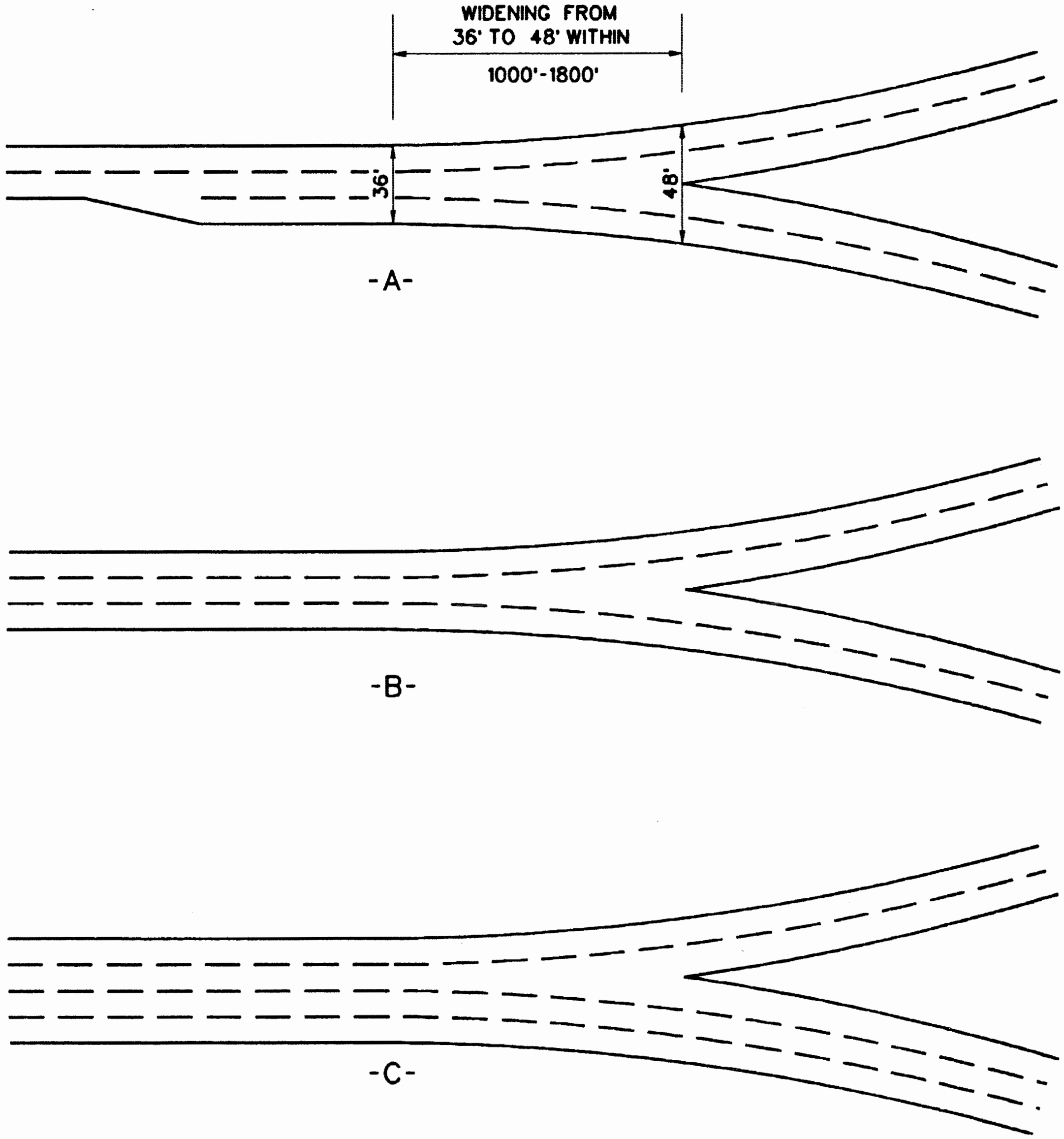
Notes:

1. See Figure 10.3A for gore details.

2. Other design elements (e.g., tapers, shoulder widths) will be determined on a case-by-case basis.

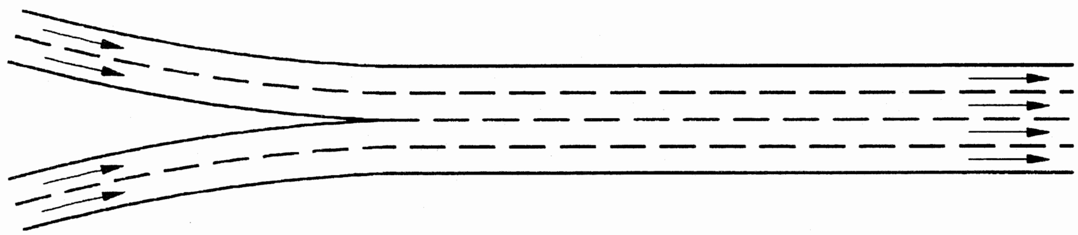
MULTILANE EXIT RAMPS

Figure 10.3N

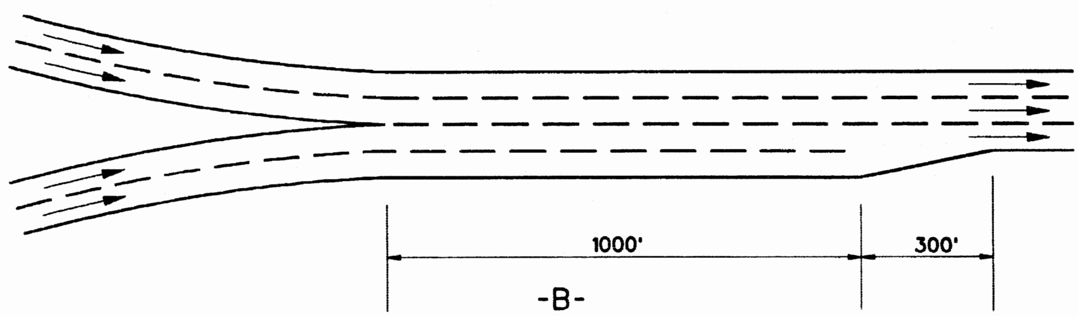


MAJOR FORKS
(Typical Schematics)

Figure 10.3O



-A-



-B-

BRANCH CONNECTIONS
(Typical Schematics)

Figure 10.3P

10.4 RAMP DESIGN

10.4.1 Design Speed

Table 10.4A provides the acceptable ranges of ramp design speed as compared to the design speed on the mainline. These design speeds apply to the ramp proper and not to the freeway/ramp junction. If a ramp will be terminated at an at-grade intersection with stop or signal control, the design speeds in the table may not apply to the ramp portion near the intersection. If the two intersecting mainlines have different design speeds, the higher of the two should control. However, the ramp design speed may vary, the portion of the ramp nearer the lower-speed highway being designed for the lower speed.

In general, the higher design speeds apply to directional ramps for right turns, such as at diamond and fully directional interchanges. The lower design speeds generally apply to loop ramps.

Table 10.4A

RAMP DESIGN SPEEDS

Highway Design Speed (mph)	50	60	65	70
Ramp Design Speed (mph)				
High Range	45	50	55	60
Mid Range	35	45	45	50
Low Range	25	25	25	25

10.4.2 Cross Sections

Figures 10.4A and 10.4B present typical cross sections for ramps on tangents and for adjacent on/off ramps separated by a raised median.

10.4.2.1 Ramp Widths

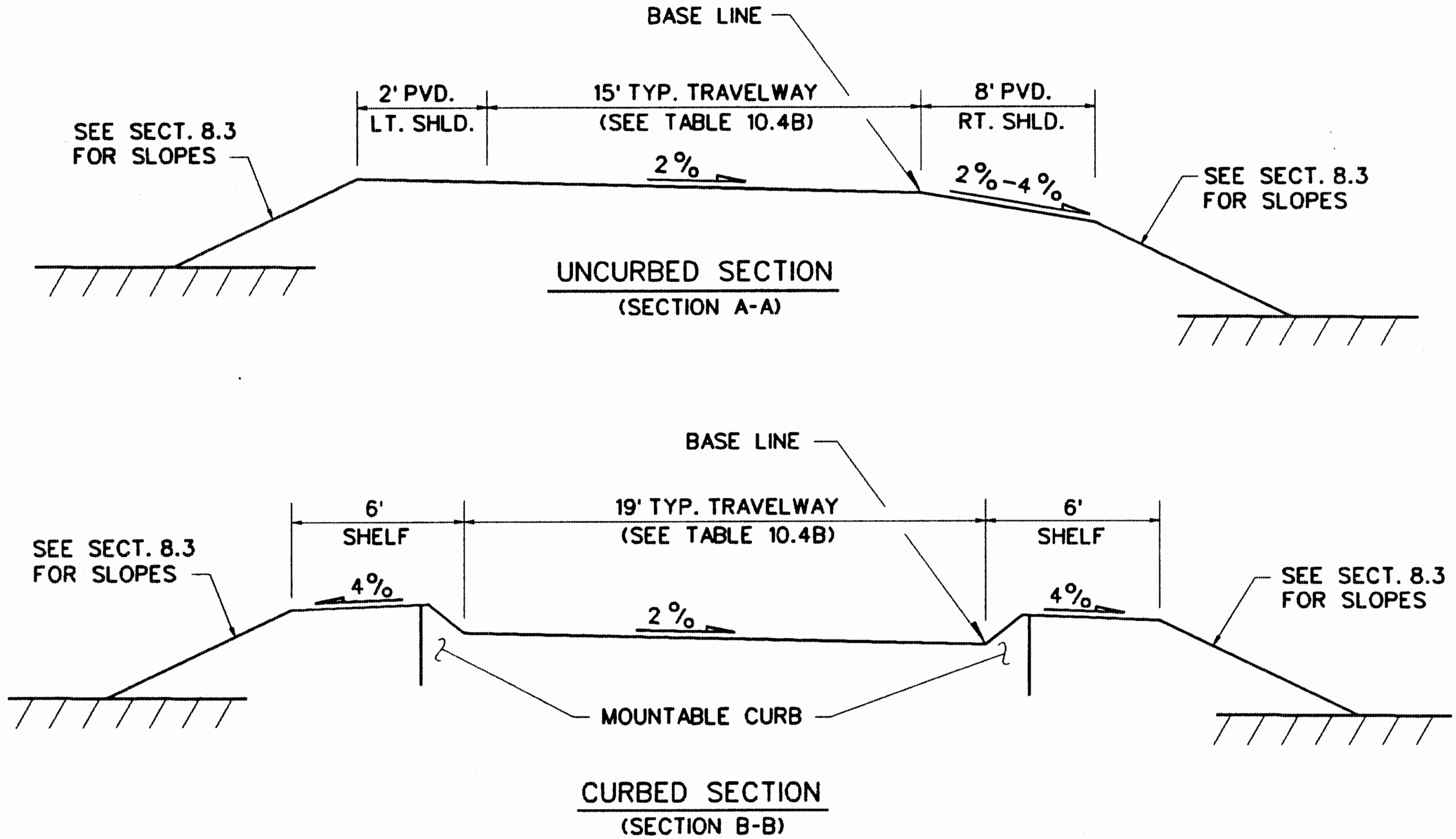
The paved width of a ramp is determined according to the controlling radius and the selected design vehicle. Table 10.4B provides the pavement widths for the various design vehicles. All freeway ramps will typically be designed for a WB-114 design vehicle. In restricted locations, a WB-67 design vehicle may be acceptable. All interchange ramps not connected to a freeway will typically be designed for a WB-67 design vehicle. The SU design vehicle may only be used for the turnaround design at diamond interchanges (see Section 10.5).

The following discusses the application and derivation for the ramp widths.

Uncurbed Ramps

The criteria in Table 10.4B for uncurbed ramps are determined as follows:

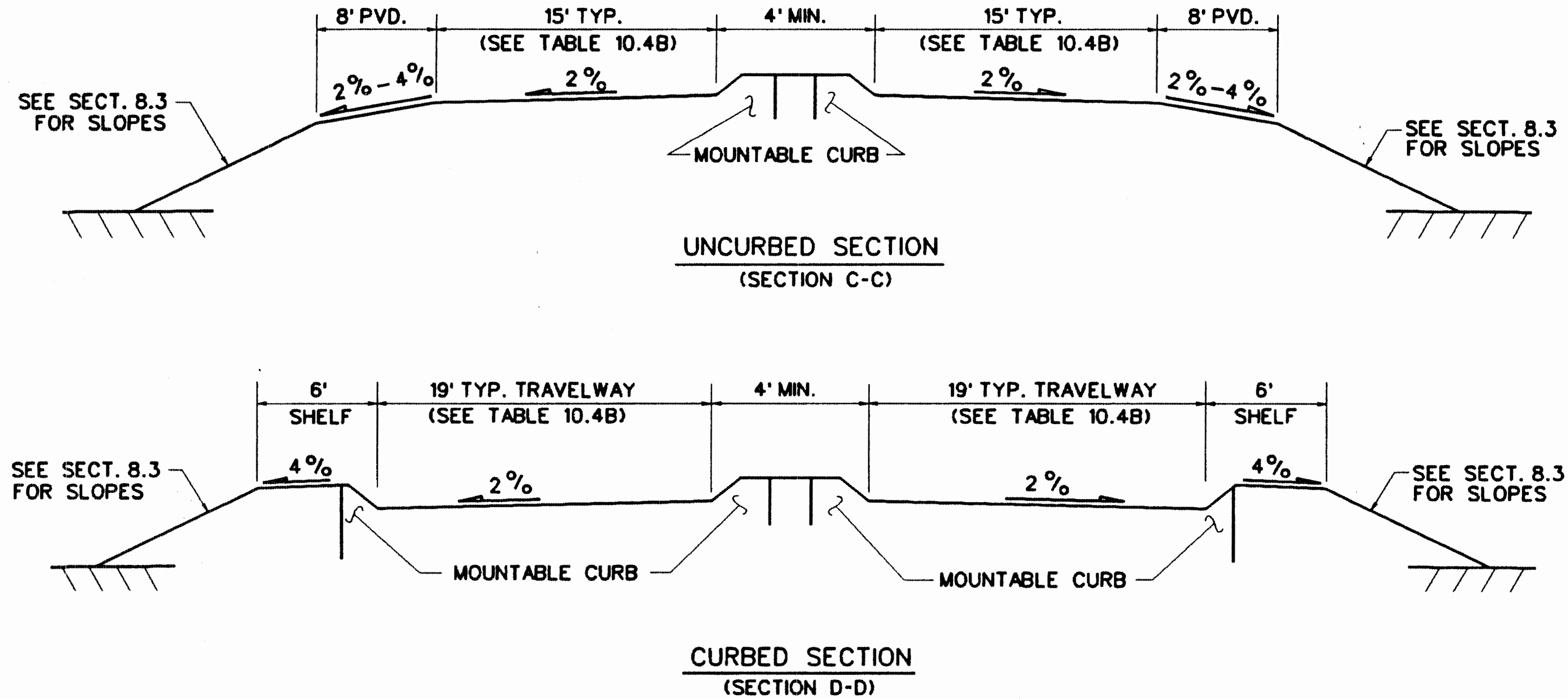
1. For each curve radius, select the width for Case IIB in Table III-21 of Reference (1) (P. 209). Deduct 10 ft total from this value for the left and right shoulders.
2. Compare the value in Step #1 to the Case IB widths in Table III-21 of Reference (1). Select the larger of the two and add 10 ft total for the left and right shoulders.
3. Compare the value in Step #2 to the calculated Case I widths for each design vehicle in Table III-20 of Reference (1) (P. 208). Select the larger of this value or the value in Step #2. This will ensure that the paved ramp width is at least adequate for the selected design vehicle, although there may be no provision for passing the design vehicle should it become disabled on the ramp.



Note: Typical sections are for ramp proper away from either terminal. Section will vary at the at-grade intersection or freeway/ramp junction.

TYPICAL RAMP SECTIONS
(Tangent)

Figure 10.4A



Notes:

1. Typical sections are for ramp proper away from either terminal. Section will vary at the at-grade intersection or freeway/ramp junction.

2. Designer may determine a median barrier is warranted to separate on/off ramps.

TYPICAL RAMP SECTIONS
(Adjacent On/Off Ramps)

Figure 10.4B

Table 10.4B

RAMP WIDTHS
(Feet)

Uncurbed Ramp

Radius on Inner Edge of Traveled Way (ft)	Selected Design Vehicle		
	SU*	WB-67	WB-114
50	28	29	37
75	27	27	33
100	26	26	31
150	26	26	26
200	26	26	26
300	25	25	25
400	25	25	25
500	25	25	25
Tangent	25	25	25

* SU design vehicle is only used for turnarounds. See Section 10.5.

Notes:
1. Widths are for entire paved area; i.e., traveled way plus shoulders.
2. Widths apply to ramp proper; they do not apply to freeway/ramp junctions or at-grade intersections.
3. For two-lane operation, ramp widths are typically 38′ (two 12′ lanes + 10′ right shoulder + 4′ left shoulder).
4. See discussion in Section 10.4.2 for derivation of widths.

Curbed Ramp

Radius on Inner Edge of Traveled Way (ft)	Selected Design Vehicle		
	SU*	WB-67	WB-114
50	25	29	37
75	23	26	33
100	22	26	31
150	21	24	25
200	21	21	23
300	20	20	20
400	20	20	20
500	20	20	20
Tangent	19	19	19

* SU design vehicle is only used for turnarounds. See Section 10.5.

Notes:
1. Widths are for curb-to-curb distance.
2. Widths apply to ramp proper; they do not apply to freeway/ramp junctions or at-grade intersections.
3. For two-lane operation, ramp widths are typically 28′ curb-to-curb.
4. See discussion in Section 10.4.2 for derivation of widths.

Curbed Ramps

The criteria in Table 10.4B for curbed ramps are determined as follows:

1. For each curve radius, select the width for Case IIB in Table III-21 of Reference (1) (P. 209).

2. Compare the value in Step #1 to the calculated Case I widths for each design vehicle in Table III-20 of Reference (1) (P. 208). Select the larger of this value or the value in Step #1. This will ensure that the paved ramp width is at least adequate for the selected design vehicle, although there may be no provision for passing the design vehicle should it become disabled on the ramp.

Bridges

The following will apply to widths of bridges on ramps:

1. Uncurbed Ramp. The bridge width is typically 29 ft regardless of the approaching ramp width.

2. Curbed Ramp. The bridge width is typically 29 ft. The approaching curbed ramp section (typically 19-ft wide) is transitioned to a 25-ft uncurbed section in advance of the bridge. The departure side of the bridge is treated similarly. See Section 8.1 and the ODOT Standard Drawings for more information on curb transitions.

10.4.2.2 Other Elements

The following presents applicable design criteria for ramp cross section elements other than width:

1. Cross Slope. The traveled way cross slopes are typically 2%. Shoulders less than 4-ft wide should be sloped at the same rate and in the same direction as the ramp. Shoulder widths 4 ft or greater typically have a cross slope of 2% to 4% and slope away from the traveled way.

2. Curbs. Curbs on ramps are used only if they are necessary for drainage or where required for right-of-way considerations. On low-speed ramps, curbs also improve delineation for the driver. If curbs are used on a ramp, they will be mountable. See Figure 10.4A.

3. Side Slopes/Ditches. Side slopes and ditches should meet the same criteria as for the mainline. Chapter Eight provides information on the design of these elements.

4. Clear Zones. The clear zone from the edge of the traveled way portion of the ramp will be determined from Table 11.2A. The design ADT will be the directional ADT on one-way ramps.

5. Barriers. Whenever practical, an additional 2 ft should be added to the shoulder when a roadside barrier is used. Where a barrier is present on a horizontal curve, the designer should determine the barrier impact on horizontal sight distance. See Chapter Six.

6. Right-of-Way. The right-of-way adjacent to the ramp should be the same as that determined for the freeway mainline in the vicinity of the interchange.

10.4.3 Horizontal Alignment

10.4.3.1 Theoretical Basis

Establishing horizontal alignment criteria requires a determination of the theoretical basis for the various alignment factors. These include the side-friction factor (f), the distribution method between side friction and superelevation, and the distribution of the superelevation transition length between the tangent and horizontal curve. For horizontal alignment on the ramp proper, the theoretical basis will be one of the following:

1. Open-Road Conditions. Chapter Six presents the theoretical basis for horizontal alignment assuming open-road conditions. In summary, this includes:

   a. relatively low side-friction factors (i.e., a relatively small level of driver discomfort);

   b. the use of AASHTO Method 5 to distribute side friction and superelevation;

   c. relatively flat longitudinal gradients for superelevation transition lengths; and

   d. distributing 75% of the superelevation transition length to the tangent and 25% to the horizontal curve.

   The designer should review Chapter Six to gain an understanding of horizontal alignment theory for open-road conditions.

2. Turning Roadway Conditions. Chapter III of the *AASHTO A Policy on Geometric Design of Highways and Streets* (Reference 1) presents the theoretical basis for horizontal alignment assuming turning roadway conditions. In summary, this includes:

   a. higher side-friction factors to reflect a higher level of driver acceptance of discomfort;

   b. a range of acceptable superelevation rates for combinations of curve radius and design speeds to reflect the need for flexibility to meet field conditions for turning roadway and ramp design; and

   c. the allowance of some flexibility in superelevation transition lengths and in the distribution between the tangent and curve.

   The designer should review the *AASHTO Policy* to gain an understanding of horizontal alignment theory for turning roadway conditions.

The selection of which theoretical basis to use will be based on the portion of the ramp under design. These are:

1. freeway/ramp junction,
2. ramp proper (direct connection ramps),
3. ramp proper (loop ramps),
4. ramp terminus (intersection control), and
5. ramp terminus (merge control).

In addition, several general controls will dictate horizontal alignment on interchange ramps. All of these are discussed in the following sections.

10.4.3.2 General Controls

The following will apply to the horizontal alignment of all ramp elements:

1. $\underline{e_{max}}$. The typical maximum superelevation rate is 0.06. An e_{max} = 0.08

may be used to meet restrictive field conditions, which is often necessary for loop ramps.

2. Axis of Rotation. This will typically be about the right edge of the travelway. This is normally the profile base line.

3. Shoulder Superelevation. The criteria presented in Chapter Six for superelevating the high side and low side of shoulders will apply to superelevated curves on ramps.

4. Minimum Length of Design Superelevation. The designer should not superelevate curves on ramps such that the design superelevation rate is maintained on the curve for a very short distance. As a general rule, the minimum distance should be about 100 ft.

5. Sight Distance. Chapter Six presents the criteria for sight distance around horizontal curves based on the degree of curve and design speed. These criteria apply to curves on ramps.

6. Compound Curves. For compound curves on the ramp proper, the minimum length of the entering flatter curve should allow for safe deceleration to the design speed of the sharper curve. Table 10.3A can be used to determine the minimum distance between the PC and PCC. For example, if the design speed at the PC is 40 mph and the design speed at the PCC is 30 mph, Table 10.3A yields a deceleration distance of 185 ft. Therefore, the minimum distance between the PC and PCC should be 185 ft. See Chapter Six for more information on compound curves.

10.4.3.3 Freeway/Ramp Junctions

Horizontal alignment at freeway/ramp junctions is based on open-road conditions. This is further discussed in Section 10.3.

10.4.3.4 Ramp Proper (Direct Connection Ramps)

Direct connection ramps refer to those ramps which are relatively direct in their alignment. These include ramps at diamond interchanges, the outer ramps at cloverleaf interchanges and ramps at directional and semi-directional interchanges.

The ramp proper, for the purpose of horizontal alignment, is considered to be approximately 100 ft - 200 ft beyond or before the gore nose. The following will determine the application of horizontal alignment to horizontal curves on the ramp proper:

1. Radius ≥ 500′. Use open-roadway conditions.

2. Radius < 500′. Desirably, use open-roadway conditions. It is acceptable to use turning roadway conditions.

10.4.3.5 Ramp Proper (Loop Ramps)

Loop ramps are those ramps on the interior portions of cloverleaf and partial cloverleaf interchanges. The ramp proper is considered to be approximately 100 ft - 200 ft beyond or before the gore nose. Because of the normally restrictive conditions for loop ramps, the curve radii are typically less than 500 ft. The horizontal alignment is typically designed assuming turning roadway conditions.

10.4.3.6 Ramp Terminus (Intersection Control)

Interchange ramps may end at at-grade intersections. These may be stop control or signal control. If horizontal curves on the ramps are relatively close to the intersection, a design speed for the curve should be selected which is appropriate for expected operations at the curve. The following will apply to horizontal curves at the ramp terminus:

1. Radius ≥ 500′. Use open-roadway conditions.

2. Radius < 500′. Desirably, use open-roadway conditions. It is acceptable to use turning roadway conditions.

10.4.3.7 Ramp Terminus (Merge Control)

Interchange ramps may terminate with a merge into the intersecting road. The horizontal alignment at the ramp merge will generally be based on open-road conditions.

10.4.4 Vertical Alignment

10.4.4.1 Grades

Maximum grades for vertical alignment cannot be as definitively expressed as those for the highway mainline. General values of limiting gradient are shown in Table 10.4C but, for any one ramp, the selected gradient is dependent upon a number of factors. These include:

1. The flatter the gradient on the ramp, the longer it will be. At restricted sites (e.g., loops), it may be necessary to provide a steeper grade to shorten the length of ramp.

2. The steepest gradients should be designed for the center portion of the ramp. Freeway/ramp junctions and landing areas at at-grade intersections should be as flat as practical.

3. Short upgrades of as much as 5% do not unduly interfere with truck and bus operations. Consequently, for new construction it is desirable to limit the maximum gradient to 5%.

4. Downgrades on ramps should follow the same guidelines as upgrades. They may, however, safely exceed these values by 2% with 8% considered a recommended maximum.

5. For approximately 200 ft within the gore nose of the freeway/ramp junction, the gradients on the ramp should be compatible with those on the mainline.

6. Practical ramp gradients and lengths can be significantly impacted by the angle of intersection between and gradients on the two highways.

Table 10.4C

RAMP GRADIENT GUIDELINES

Ramp Design Speed (mph)	20	30	40	50
Maximum Grade Range (%)	6-8	5-7	4-6	3-5

Notes:

1. *Downgrades may exceed the table values by 2% but should not exceed 8%.*

2. *The 6% - 8% grades and 20 mph design speed should only be used in extreme conditions and where the restrictive geometric elements are clearly visible to the driver.*

10.4.4.2 Vertical Curvature

Vertical curves on ramps should be designed the same as those on the mainline. They should desirably be designed to meet the stopping sight distance criteria as the minimum design. The ramp profile often assumes the shape of the letter S with a sag vertical curve at one end and with a crest vertical curve at the other. In addition, the vertical curvature of the ramp should be compatible with that of the mainline for approximately 200 ft before or beyond the gore nose. Where a crest or sag vertical curve extends onto the freeway/ramp junction, the following will apply:

1. Loop Ramps. The length of the vertical curve may be determined by using a design speed intermediate between that on the loop and the highway mainline.

2. Other Ramps. For other than loop ramps, vertical curve lengths should desirably be based on a minimum 50-mph design speed.

See Chapter Seven for additional details on the design of vertical curves.

10.4.5 Roadside Safety

The criteria in Chapter Eleven (e.g., clear zones, barrier warrants) will apply to the roadside safety design of interchange ramps. One special situation is the warrant for a median barrier between adjacent on/off ramps, which will be determined on a case-by-case basis. This situation typically occurs at full or partial cloverleaf interchanges.

10.5 OTHER INTERCHANGE DESIGN CONSIDERATIONS

10.5.1 Capacity

The operational goal of the interchange design is to allow the interchange to operate at an acceptable level of service. In addition, the designer should ensure that the operation of the ramp/minor road intersection in urban areas will not impair the operation of the mainline. This may involve a consideration of the operational characteristics of the minor road for some distance in either direction from the interchange.

The *Highway Capacity Manual* provides the required information on how to calculate the level of service for the freeway mainline, the freeway/ramp junction, the ramp and any weaving movements. In addition, a publication *Procedure for Analysis and Design of Weaving Sections; A Users Guide* (Reference (5)) provides additional information and sample problems for designing weaving sections.

10.5.2 Collector-Distributor Roads

In general, interchanges that are designed with single exits are superior to those with two exits, especially if one of the exits is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single-exit design may improve the operational efficiency of the entire interchange.

Collector-distributor (C-D) roads use the single exit approach to improve the interchange operational characteristics. C-D roads will:

1. remove weaving maneuvers from the mainline and transfer them to the slower speed C-D roads,

2. provide high-speed single exits and entrances from and onto the mainline,

3. satisfy driver expectancy by placing the exit in advance of the separation structure,

4. simplify signing and the driver decision-making process, and

5. provide uniformity of exit patterns.

C-D roads are most often warranted when traffic volumes are so high that the interchange without them cannot operate at an acceptable level of service, especially in weaving sections. They are particularly advantageous at full cloverleaf interchanges where the weaving between the ramp/mainline traffic can be very difficult. Figure 10.2F illustrates a schematic of a C-D within a full cloverleaf interchange.

C-D roads may be one or two lanes, depending upon the traffic volumes and weaving conditions. Lane balance should be maintained at the exit and entrance points of the C-D road. Desirably, the design speed of the C-D road should be within 10 mph of the freeway mainline but, at a minimum, should not be less than 40 mph. The separation between the C-D road and mainline should be as wide as practical but not less than that required to provide the applicable shoulder widths and a longitudinal barrier between the two.

10.5.3 Frontage Roads

The designer must consider the impact of frontage roads, where present, on interchange design. At most interchanges, it is impractical to separate the intersections of the ramp and frontage road with the crossing road. In these cases, the only alternative is to combine the ramp and frontage road before the intersection with the crossing road. This can apply to either the exit or entrance ramp.

Figure 10.5A provides the basic schematic for this design. The critical design element is the distance "A" between the ramp/frontage road merge and the crossing road. This distance must be sufficient to allow traffic weaving, vehicular deceleration and stopping, and vehicular storage to avoid interference with the merge point. Table 10.5A presents general guidelines which may be used to estimate this distance during the preliminary design phase. A number of assumptions have been made including weaving volume, operating speeds and intersection queue distance. Therefore, a detailed analysis will be necessary to firmly establish the needed distance to properly accommodate vehicular operation.

The following summarizes the available options for coordinating the design of the interchange ramps, frontage road and crossing road:

1. Slip Ramps. Slip ramps are typically used to connect the freeway with one-way frontage roads before (or after) the intersection with the crossing road. Newly constructed slip ramps to a two-way frontage road are unacceptable because they may induce wrong-way entry onto the freeway and may cause accidents at the intersection of the ramp and frontage road.

2. Buttonhook Ramps. For two-way frontage roads, buttonhook ramps are typically used to connect the freeway with the frontage road before (or after) the intersection with the crossing road.

3. Separate Intersections. As an option to providing an intersection between the freeway ramp and frontage road, it may be practical to provide separate ramp/crossing road and frontage road/crossing road intersections. This may be accomplished by curving the frontage road away from the ramp and intersecting the frontage road with the crossing road outside the ramp limits of no access. This treatment allows the two intersections to operate independently, and it eliminates the operational and signing problems of providing the same point of exit and entrance for the frontage road and freeway ramp.

Section 10.6 presents typical interchange design figures (e.g., for diamond and cloverleaf interchanges). The Section illustrates interchanges with both one-way and two-way frontage roads. Section 10.5.7 presents typical figures for at-grade intersections for ramp termini. These include figures for ramp/frontage road intersections (e.g., buttonhook ramps).

Chapter Eight discusses overall design criteria for frontage roads (e.g., functional class, outer separation).

10.5.4 Freeway Lane Drops

Freeway lane drops, where the basic number of lanes is decreased, must be carefully designed. They should normally occur on the freeway mainline away from any other turbulence such as interchange exits and entrances. However, it may be advantageous to drop a basic freeway lane at a two-lane exit.

Figure 10.5B illustrates the recommended design of a lane drop beyond an interchange. The following criteria are important when designing a freeway lane drop:

Table 10.5A

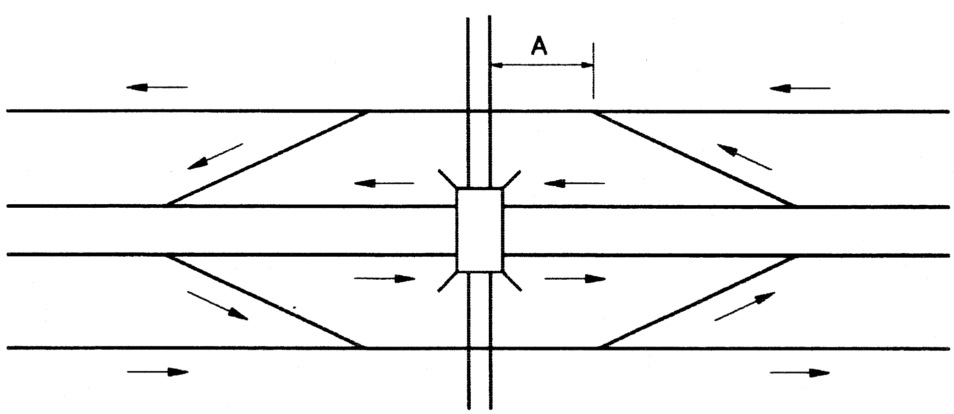
DISTANCE "A" FROM EXIT RAMP/FRONTAGE ROAD TO INTERSECTION WITH MINOR ROAD

Frontage Road Volume (vph)[1]	Exit Ramp Volume (vph)[2]	"A" (ft)		
		Typical Minimum	Typical Desirable	Special Conditions
200	140	380	500	260
400	275	460	560	360
600	410	500	630	400
800	550	540	690	430
1000	690	590	760	450
1200	830	640	870	480
1400	960	690	970	500
1600	1100	770	1070	530
1800	1240	860	1180	550
2000	1380	970	1300	580

1 Total frontage road and exit ramp volume between merge to intersection with minor road.

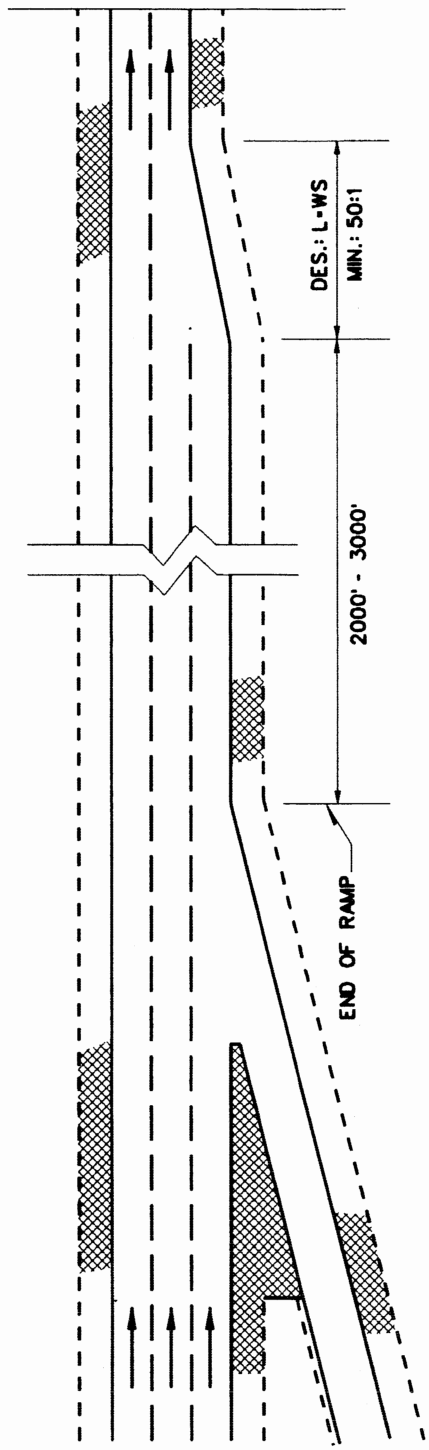
2 Assumed to be 69% of total volume in first column.

Note: Table values are acceptable for planning purposes; final dimensions will be based on a detailed operational analysis.



RAMP/FRONTAGE ROAD INTERSECTION

Figure 10.5A



FREEWAY LANE DROP
(Typical Schematic)

Figure 10.5B

1. Location. In urban areas, desirably the lane drop should occur approximately 2,000 - 3,000 ft beyond the previous interchange. Under restricted conditions 1,000 ft is acceptable; the MUTCD signing distance is the absolute minimum. For rural areas the minimum distance is a ½ mile. This distance allows adequate signing and driver adjustments from the interchange, but yet is not so far downstream that drivers become accustomed to the number of lanes and are surprised by the lane drop. In addition, a lane should not be dropped on a horizontal curve or where other signing is required, such as for an upcoming exit.

   In urban areas, interchanges may be closely spaced for considerable lengths of highway. In these cases, it may be necessary to drop a freeway lane at an exit. Where this is necessary, it is preferable to drop a freeway lane at a two-lane exit rather than a single-lane exit.

   Figure 10.3E illustrates the recommended design. One key design feature is the "escape taper" provided just beyond the exit gore. Some drivers may miss the signs which notify them that the mainline lane is being dropped at the exit. The escape taper provides these drivers with an opportunity to merge left into the remaining through lanes. As discussed in Section 10.1 on the basic number of lanes, a lane should not be dropped at an exit unless there is a large decrease in traffic demand for a significant length of freeway.

2. Transition. Desirably, the transition taper length will be L = WS, where W is the lane width and S is the design speed. The minimum taper rate that can be used is 50:1 (see Figure 10.5B).

3. Sight Distance. Decision sight distance (DSD) should be available to any point within the entire lane transition. See Chapter Five for applicable DSD values. When determining the availability of DSD, the desirable height of object will be 0.0 ft (the roadway surface); it is acceptable to use 6 inches. This criteria would favor, for example, placing a freeway lane drop within a sag vertical curve rather than just beyond a crest.

4. Right-Side versus Left-Side Drop. Right-side freeway lane drops are preferred; however, a left-side lane reduction may be warranted at specific locations.

5. Shoulders. The full-width right shoulder will be maintained through a right-side lane drop. If a left-side lane drop will be used to reduce the number of lanes from three to two, the left shoulder will be reduced from 10 ft (or 12 ft) to 4 ft. The full 10-ft left shoulder should be maintained for a distance of approximately 200 ft beyond the merge point of the dropped lane. This provides an area to allow a driver, who may have missed the signing, an opportunity to safely merge with the through traffic.

10.5.5 Access Control

Proper access control must be provided along the crossing road in the vicinity of the ramp/crossing road intersection or along a frontage road where present. This will ensure that the intersection has approximately the same degree of freedom and absence of conflict as the freeway itself. The access control criteria should be consistent with these goals.

Section 10.6 presents several typical interchange design figures. These provide ODOT policy for the location of the limits of no access lines along the ramp, at ramp/crossing road intersections, across from the ramp terminal and along frontage roads.

As indicated in the figures, the limits of no access extend a distance along the crossing road away from the ramp or frontage road intersection. The 100 ft in urban areas and 300 ft in rural areas should usually satisfy any congestion concerns. However, in areas where the potential for development exists which would create traffic problems, it may be appropriate to consider longer lengths of access control. In addition, many areas have changed over the years from rural to urban. As indicated, ODOT has adopted different criteria for the access control at urban and rural interchanges. However, a change in area character alone is not a sufficient justification to alter the location of the limit-of-no-access line when an existing interchange will be rehabilitated or when ODOT receives requests for additional access points from outside interests.

The figures in Section 10.6 note that, on the crossing road, the limit of no access should extend the indicated distance beyond "the ramp terminal." For an exit ramp, this is defined as the tangent point (PT) of a radius return on the crossing road or the end of a taper for an entrance onto the crossing road (e.g., for an acceleration lane); i.e., the ramp terminal ends where the typical section of the crossing road resumes. A similar definition applies to ramp terminals for entrance ramps.

10.5.6 Turnarounds

10.5.6.1 General

Turnarounds are an interchange design feature which may be warranted at urban diamond interchanges with one-way frontage roads. Figure 10.5C illustrates the basic schematics of a turnaround. Depending upon specific site conditions, this arrangement may significantly improve traffic operations at the interchange. The major operational feature of the turnaround is that it provides access for traffic on the freeway to the one-way frontage in the opposite direction without passing through the two at-grade intersections on the crossing road.

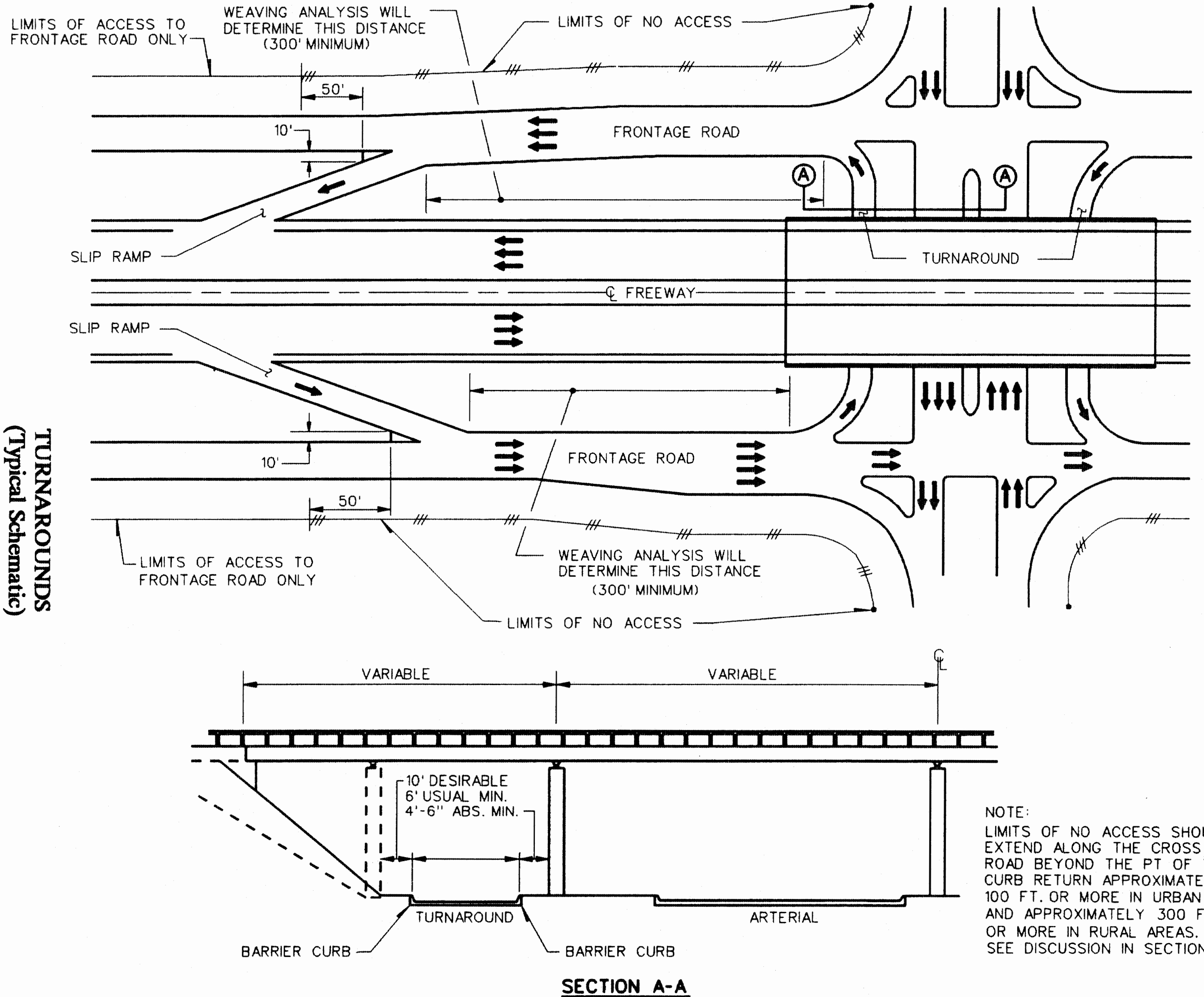
Some advantages and disadvantages of the turnaround are as follows:

Advantages

1. It preserves and enhances the accessibility to property abutting one-way frontage roads.
2. U-turning vehicles do not have to stop at the two at-grade intersections.
3. The capacity of the at-grade intersections is improved.

Disadvantages

1. It is more costly than a diamond due to the wider structure.
2. It can be confusing because it violates driver expectancy (i.e., driving to the left of the oncoming traffic).
3. Because of the left-side on and off maneuvers onto the frontage roads, it cannot be used with two-way frontage roads.
4. Longer distances are needed between the ramp/frontage road intersection and the crossing street.



NOTE:
LIMITS OF NO ACCESS SHOULD EXTEND ALONG THE CROSS ROAD BEYOND THE PT OF THE CURB RETURN APPROXIMATELY 100 FT. OR MORE IN URBAN AREAS AND APPROXIMATELY 300 FT. OR MORE IN RURAL AREAS. SEE DISCUSSION IN SECTION 10.5.

TURNAROUNDS
(Typical Schematic)
Figure 10.5C

10.5.6.2 Warrants

The warrant for the turnaround will be determined on a site-by-site basis. Where a turnaround will not be used at a new urban diamond interchange, the designer should consider arranging the end spans of overpasses, or grade lines of underpasses, so that turnarounds may be added in the future, if warranted.

10.5.6.3 Design Criteria

Design criteria presented for ramps in Section 10.4 apply, in general, to turnarounds. For example, the ramp width criteria in Table 10.4B will apply. The minimum design vehicle for the design of a turnaround is the SU design vehicle. Where the SU is used, proper signing must be provided to notify trucks not to use the turnaround. This must be coordinated with the Traffic Engineering Division.

10.5.7 Ramp/Crossing Road Intersection

At service interchanges, the ramp will often end with an at-grade intersection with either a crossing road or a frontage road. In general, the intersection should be treated as described in Chapter Nine. This will involve a consideration of capacity and the physical geometric design elements such as sight distance, angle of intersection, acceleration lanes, channelization and turning lanes. However, several points warrant special attention in the design of the ramp/crossing road intersection:

1. Capacity. In urban areas where traffic volumes are often high, inadequate capacity of the ramp/crossing road intersection can adversely affect the operation of the ramp/freeway junction. In a worst case situation the safety and operation of the mainline itself may be impaired by a backup onto the freeway. Therefore, special attention should be given to providing sufficient capacity and storage for an at-grade intersection or a merge with the crossing road. This could lead to the addition of lanes at the intersection or on the ramp proper, or it could involve traffic signalization where the ramp traffic will be given priority. The analysis must also consider the operational impacts of the traffic characteristics in either direction on the intersecting road.

   The Project Engineer is responsible for capacity analyses with assistance from the Geometric Design Branch. Any signalization is coordinated through the Traffic Engineering Division.

2. Sight Distance. Chapter Nine discusses the criteria for intersection sight distance. These criteria also apply to a ramp/crossing road intersection. Special attention must be given to the location of the bridge pier, abutment, sidewalk, bridge rail, roadside barrier, etc. These may present major sight distance obstacles. The bridge obstruction and the required intersection sight distance may result in the need to relocate the ramp/crossing road intersection.

3. Wrong-Way Movements. Wrong-way movements may originate at the ramp/ crossing road intersection. The intersection must be properly signed and designed to minimize the potential for a wrong-way movement.

See Section 10.8 for typical design details for ramp/crossing road or ramp/frontage road intersections.

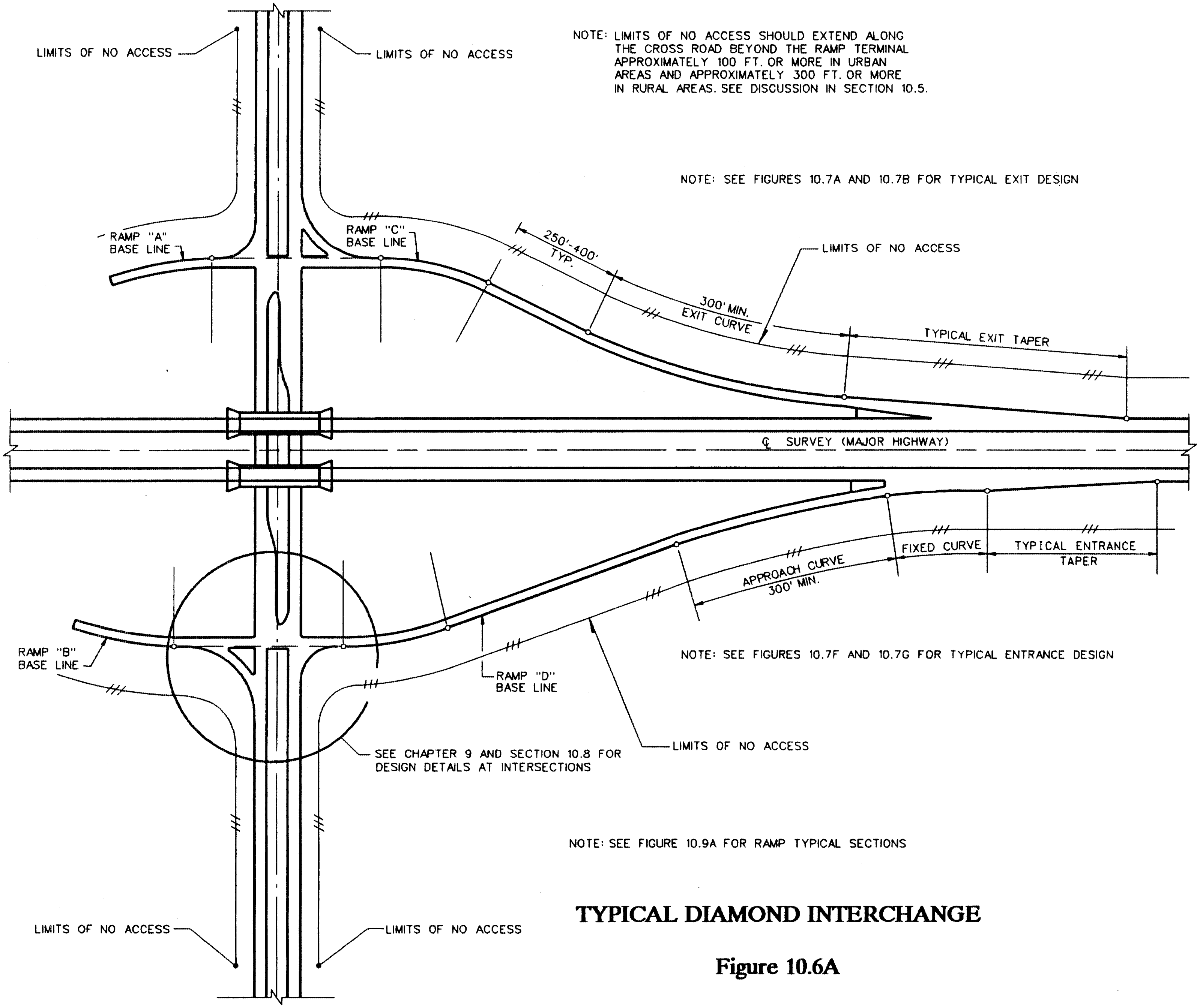
10.6 TYPICAL INTERCHANGE DESIGN FIGURES

To provide the designer with an indication of the basic layout of various types of interchanges, this Section presents typical interchange designs. Table 10.6A summarizes the Figures in Section 10.6.

Table 10.6A

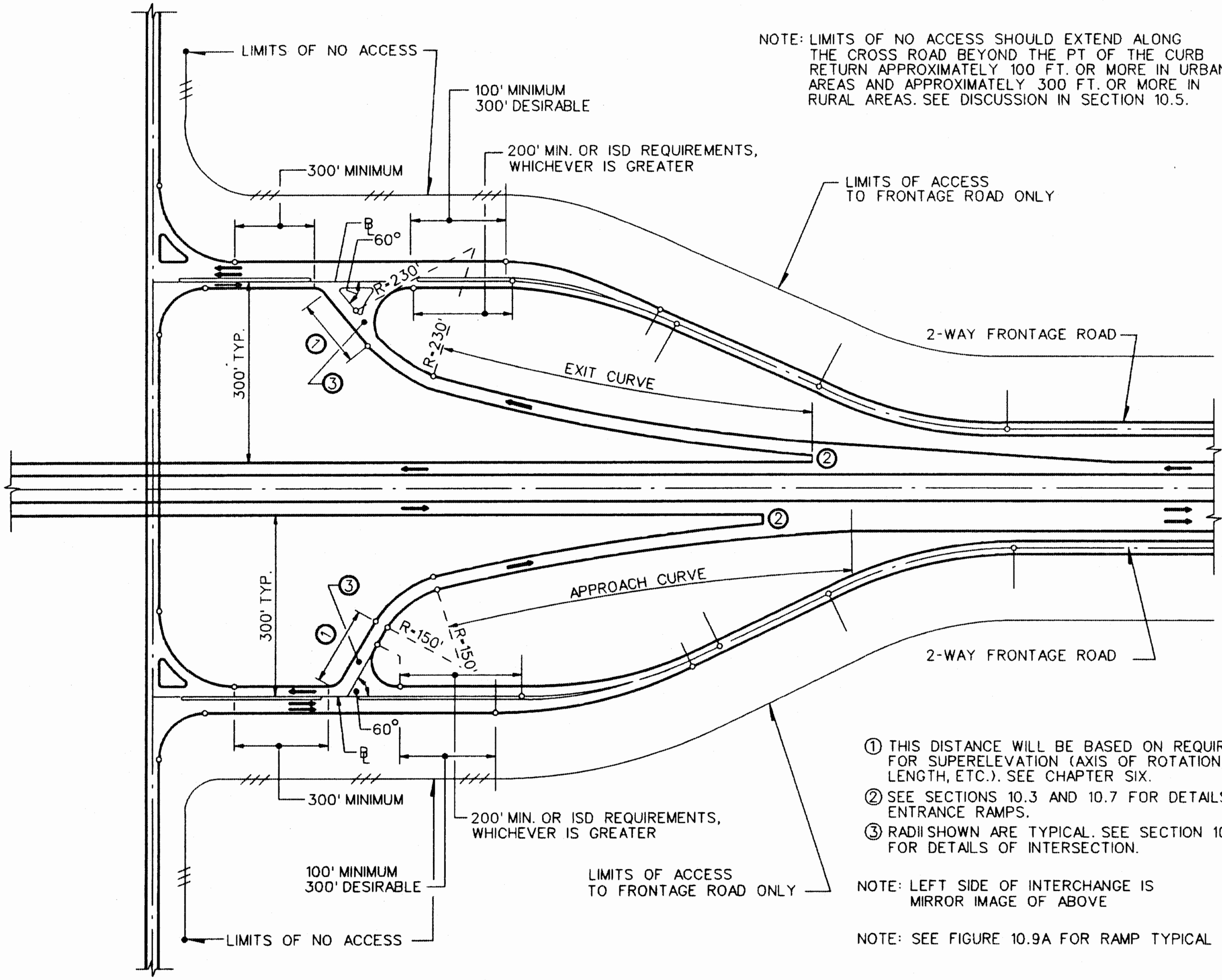
TYPICAL INTERCHANGE DESIGN FIGURES

Figure Number	Figure Title
Figure 10.6A	Typical Diamond Interchange (without Frontage Roads)
Figure 10.6B	Typical Diamond Interchange (with Two-Way Frontage Roads)
Figure 10.6C	Partial Cloverleaf Interchange (Two Quadrants)
Figure 10.6D	Cloverleaf Interchange (without Collector-Distributor Roads)
Figure 10.6E	Loop Design (Uncurbed)
Figure 10.6F	Loop Design (Curbed)



TYPICAL DIAMOND INTERCHANGE

Figure 10.6A



NOTE: LIMITS OF NO ACCESS SHOULD EXTEND ALONG THE CROSS ROAD BEYOND THE PT OF THE CURB RETURN APPROXIMATELY 100 FT. OR MORE IN URBAN AREAS AND APPROXIMATELY 300 FT. OR MORE IN RURAL AREAS. SEE DISCUSSION IN SECTION 10.5.

① THIS DISTANCE WILL BE BASED ON REQUIREMENTS FOR SUPERELEVATION (AXIS OF ROTATION, TRANSITION LENGTH, ETC.). SEE CHAPTER SIX.

② SEE SECTIONS 10.3 AND 10.7 FOR DETAILS OF EXIT AND ENTRANCE RAMPS.

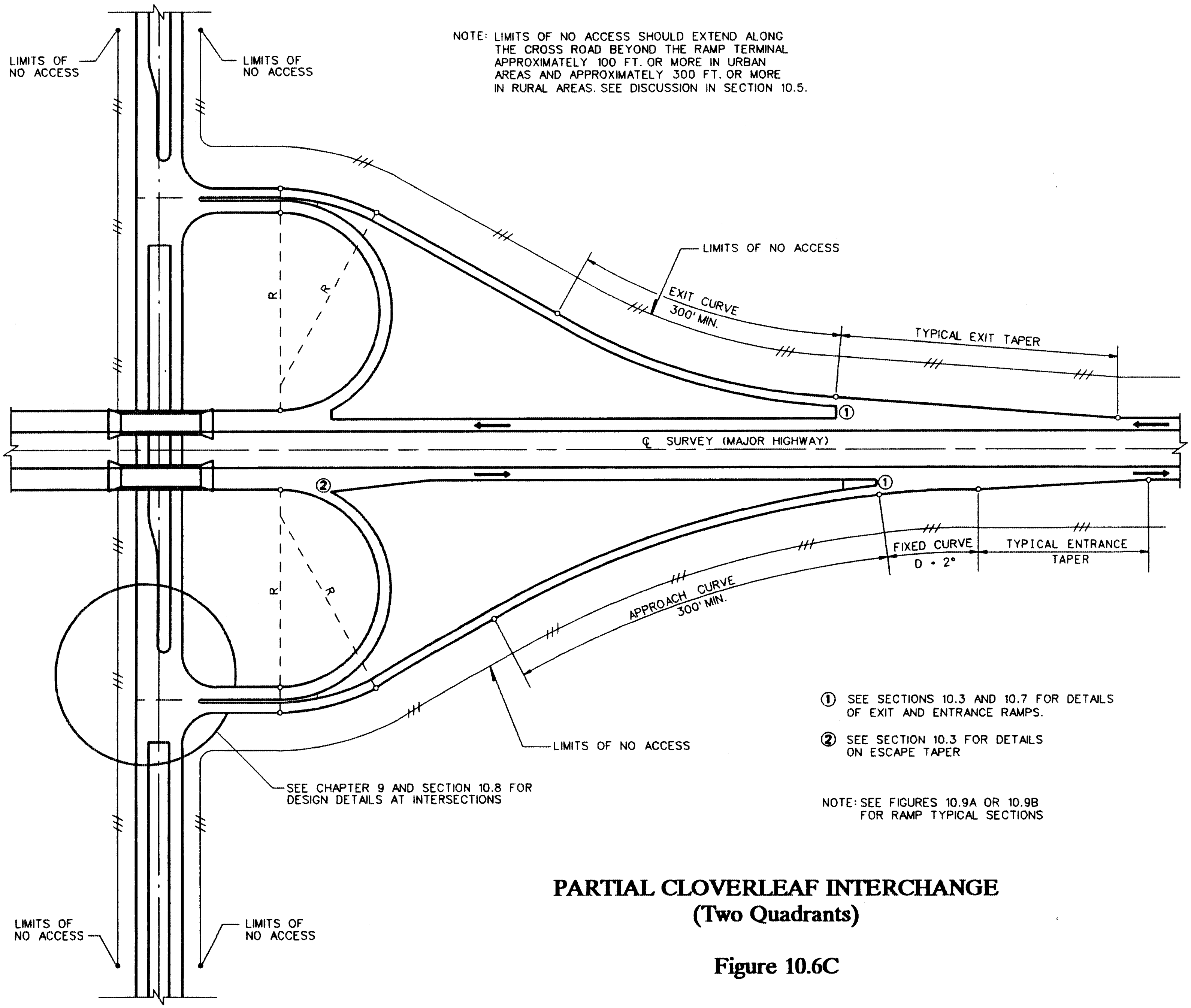
③ RADII SHOWN ARE TYPICAL. SEE SECTION 10.8 FOR DETAILS OF INTERSECTION.

NOTE: LEFT SIDE OF INTERCHANGE IS MIRROR IMAGE OF ABOVE

NOTE: SEE FIGURE 10.9A FOR RAMP TYPICAL SECTIONS

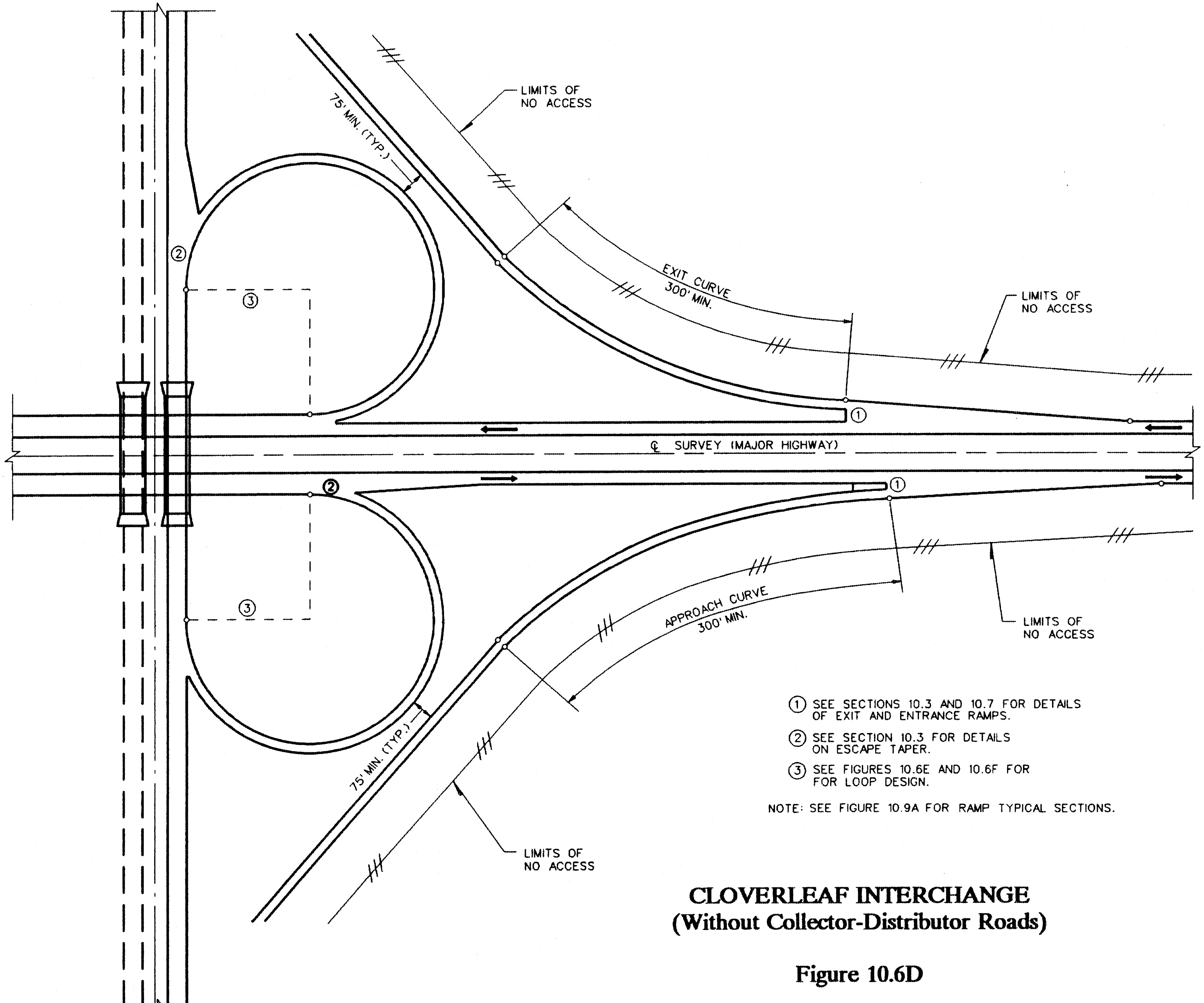
TYPICAL DIAMOND INTERCHANGE
(With Two-Way Frontage Roads)

Figure 10.6B



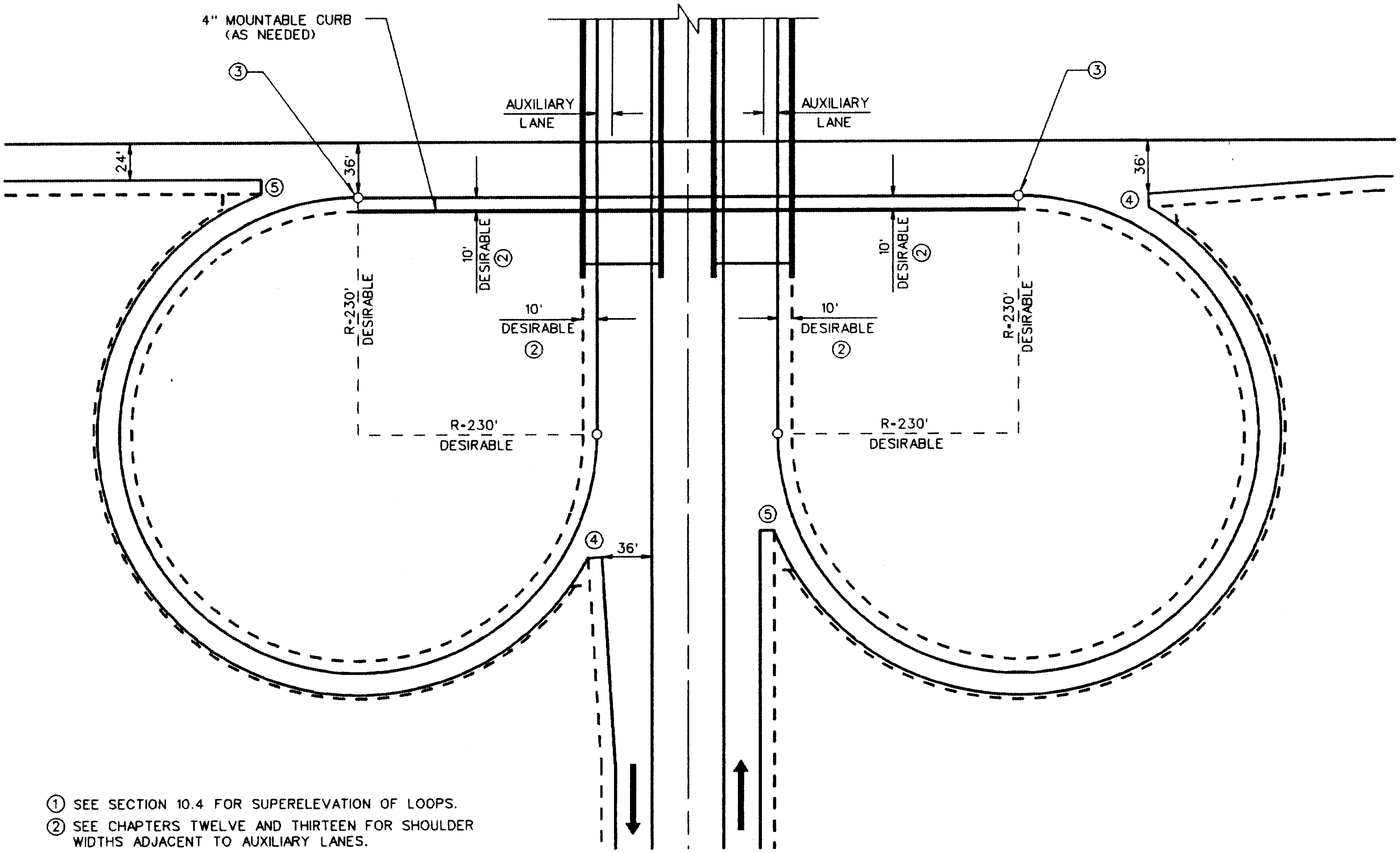
PARTIAL CLOVERLEAF INTERCHANGE
(Two Quadrants)

Figure 10.6C



CLOVERLEAF INTERCHANGE
(Without Collector-Distributor Roads)

Figure 10.6D



(1) SEE SECTION 10.4 FOR SUPERELEVATION OF LOOPS.

(2) SEE CHAPTERS TWELVE AND THIRTEEN FOR SHOULDER WIDTHS ADJACENT TO AUXILIARY LANES.

(3) VARY SHOULDER WIDTH FROM 8' TO 10' AND CURB HEIGHT FROM 0" TO 4"

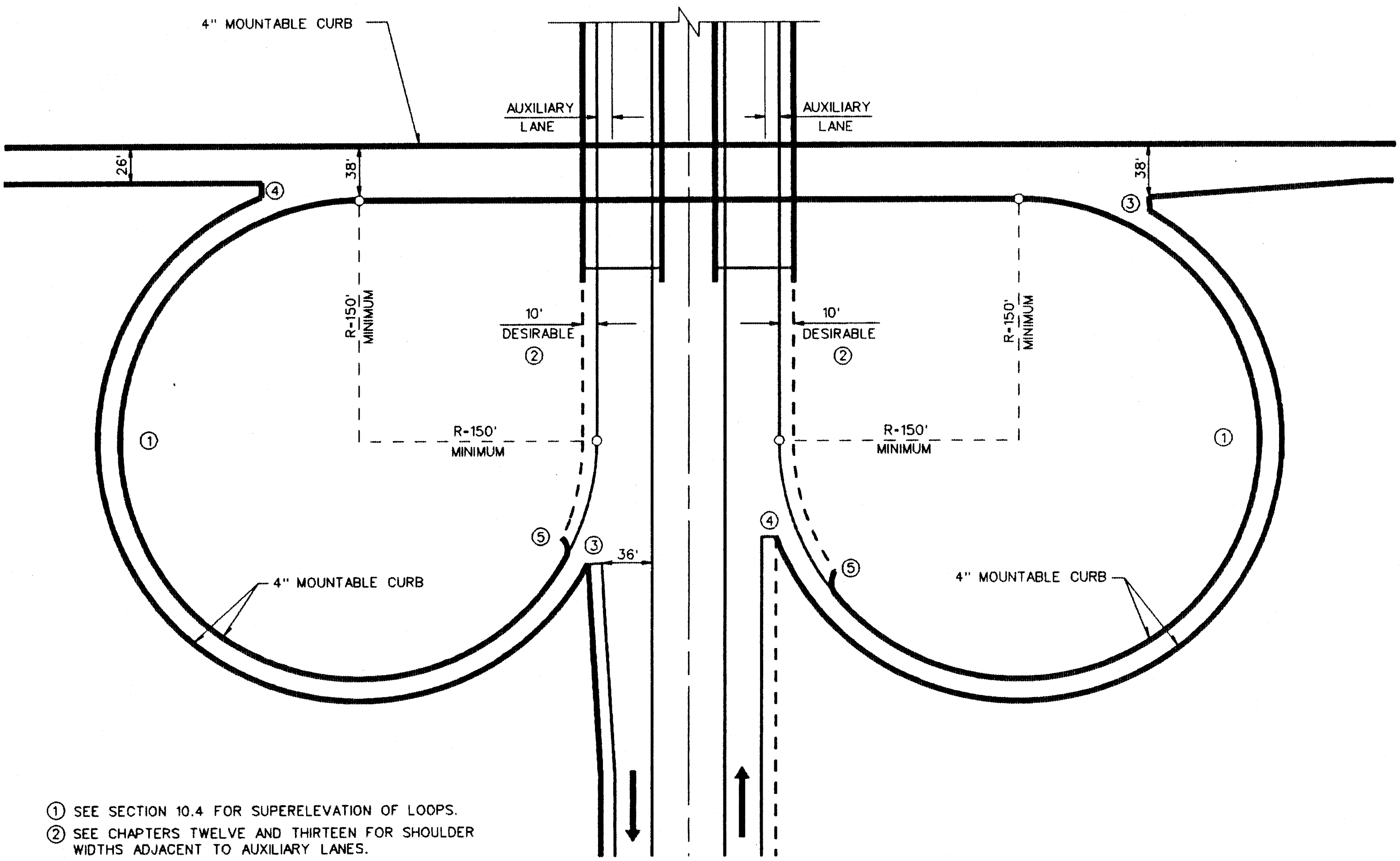
(4) SEE SECTION 10.3 FOR DETAILS ON ESCAPE TAPER.

(5) SEE SECTION 10.3 FOR DETAILS OF ENTRANCE RAMPS.

NOTE: SEE FIGURE 10.9A FOR RAMP TYPICAL SECTIONS.

LOOP DESIGN
(Uncurbed)

Figure 10.6E



① SEE SECTION 10.4 FOR SUPERELEVATION OF LOOPS.
② SEE CHAPTERS TWELVE AND THIRTEEN FOR SHOULDER WIDTHS ADJACENT TO AUXILIARY LANES.
③ SEE SECTION 10.3 FOR DETAILS ON ESCAPE TAPER.
④ SEE SECTION 10.3 FOR DETAILS OF ENTRANCE RAMPS.
⑤ SEE SECTION 8.1 AND ODOT STD. DRAWINGS FOR CURB TRANSITIONS.

NOTE: SEE FIGURE 10.9A FOR RAMP TYPICAL SECTIONS.

LOOP DESIGN
(Curbed)

Figure 10.6F

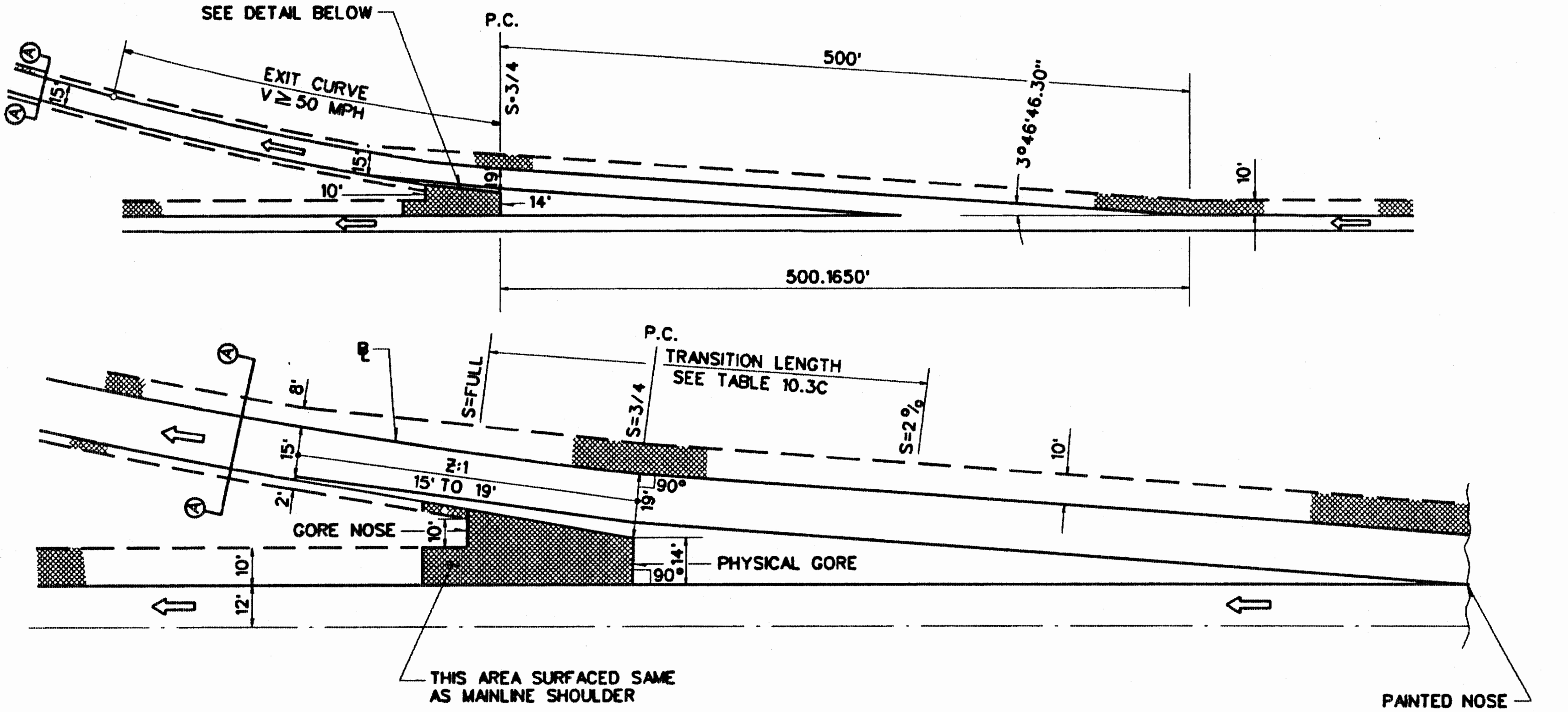
10.7 TYPICAL FREEWAY/RAMP JUNCTION FIGURES

This Section presents design details used by ODOT for freeway/ramp junction designs. Table 10.7A summarizes the figures in Section 10.7.

Table 10.7A

TYPICAL FREEWAY RAMP JUNCTION FIGURES

Figure Number	Figure Title
Figure 10.7A	ODOT Typical Taper Exit Ramp (Uncurbed, V=60 on Mainline)
Figure 10.7B	ODOT Typical Taper Exit Ramp (Uncurbed, V=70 on Mainline)
Figure 10.7C	ODOT Typical Parallel-Lane Exit Ramp (Uncurbed)
Figure 10.7D	ODOT Typical Exit Gore (Curbed)
Figure 10.7E	Escape Taper (Lane Drop at Exit Ramp)
Figure 10.7F	ODOT Typical Taper Entrance Ramp (Uncurbed, V=60 on Mainline)
Figure 10.7G	ODOT Typical Taper Entrance Ramp (Uncurbed, V=70 on Mainline)
Figure 10.7H	ODOT Typical Parallel-Lane Entrance Ramp (Uncurbed)
Figure 10.7I	ODOT Typical Entrance Gore (Curbed)
Figure 10.7J	Exit Terminals on Curvilinear Alignment (D=00°45′ Maximum)
Figure 10.7K	Exit Terminals on Curvilinear Alignment (D=00°45′ Minimum)
Figure 10.7L	Entrance Terminals on Curvilinear Alignment



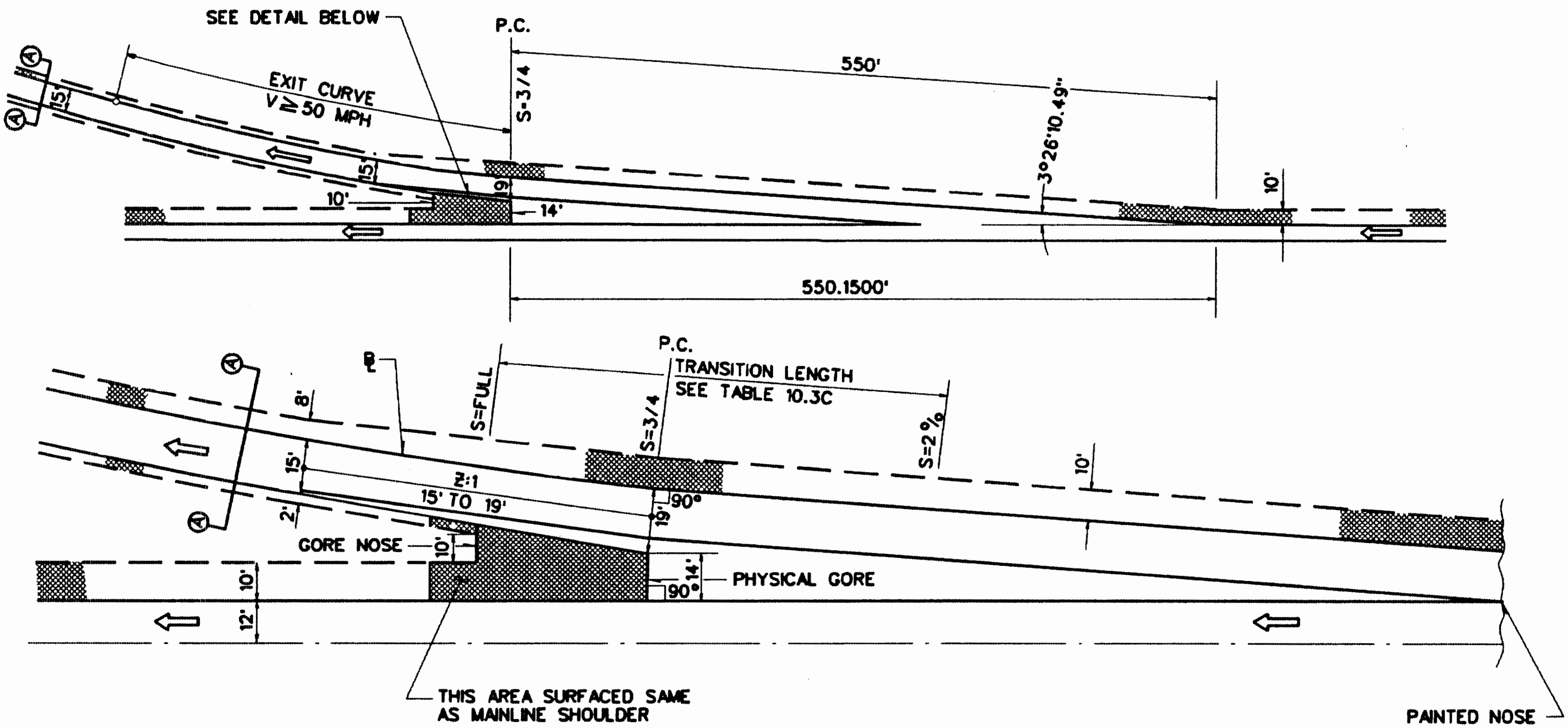
Design Speed of Ramp Curve (mph)	Z — Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where V=60 mph for mainline and V≥50 mph for exit curve. In restricted locations, the exit curve may be designed for V=45 mph.
2. See Table 10.3C for superelevation information.
3. See Figure 10.7D for typical curbed exit gore.
4. See Figure 10.9A for typical ramp section.
5. See Figures 10.7J and 10.7K for exits on curvilinear alignment.
6. A 300′ minimum length of exit curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL TAPER EXIT RAMP
(Uncurbed, V=60 on Mainline)

Figure 10.7A



Notes:

1. Figure applies where V = 70 mph for mainline and V≥50 mph for exit curve.
2. See Table 10.3D for superelevation information.
3. See Figure 10.7D for typical curbed exit gore.
4. See Figure 10.9A for typical ramp section.
5. See Figures 10.7J and 10.7K for exits on curvilinear alignment.
6. A 300′ minimum length of exit curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

Design Speed of Ramp Curve (mph)	Z – Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

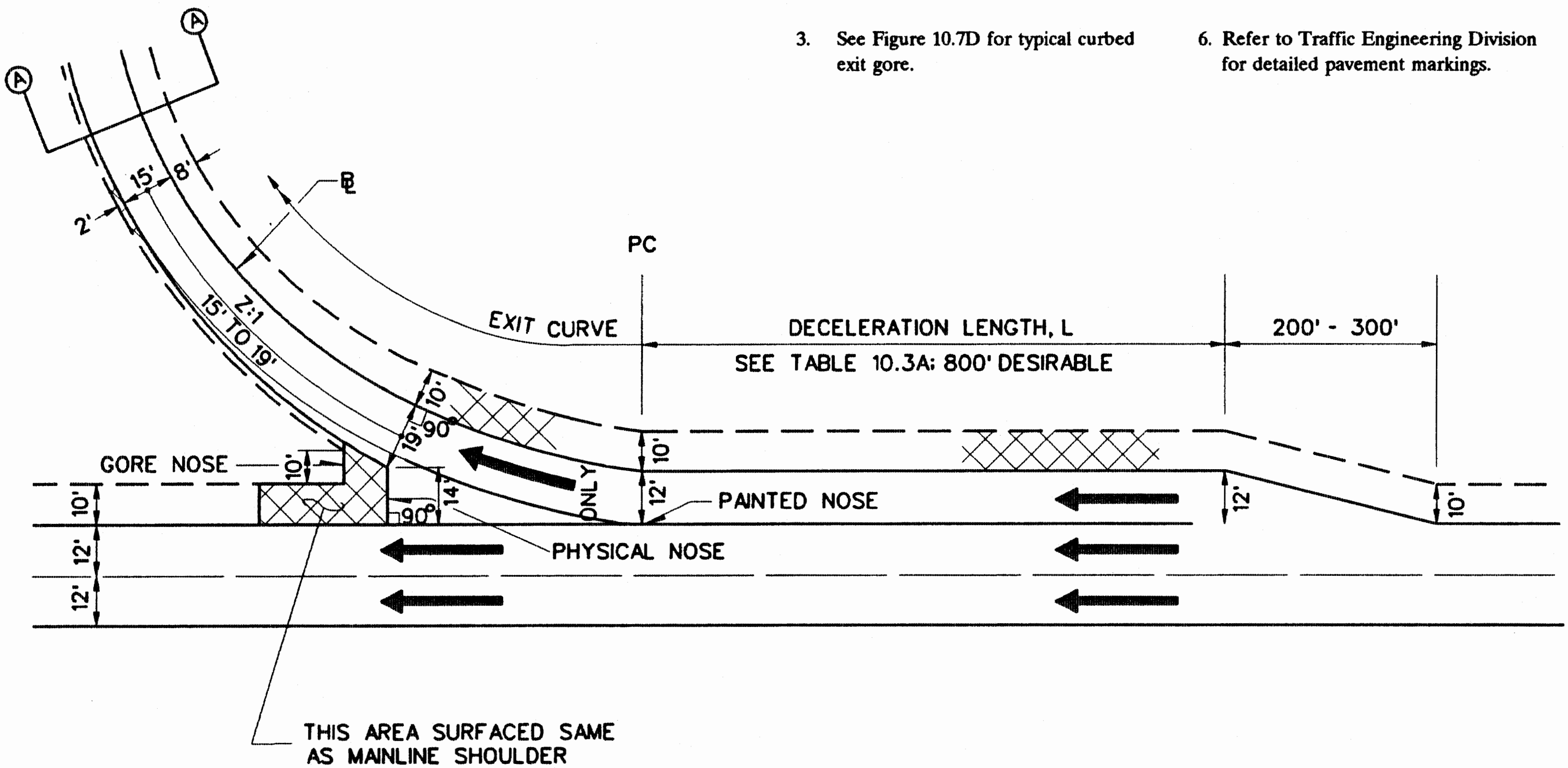
ODOT TYPICAL TAPER EXIT RAMP (Uncurbed, V=70 on Mainline)

Figure 10.7B

Design Speed of Ramp Curve (mph)	Z — Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

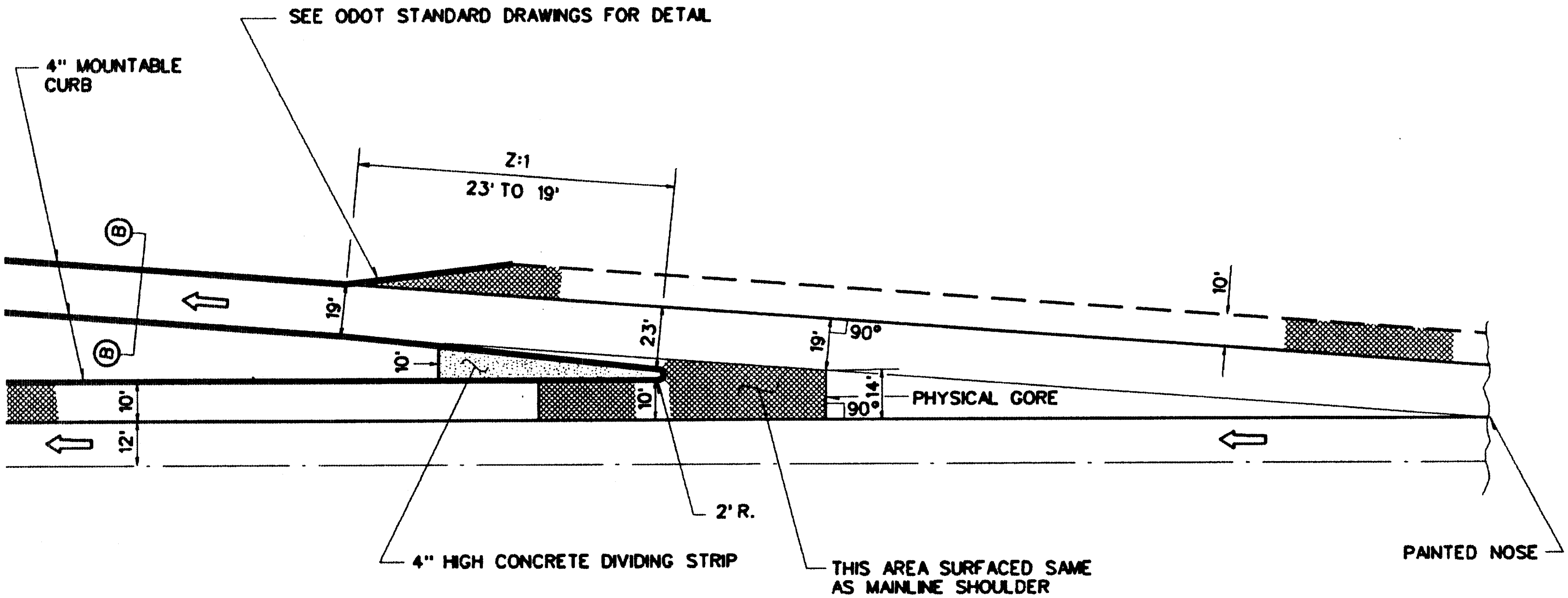
Notes:

1. Figure applies where taper design is inappropriate.
2. See Chapter Six for superelevation information.
3. See Figure 10.7D for typical curbed exit gore.
4. See Figure 10.9A for typical ramp section.
5. A 300′ minimum length of exit curve is desirable.
6. Refer to Traffic Engineering Division for detailed pavement markings.



ODOT TYPICAL PARALLEL-LANE EXIT RAMP
(Uncurbed)

Figure 10.7C



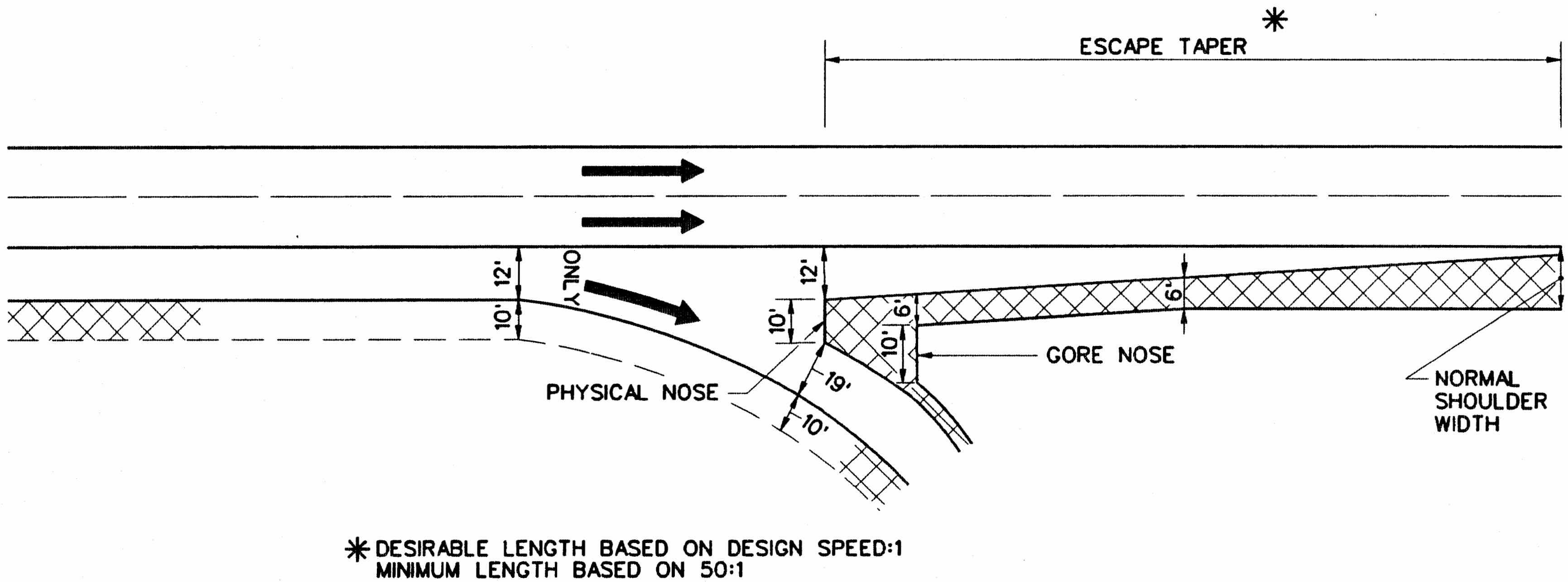
Design Speed of Ramp Curve (mph)	Z — Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where curb is warranted on ramp. See Section 10.4.
2. See Figure 10.9A for typical ramp section.
3. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL EXIT GORE
(Curbed)

Figure 10.7D



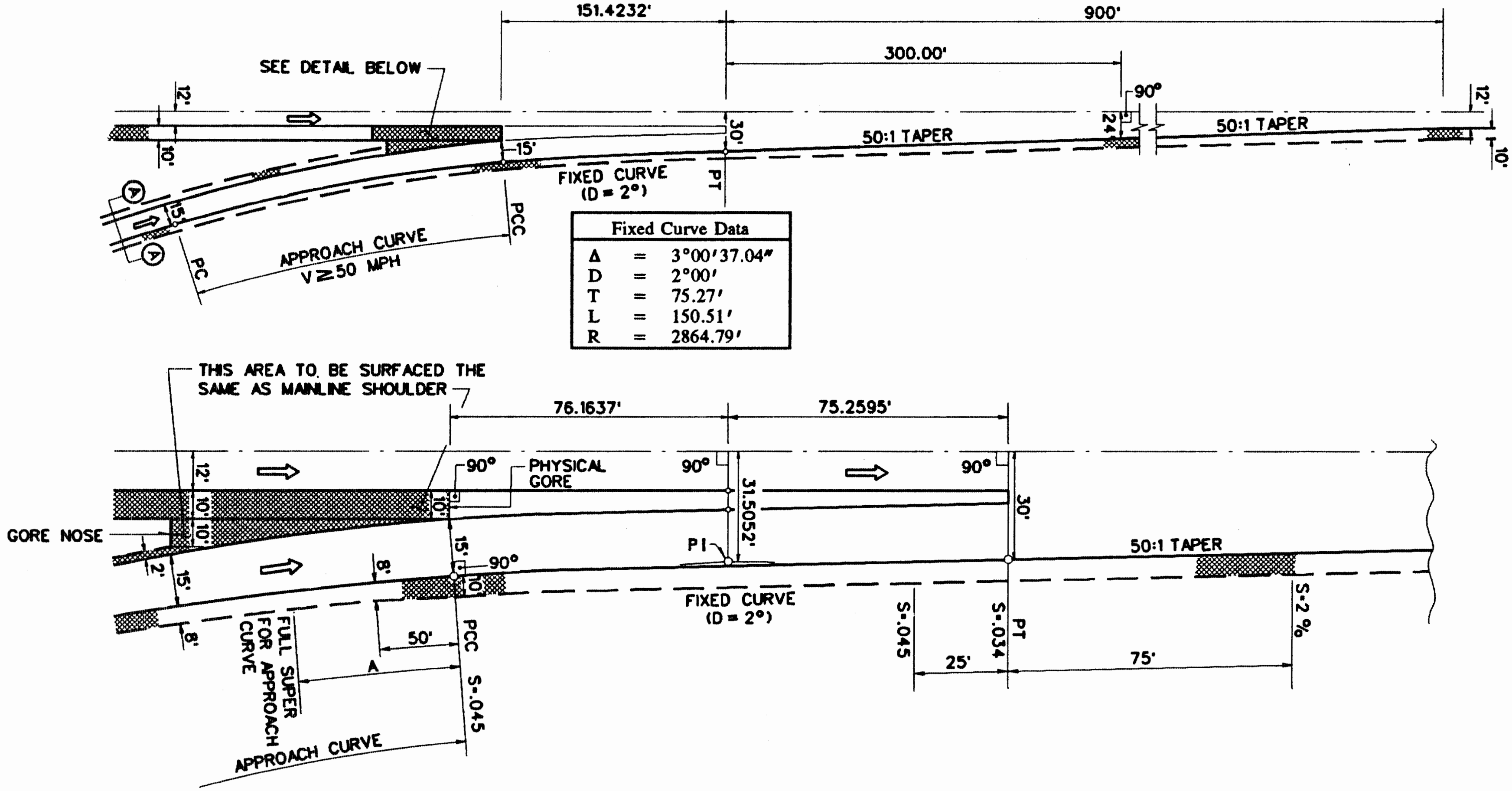
Notes:

1. Figure applies to an uncurbed gore section. See Figure 10.7D for details on a curbed gore.

2. Refer to Traffic Engineering Division for details on pavement markings.

ESCAPE TAPER
(Lane Drop at Exit Ramp)

Figure 10.7E



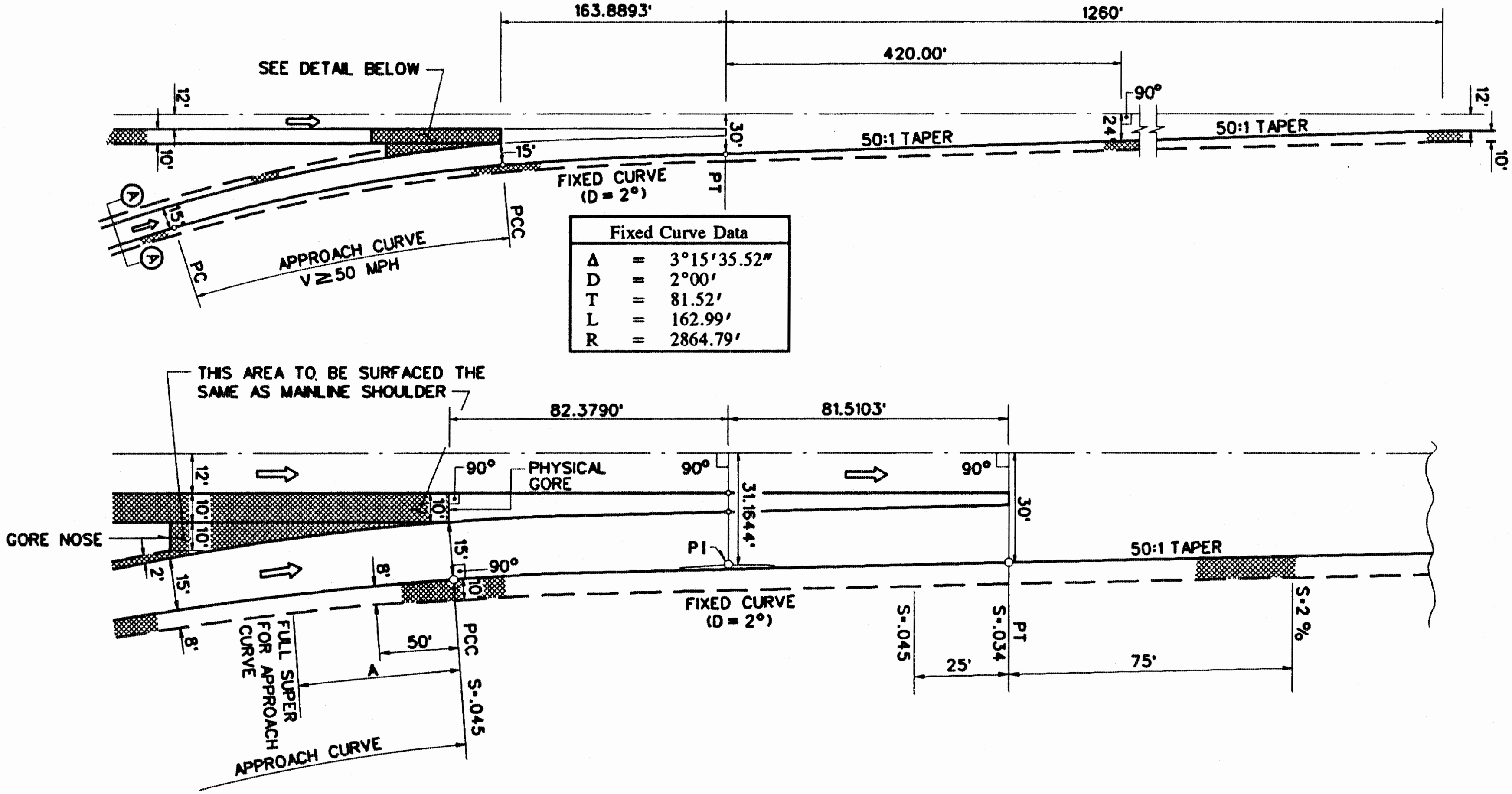
Fixed Curve Data		
Δ	=	3°00'37.04"
D	=	2°00'
T	=	75.27'
L	=	150.51'
R	=	2864.79'

Notes:

1. Figure applies where V=60 mph for mainline and V≥50 mph for approach curve. In restricted locations, the approach curve may be designed for V=45 mph.
2. See Table 10.3H for superelevation information on the approach curve, including the dimension "A".
3. See Figure 10.7I for typical curbed entrance gore.
4. See Figure 10.9A for typical ramp section.
5. See Figure 10.7L for entrances on curvilinear alignment.
6. A 300' minimum length of approach curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL TAPER ENTRANCE RAMP
(Uncurbed, V=60 on Mainline)

Figure 10.7F



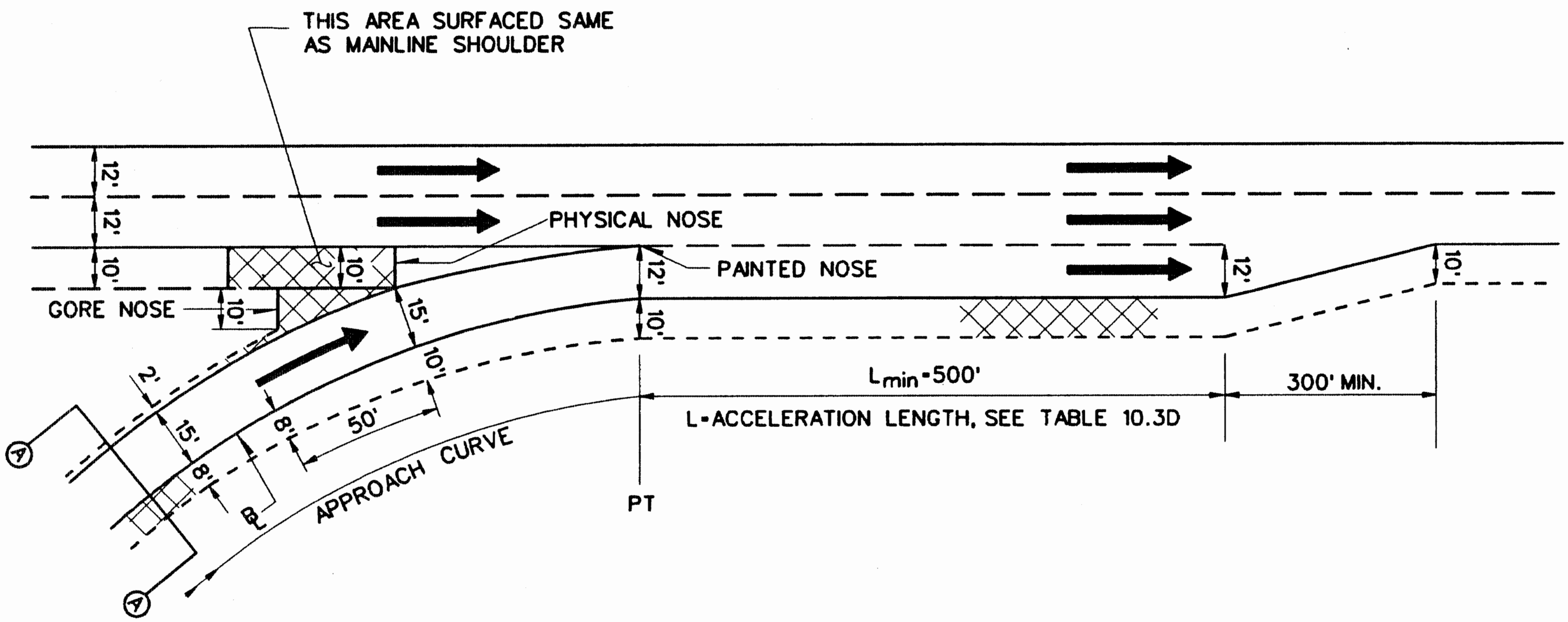
Fixed Curve Data		
Δ	=	3°15'35.52"
D	=	2°00'
T	=	81.52'
L	=	162.99'
R	=	2864.79'

Notes:

1. Figure applies where V=70 mph for mainline and V≥50 mph for approach curve.
2. See Table 10.3H for superelevation information on the approach curve, including the dimension "A".
3. See Figure 10.7I for typical curbed entrance gore.
4. See Figure 10.9A for typical ramp section.
5. See Figure 10.7L for entrances on curvilinear alignment.
6. A 300' minimum length of approach curve is desirable.
7. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL TAPER ENTRANCE RAMP
(Uncurbed, V=70 on Mainline)

Figure 10.7G

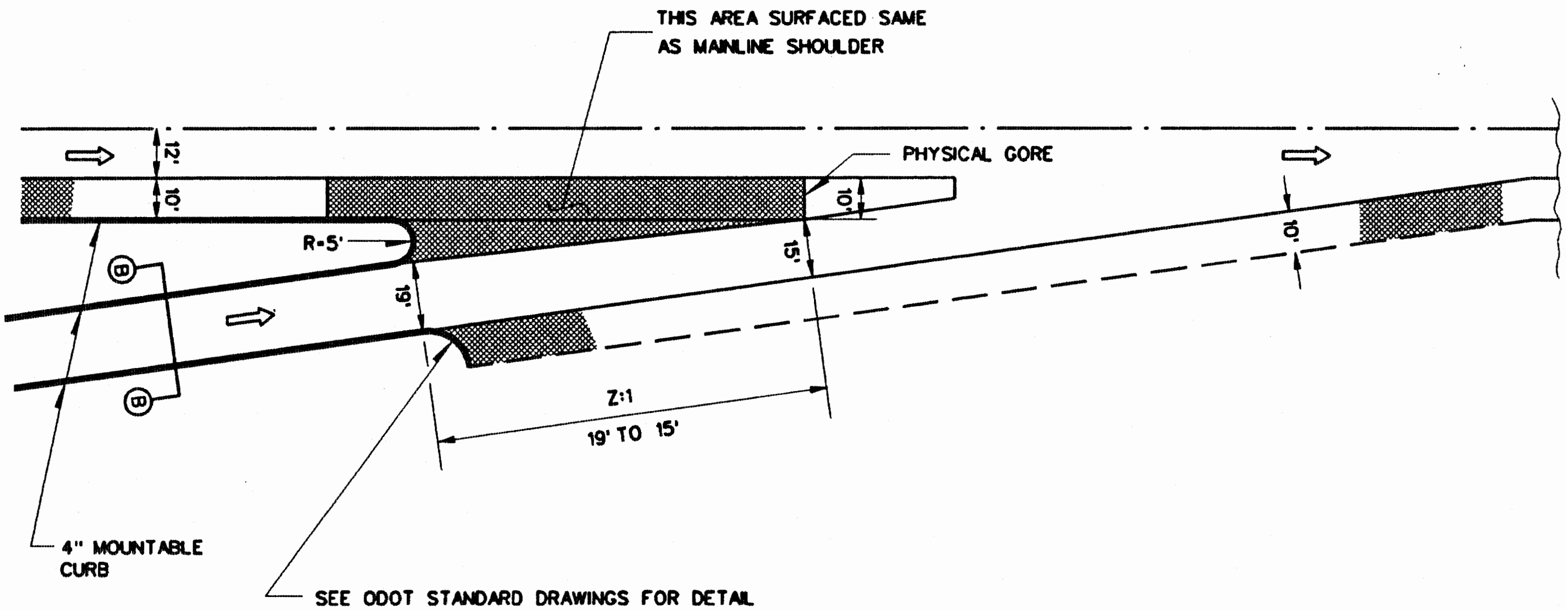


Notes:

1. Figure applies where taper design is inappropriate.
2. See Table 10.3C for superelevation information.
3. See Figure 10.7I for typical curbed entrance gore.
4. See Figure 10.9A for typical ramp section.
5. A 300′ minimum length of approach curve is desirable.
6. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL PARALLEL-LANE ENTRANCE RAMP
(Uncurbed)

Figure 10.7H



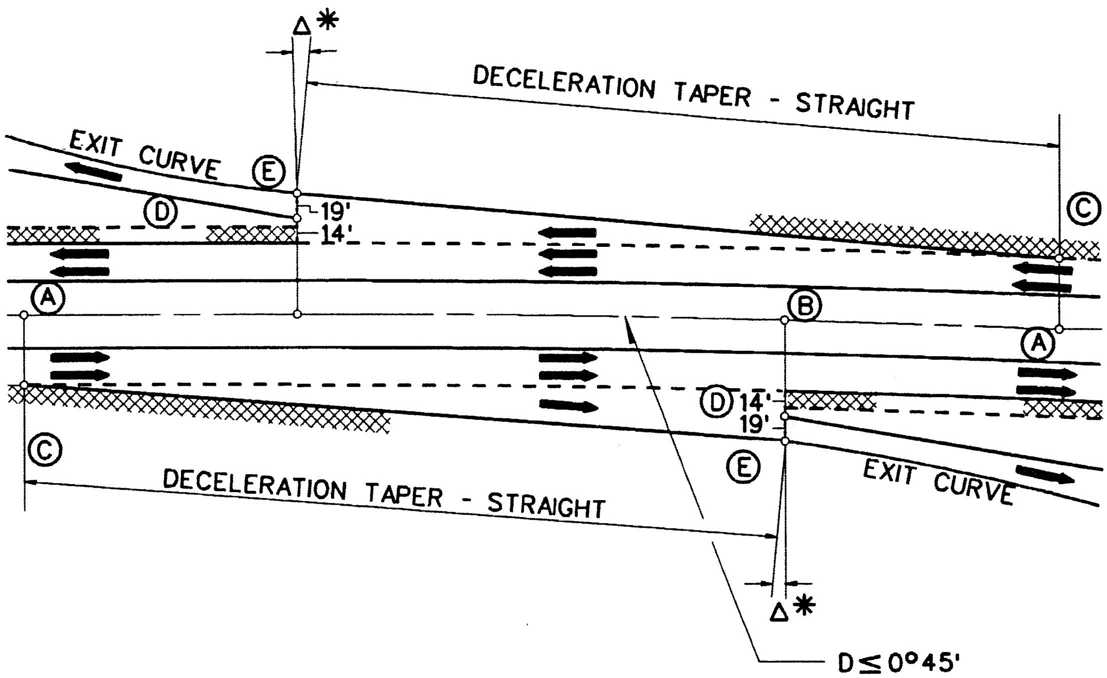
Design Speed of Ramp Curve (mph)	Z – Length of Taper per Foot of Ramp Width Reduction (ft)
30	15
40	20
50	25
60	30
70 - Tangent	35

Notes:

1. Figure applies where curb is warranted on ramp. See Section 10.9.
2. See Figure 10.9A for typical ramp section.
3. Refer to Traffic Engineering Division for detailed pavement markings.

ODOT TYPICAL ENTRANCE GORE
(Curbed)

Figure 10.7I



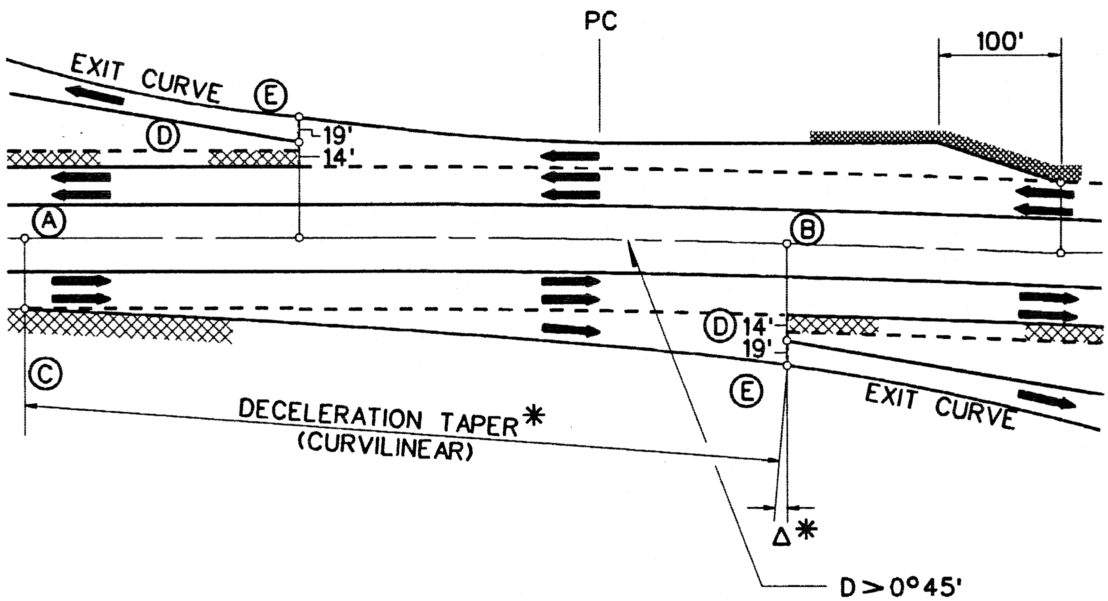
* FOR A DECELERATION TAPER = 500',Δ = 3°46'46.30"
FOR A DECELERATION TAPER = 550',Δ = 3°26' 10.49"

Note: When the main roadway alignment is on a curve whose degree does not exceed 00°45′, the following geometric controls will be required. Points "A" and "B" are located on the centerline of the median or, in the case of a median transition, on the base line of the lane such that the Arc "A" - "B" is 500′ in length for 60 mph and 550′ for 70 mph. Through Point "A", a radial line is passed to obtain Point "C," the beginning of the taper. Similarly, a radial line is passed through "B" to obtain Point "D" 14′ beyond the outside edge of pavement. From Point "D," deflect an angle of Δ and establish Point "E" 19′ beyond Point "D." The straight line connecting Point "C" and Point "E" establishes the bearing and length of the taper and the bearing of the tangent of the Exit Curve.

These geometric controls are to be used in conjunction with the criteria on exit ramps in Section 10.3. See the Geometric Design Branch for more information.

EXIT TERMINALS ON CURVILINEAR ALIGNMENT
(D=00°45′ max)

Figure 10.7J



* FOR A DECELERATION TAPER = 500', Δ = 3°46'46.30' AND OFFSET 6.6' PER STATION
FOR A DECELERATION TAPER = 550', Δ = 3°26'10.49" AND OFFSET 6.0' PER STATION

Note: When the main roadway is on a curve whose degree exceeds 00°45′ minimum, the following geometric controls will be required <u>on the inside of the roadway curve</u>. Points "A" and "B" are located 500′ (or 550′) apart along the arc on the centerline of median or, in the case of a median transition, on the base line of the lane. A radial line is passed through Point "A" to establish Point "C," the beginning of the taper. A radial line is also passed through Point "B" to establish Point "D" 14′ from the edge of pavement. From Point "D," deflect an angle of Δ and establish Point "E" 19′ beyond Point "D."

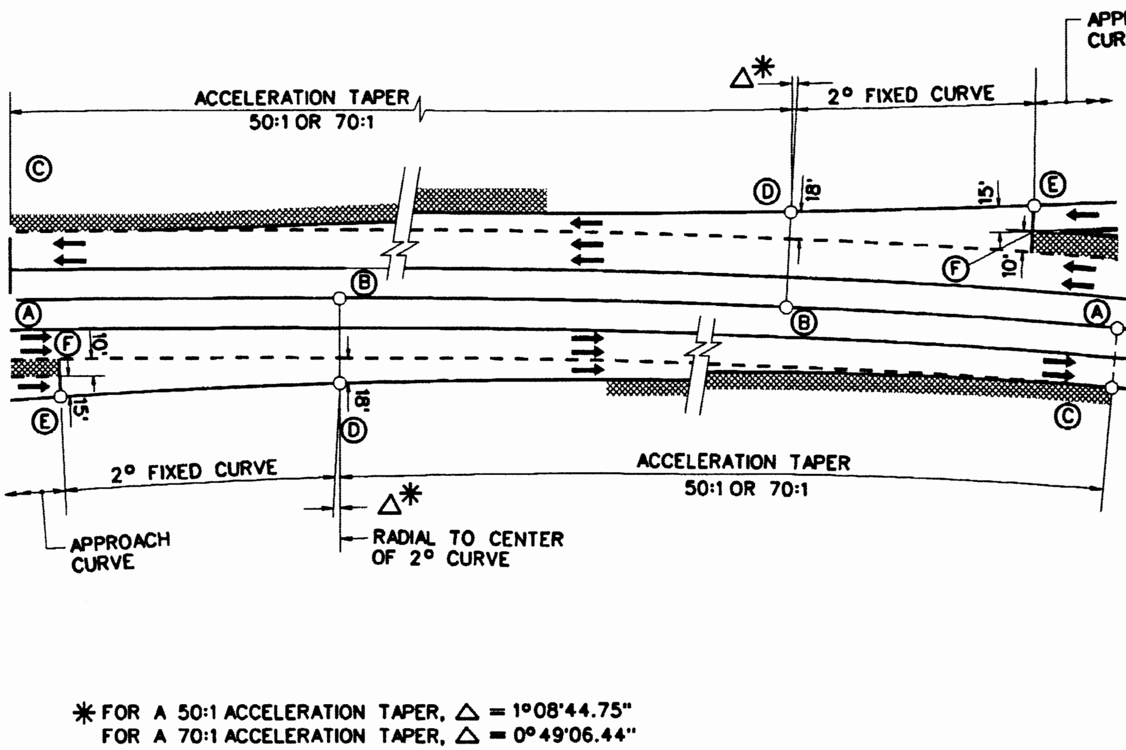
The bearing of the line from Point "D" to Point "E" is the bearing of the radius of the exit curve. The survey data for the ramp will begin at Point "E." The edge line on the inside of the ramp from Point "C" to Point "E" shall be identified as "Offset 6.6 Feet per Station for 60 mph" or "Offset 6.0 Feet per Station for 70 mph."

For the design of exit terminals on the <u>outside of the roadway curve</u>, consult with the Geometric Design Branch. For D > 00°45′ and the exit ramp on the outside of the curve, the use of a parallel-lane exit ramp should be considered.

These geometric controls are to be used in conjunction with the criteria on exit ramps in Section 10.3. See the Geometric Design Branch for more information.

EXIT TERMINALS ON CURVILINEAR ALIGNMENT
(D=00°45′ Minimum)

Figure 10.3K



Note: When the main roadway alignment is on a curve, the following geometric controls will be required. Points "A" and "B" are located on the centerline of survey or baseline of construction, such that the arc length "A" to "B" will be either a 900′ arc length for a 50:1 taper or a 1260′ arc length for a 70:1 taper rate. A radial line is passed through "A" to establish Point "C," on outside edge of pavement; also, a radial is passed through "B" to produce Point "D" 18′ beyond the outside edge of main line pavement. Center of 2° curve is established by projecting a deflection angle of Δ from a radial line on ℄ curve passed through Point "D" as shown on detail. Point "E" is established on a radial line on 2° curve passed through F, a point of intersection of an offset curve concentric with curve on base line and an offset curve concentric with 2° curve on ramp.

Above design is applicable where design speed of approach curve is 50 mph or more. Where V<50 mph, a parallel-lane entrance design should be considered.

These controls are to be used in conjunction with the criteria on entrance ramps in Section 10.3. See the Geometric Design Branch for more information.

ENTRANCE TERMINALS ON CURVILINEAR ALIGNMENT

Figure 10.3L

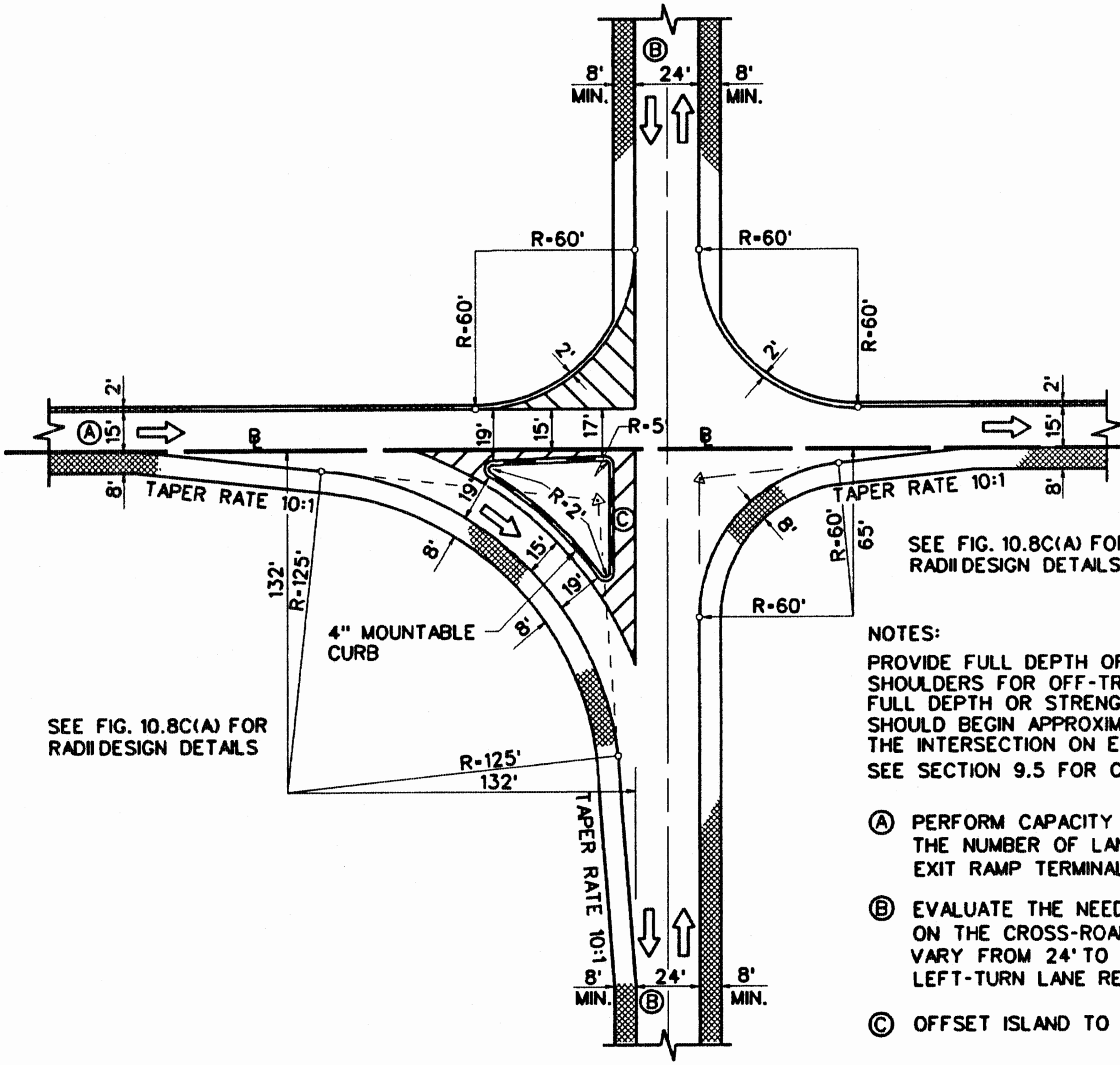
10.8 TYPICAL RAMP TERMINAL FIGURES

This Section presents design details for ramp/crossing road or ramp/frontage road intersections. Table 10.8A summarizes the figures in Section 10.8.

Table 10.8A

TYPICAL RAMP TERMINAL FIGURES

Figure Number	Figure Title
Figure 10.8A	Ramp Terminal at 2-Lane Crossroad (Rural) (WB-114 Design Vehicle)
Figure 10.8B	Ramp Terminal at 2-Lane Crossroad (Rural) (WB-114 Design Vehicle)
Figure 10.8C	Corner Radii Designs (Ramp Terminals)
Figure 10.8D	Ramp Terminal at 4-Lane Undivided Crossroad (Rural) (WB-114 Design Vehicle)
Figure 10.8E	Ramp Terminal at 4-Lane Undivided Crossroad (Rural) (WB-114 Design Vehicle)
Figure 10.8F	Ramp Terminal at 4-Lane Divided Crossroad (Rural) (WB-114 Design Vehicle)
Figure 10.8G	Ramp Terminal at 4-Lane Divided Crossroad (Rural) (WB-114 Design Vehicle)
Figure 10.8H	Ramp Terminal at 4-Lane Undivided Crossroad (Urban) (WB-114 Design Vehicle)
Figure 10.8I	Ramp Terminal at 4-Lane Undivided Crossroad (Urban) (WB-114 Design Vehicle)
Figure 10.8J	Ramp Terminal at 4-Lane Divided Crossroad (Urban) (WB-114 Design Vehicle)
Figure 10.8K	Buttonhook Exit Terminal to Frontage Road (WB-114 Design Vehicle)
Figure 10.8L	Buttonhook Entrance Terminal to Frontage Road (WB-114 Design Vehicle)
Figure 10.8M	Ramp Terminal at 2-Lane Crossroad (Rural) (WB-67 Design Vehicle)
Figure 10.8N	Ramp Terminal at 2-Lane Crossroad (Rural) (WB-67 Design Vehicle)
Figure 10.8O	Ramp Terminal at 4-Lane Undivided Crossroad (Rural) (WB-67 Design Vehicle)



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.
SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

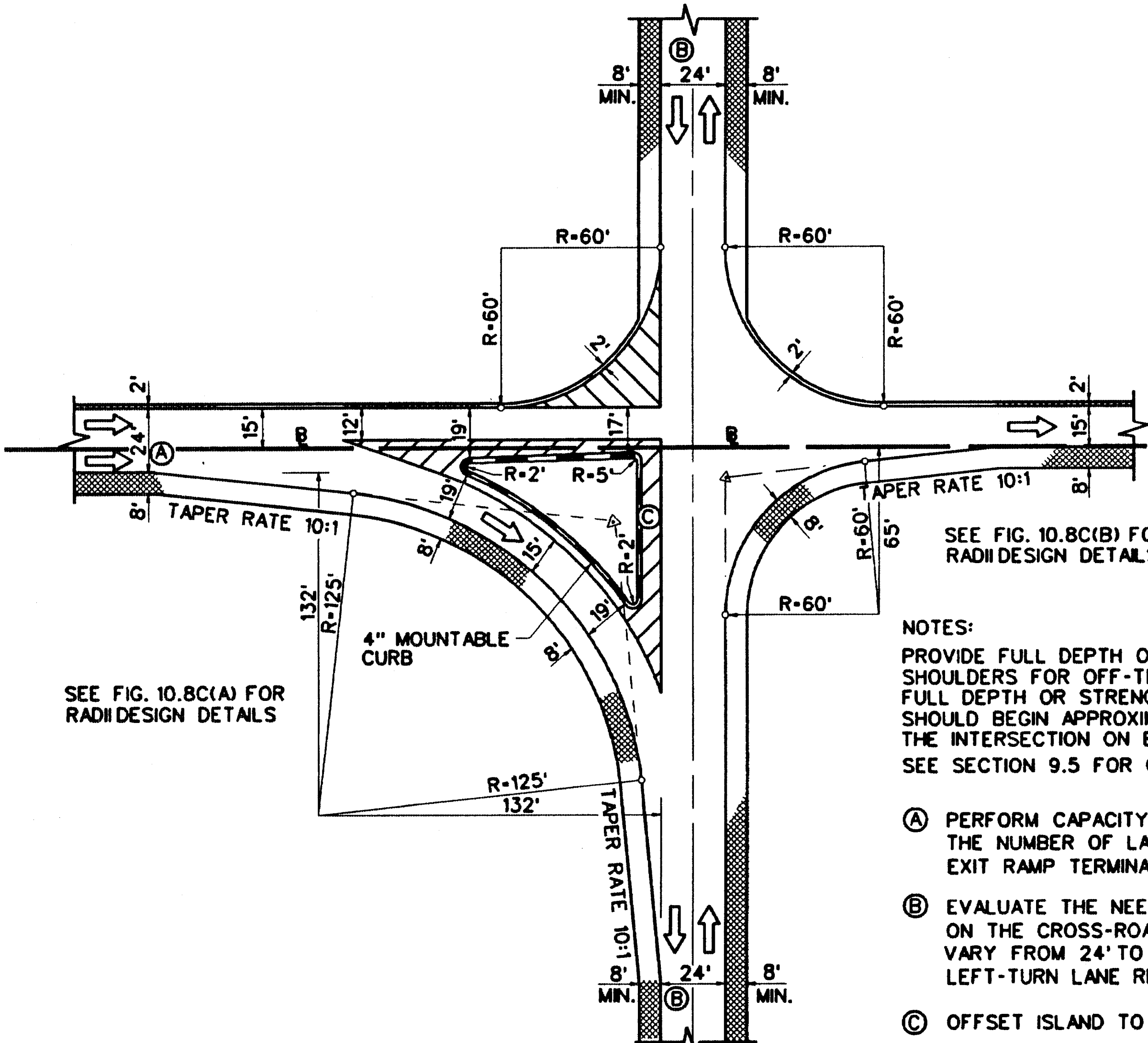
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 24' TO 40' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

RAMP TERMINAL AT 2-LANE CROSSROAD (RURAL)
(WB-114 Design Vehicle)

Figure 10.8A



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.
SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

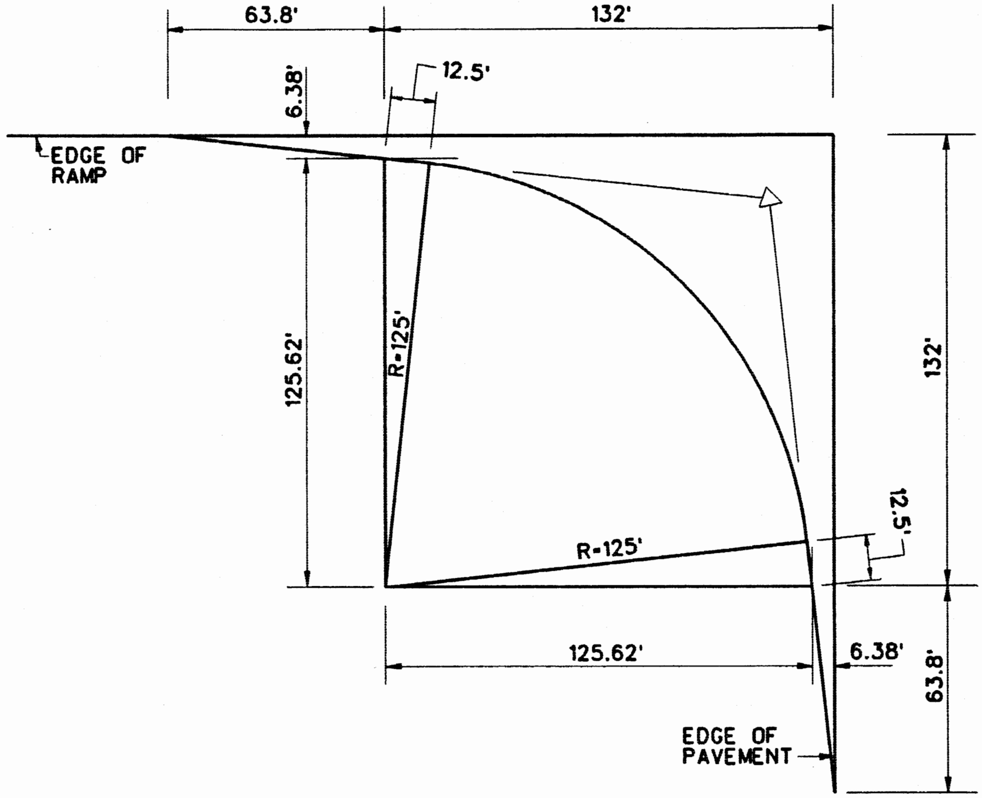
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 24' TO 40' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

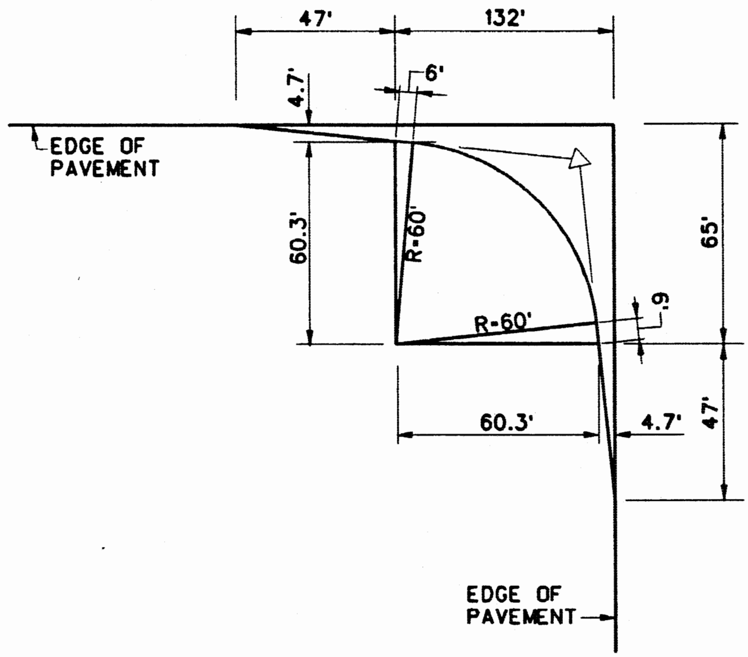
Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

RAMP TERMINAL AT 2-LANE CROSSROAD (RURAL)
(WB-114 Design Vehicle)

Figure 10.8B



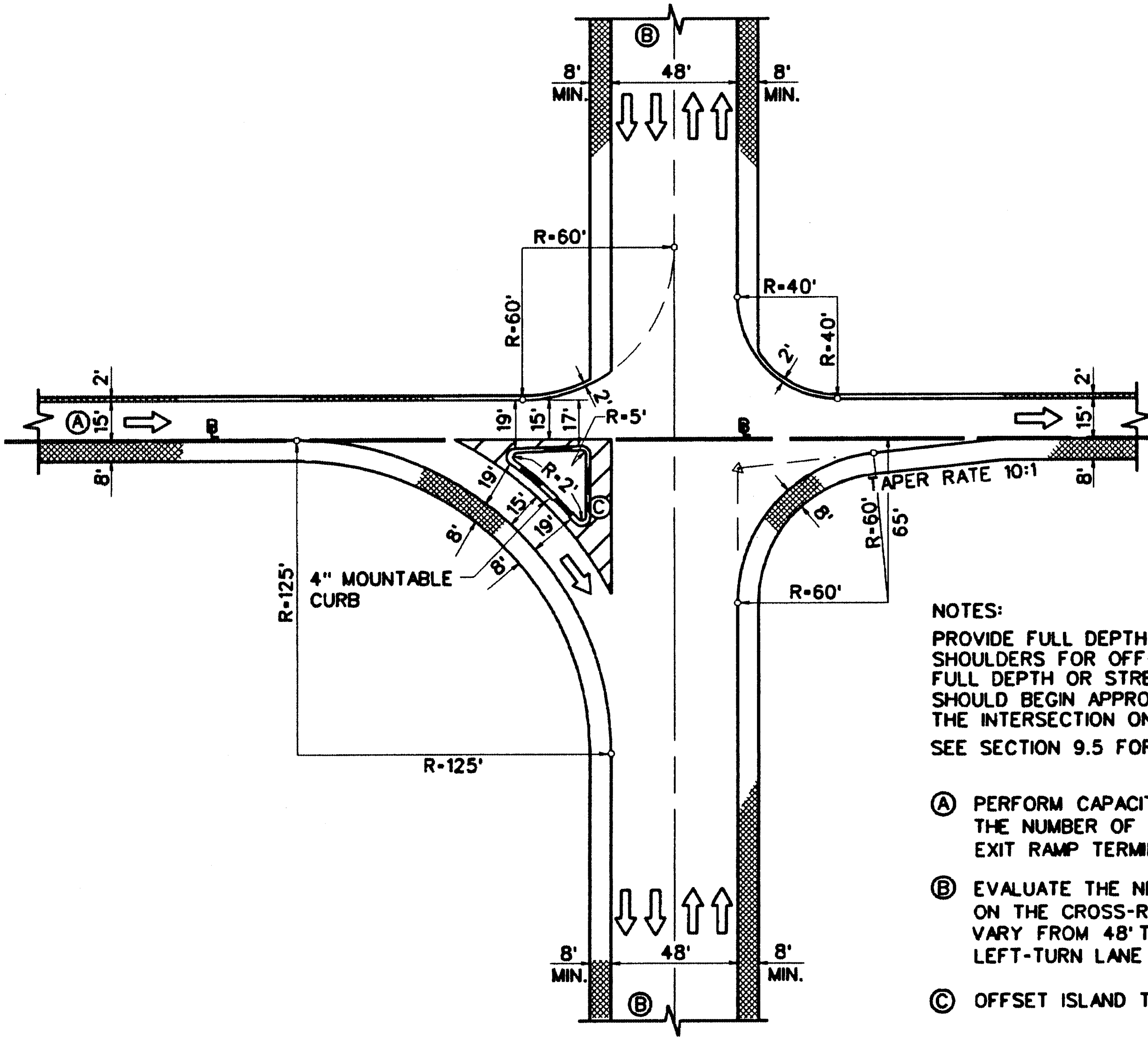
(A) 125' RADIUS - 7' OFFSET - 10:1 TAPER



(B) 60' RADIUS - 5' OFFSET - 10:1 TAPER

CORNER RADII DESIGNS
(Ramp Terminals)

Figure 10.8C



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

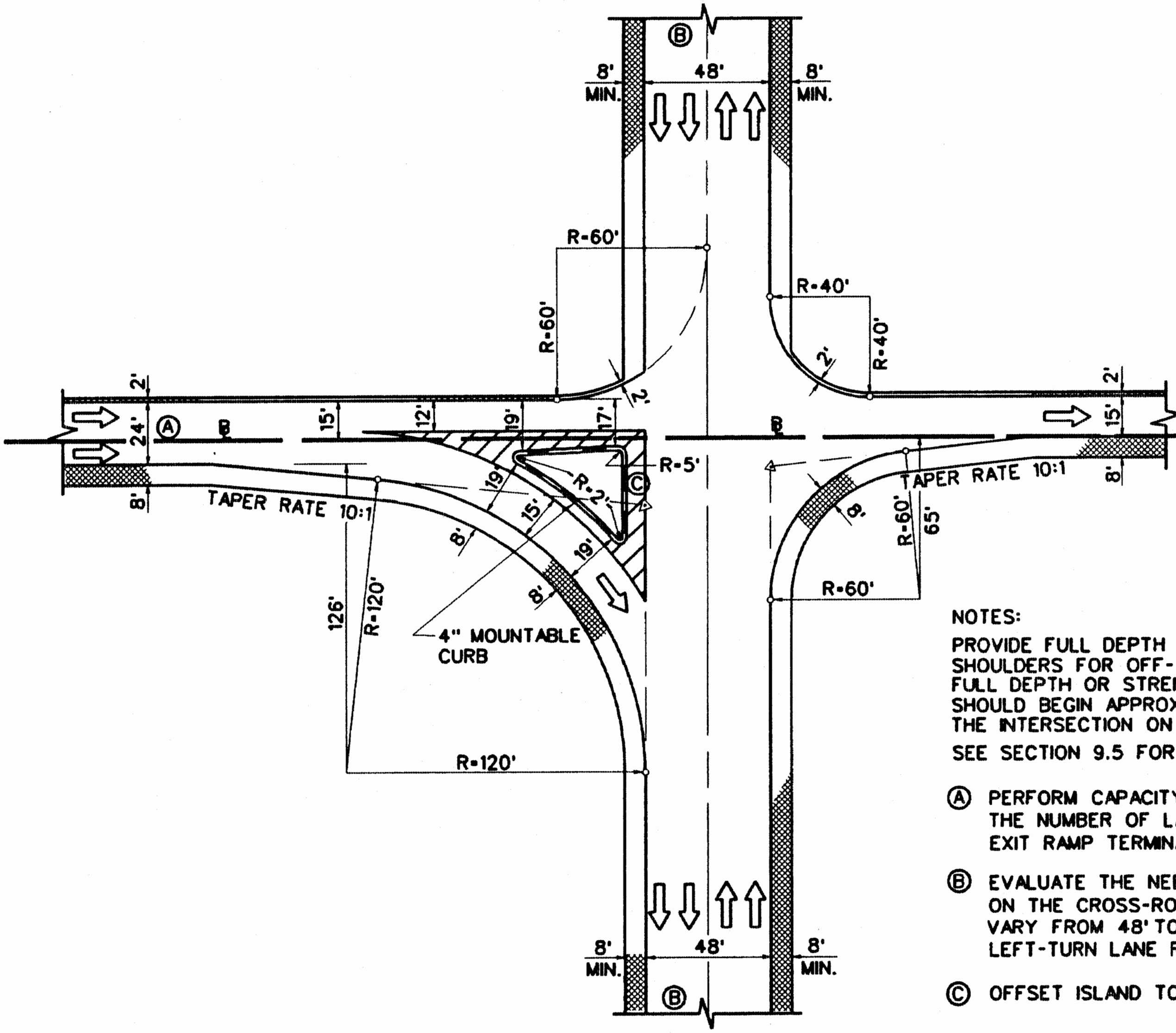
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 48' TO 64' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (RURAL)
(WB-114 Design Vehicle)

Figure 10.8D



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

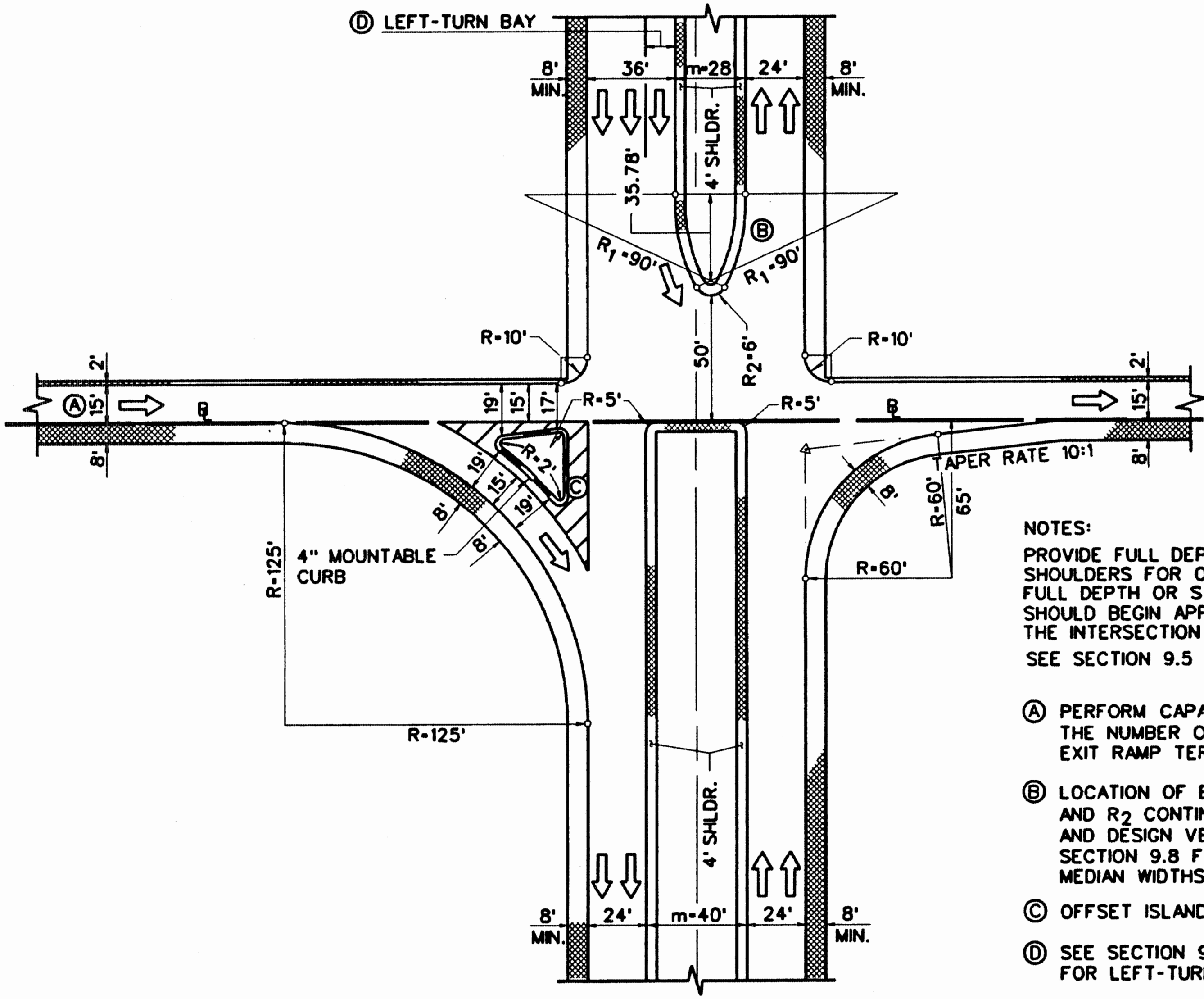
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 48' TO 64' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (RURAL)
(WB-114 Design Vehicle)

Figure 10.8E



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.
SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

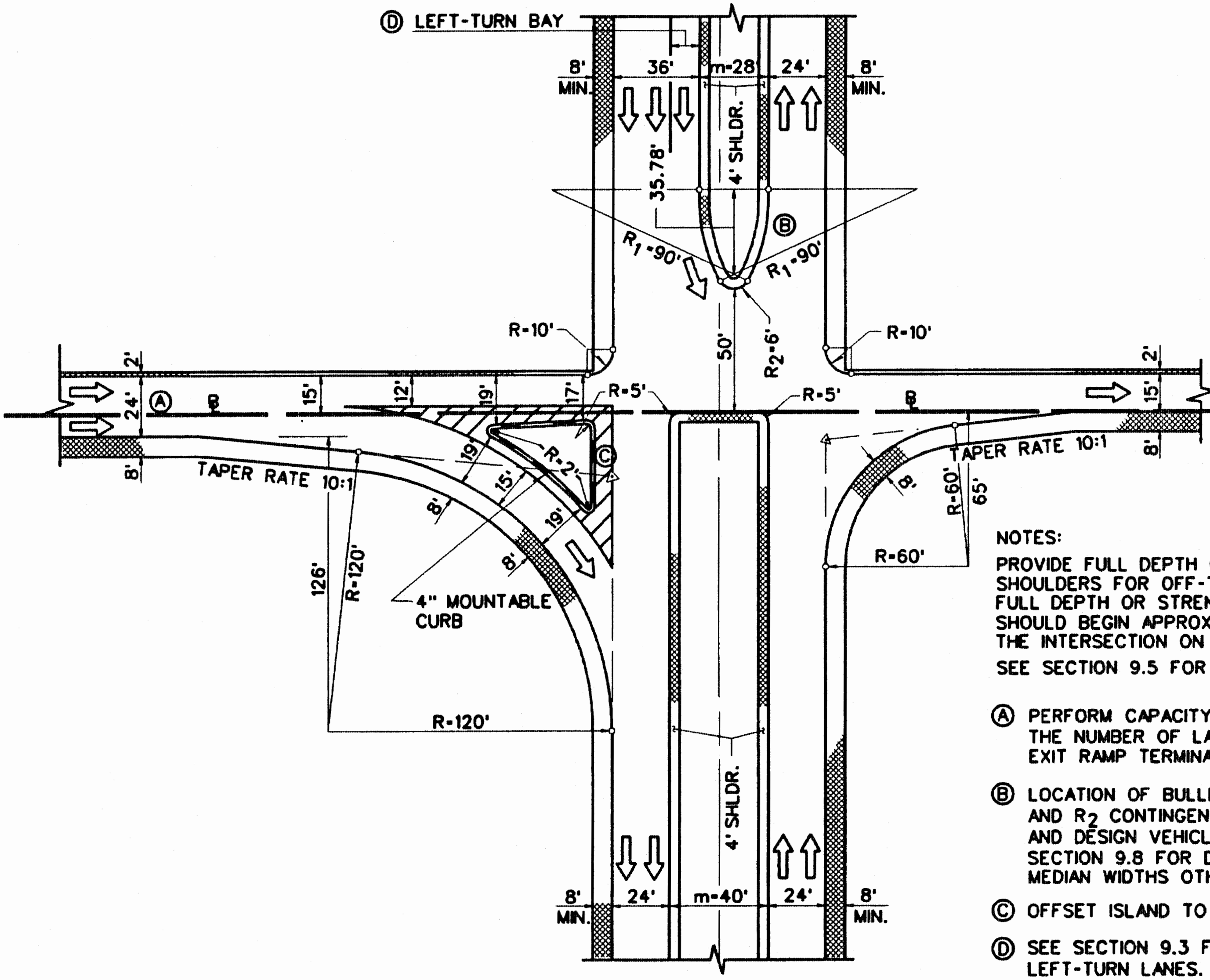
Ⓑ LOCATION OF BULLET NOSE AND RADIUS R_1 AND R_2 CONTINGENT UPON MEDIAN WIDTH AND DESIGN VEHICLE TURNING PATH. SEE SECTION 9.8 FOR DESIGN CRITERIA FOR MEDIAN WIDTHS OTHER THAN 40'.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

Ⓓ SEE SECTION 9.3 FOR DESIGN CRITERIA FOR LEFT-TURN LANES.

RAMP TERMINAL AT 4-LANE DIVIDED CROSSROAD (RURAL)
(WB-114 Design Vehicle)

Figure 10.8F



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

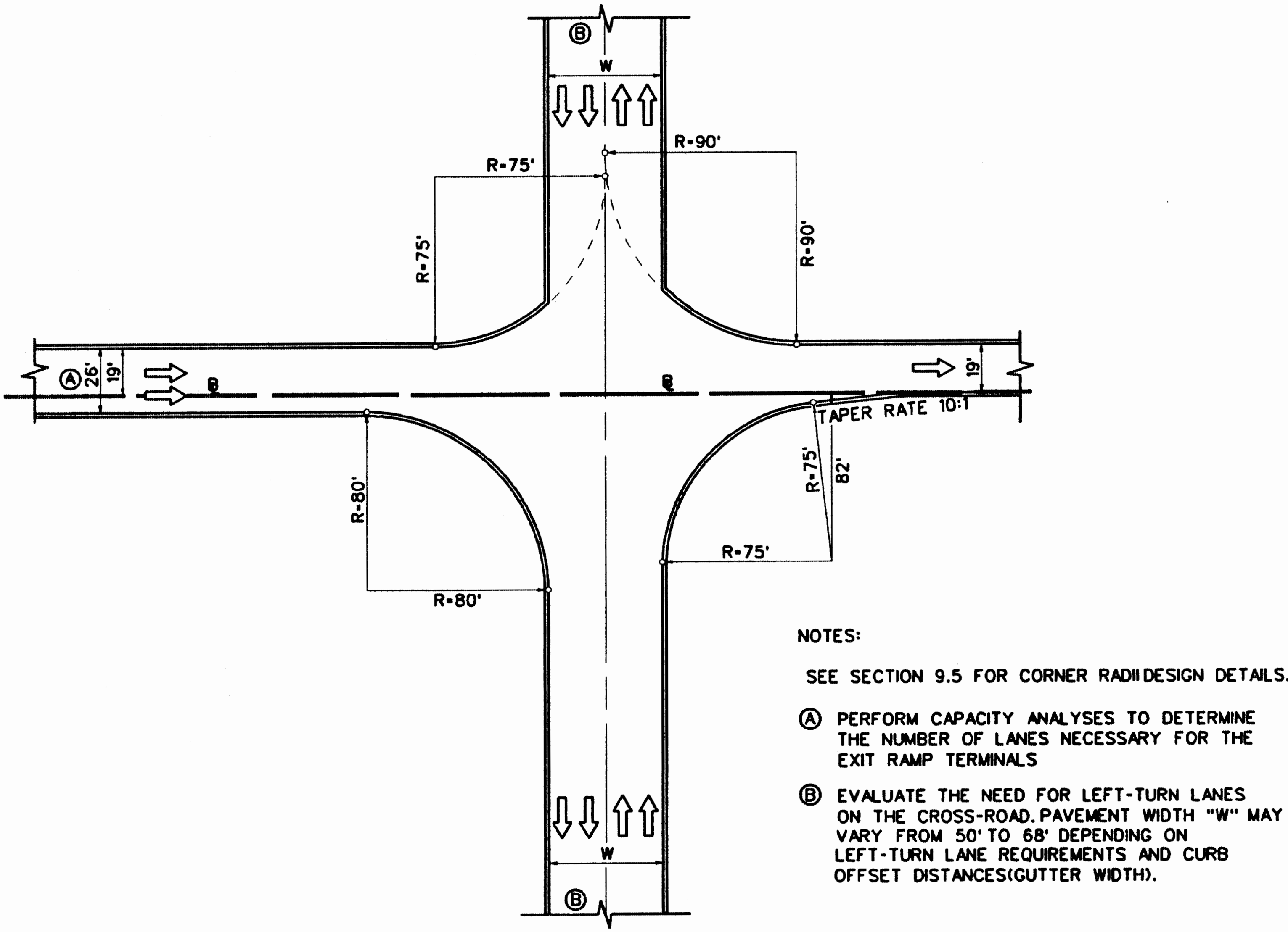
Ⓑ LOCATION OF BULLET NOSE AND RADIUS R_1 AND R_2 CONTINGENT UPON MEDIAN WIDTH AND DESIGN VEHICLE TURNING PATH. SEE SECTION 9.8 FOR DESIGN CRITERIA FOR MEDIAN WIDTHS OTHER THAN 40'.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

Ⓓ SEE SECTION 9.3 FOR DESIGN CRITERIA FOR LEFT-TURN LANES.

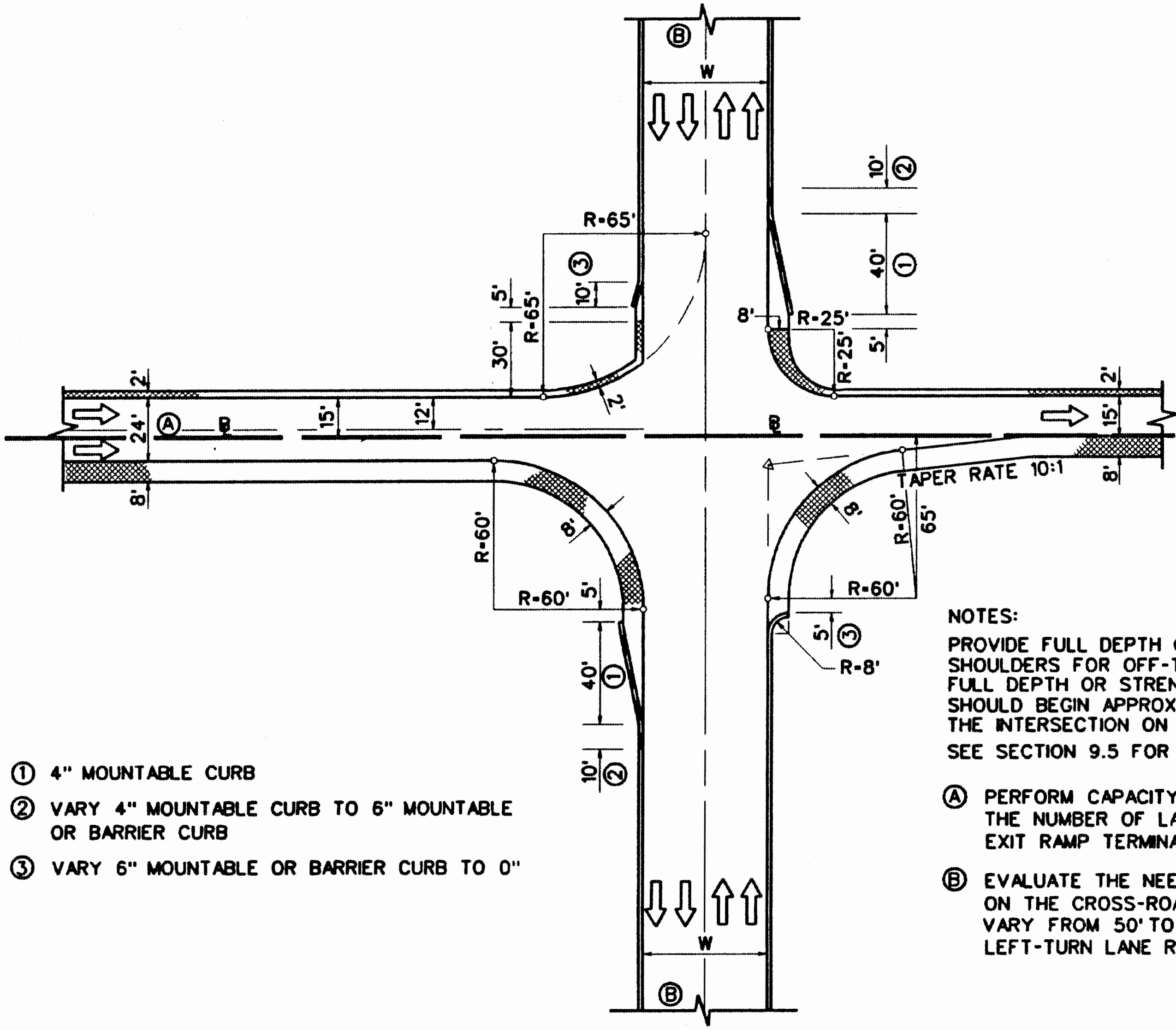
RAMP TERMINAL AT 4-LANE DIVIDED CROSSROAD (RURAL)
(WB-114 Design Vehicle)

Figure 10.8G



RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (URBAN)
(WB-114 Design Vehicle)

Figure 10.8H



① 4" MOUNTABLE CURB

② VARY 4" MOUNTABLE CURB TO 6" MOUNTABLE OR BARRIER CURB

③ VARY 6" MOUNTABLE OR BARRIER CURB TO 0"

NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 50' TO 68' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

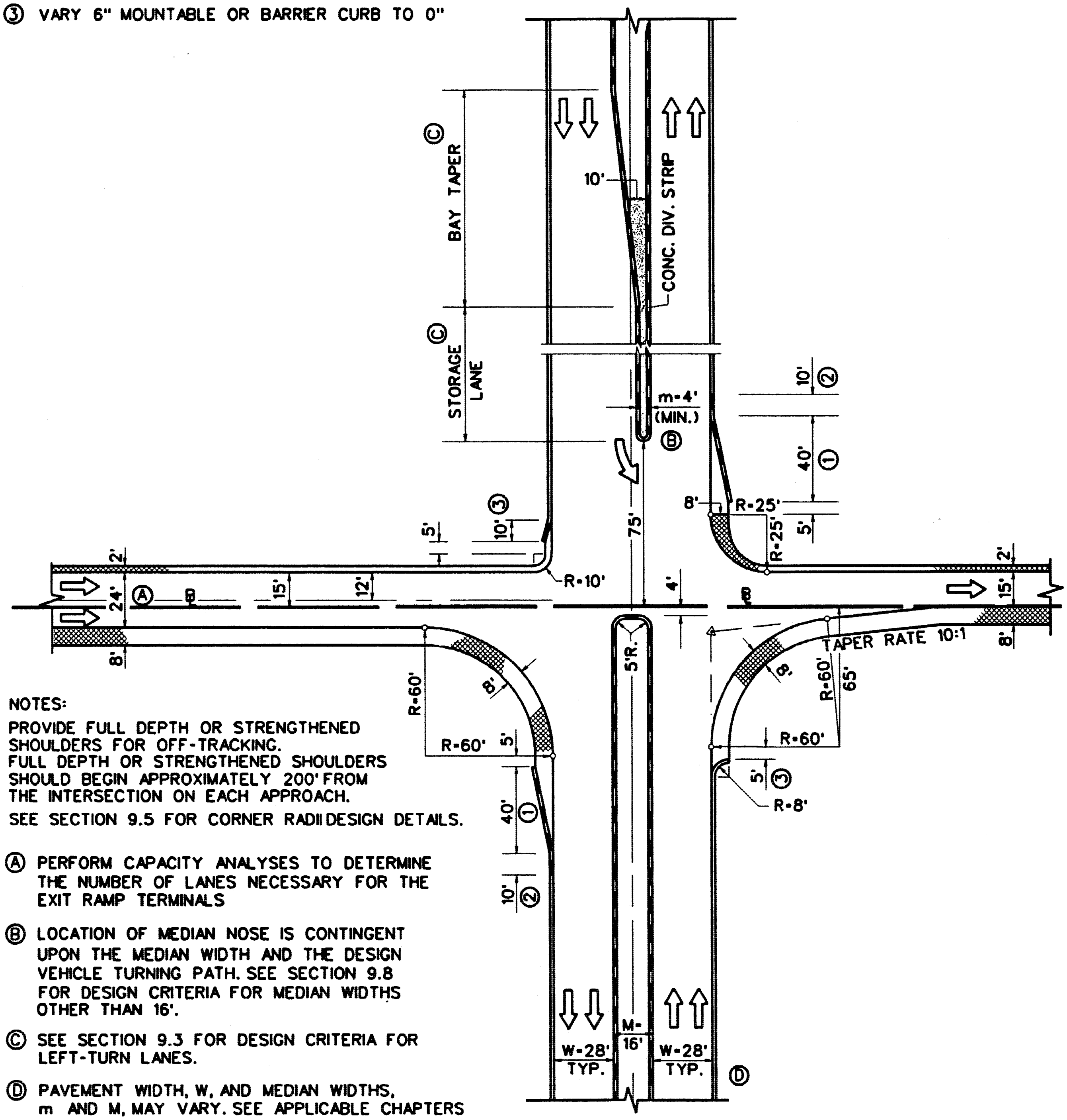
RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (URBAN)
(WB-114 Design Vehicle)

Figure 10.8I

① 4" MOUNTABLE CURB

② VARY 4" MOUNTABLE CURB TO 6" MOUNTABLE OR BARRIER CURB

③ VARY 6" MOUNTABLE OR BARRIER CURB TO 0"



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING. FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ LOCATION OF MEDIAN NOSE IS CONTINGENT UPON THE MEDIAN WIDTH AND THE DESIGN VEHICLE TURNING PATH. SEE SECTION 9.8 FOR DESIGN CRITERIA FOR MEDIAN WIDTHS OTHER THAN 16'.

Ⓒ SEE SECTION 9.3 FOR DESIGN CRITERIA FOR LEFT-TURN LANES.

Ⓓ PAVEMENT WIDTH, W, AND MEDIAN WIDTHS, m AND M, MAY VARY. SEE APPLICABLE CHAPTERS FOR WIDTHS.

RAMP TERMINAL AT 4-LANE DIVIDED CROSSROAD (URBAN)
(WB-114 Design Vehicle)

Figure 10.8J

NOTES:

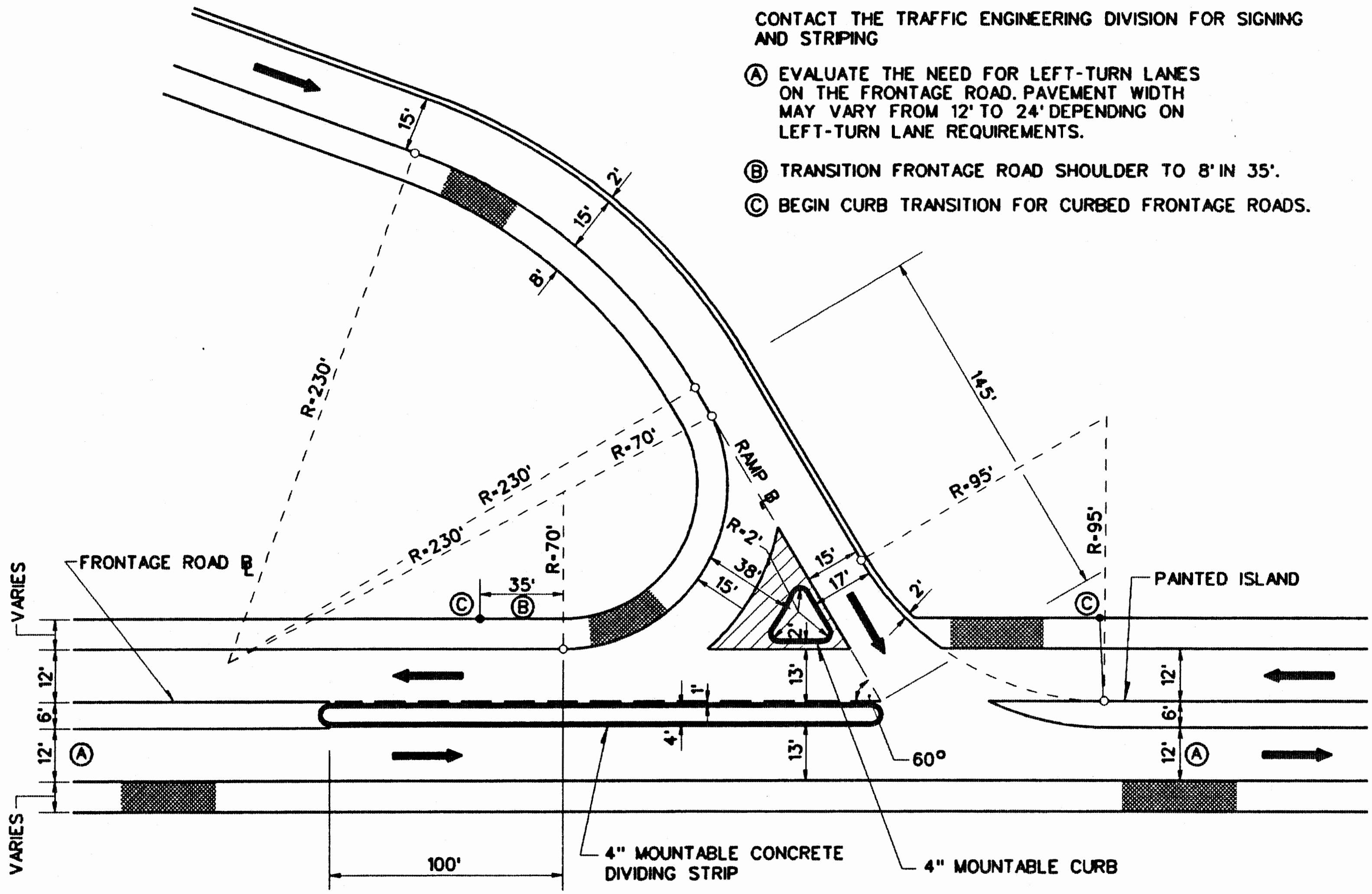
PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.

CONTACT THE TRAFFIC ENGINEERING DIVISION FOR SIGNING AND STRIPING

(A) EVALUATE THE NEED FOR LEFT-TURN LANES ON THE FRONTAGE ROAD. PAVEMENT WIDTH MAY VARY FROM 12' TO 24' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

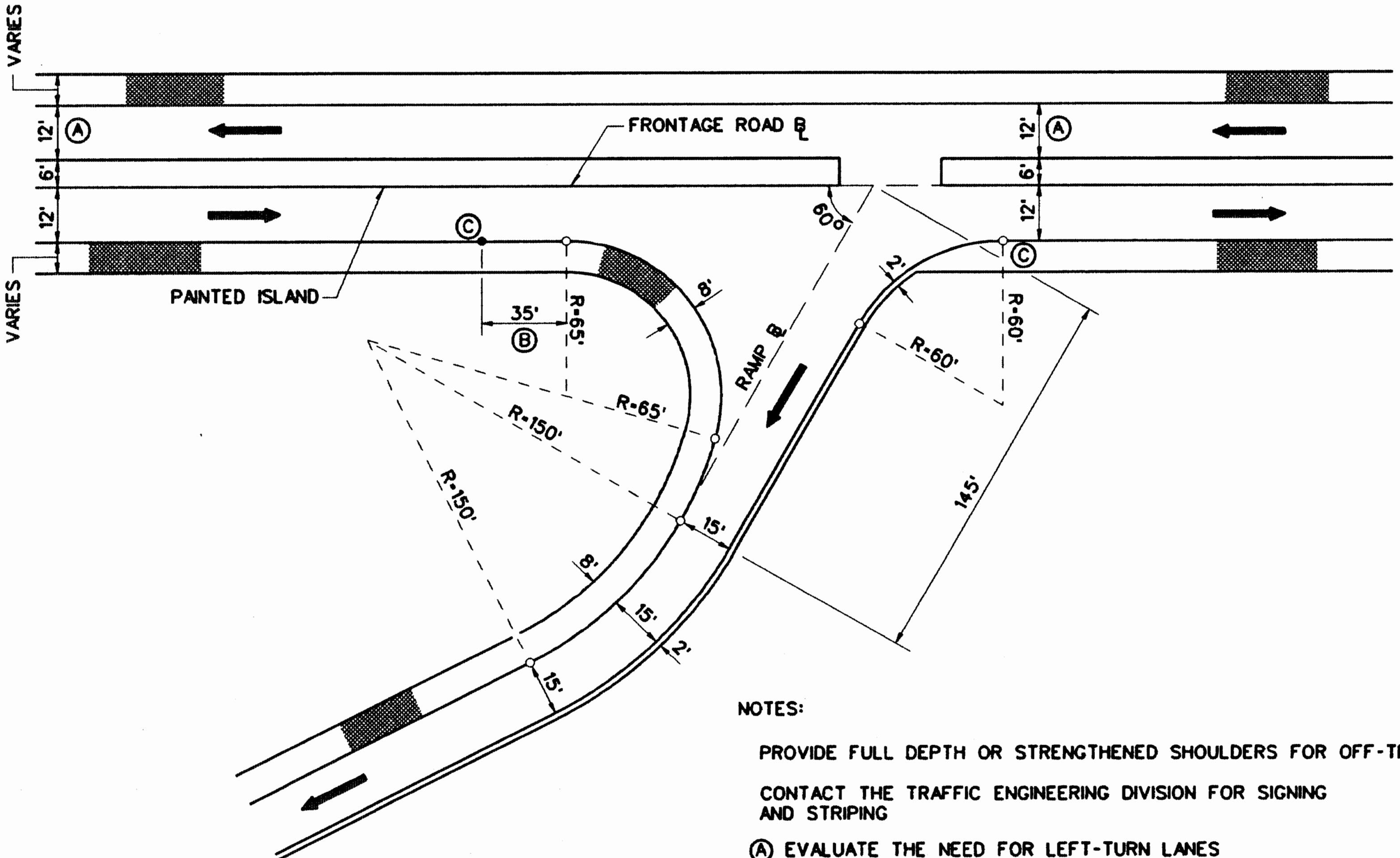
(B) TRANSITION FRONTAGE ROAD SHOULDER TO 8' IN 35'.

(C) BEGIN CURB TRANSITION FOR CURBED FRONTAGE ROADS.



BUTTONHOOK EXIT TERMINAL TO FRONTAGE ROAD
(WB-114 Design Vehicle)

Figure 10.8K



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.

CONTACT THE TRAFFIC ENGINEERING DIVISION FOR SIGNING AND STRIPING

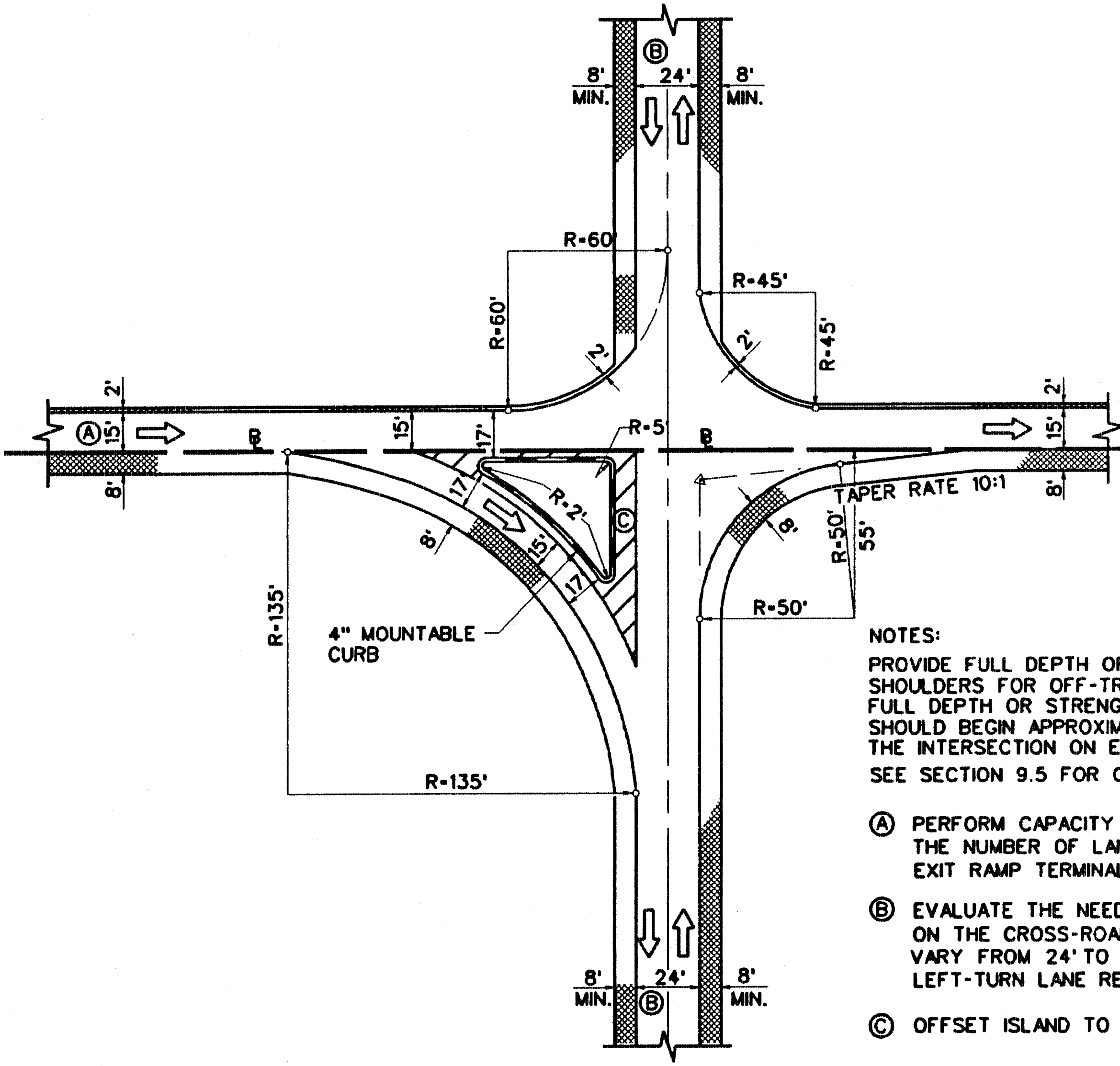
Ⓐ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE FRONTAGE ROAD. PAVEMENT WIDTH MAY VARY FROM 12' TO 24' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓑ TRANSITION FRONTAGE ROAD SHOULDER TO 8' IN 35'.

Ⓒ BEGIN CURB TRANSITION FOR CURBED FRONTAGE ROADS.

BUTTONHOOK ENTRANCE TERMINAL TO FRONTAGE ROAD
(WB-114 Design Vehicle)

Figure 10.8L



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.
SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

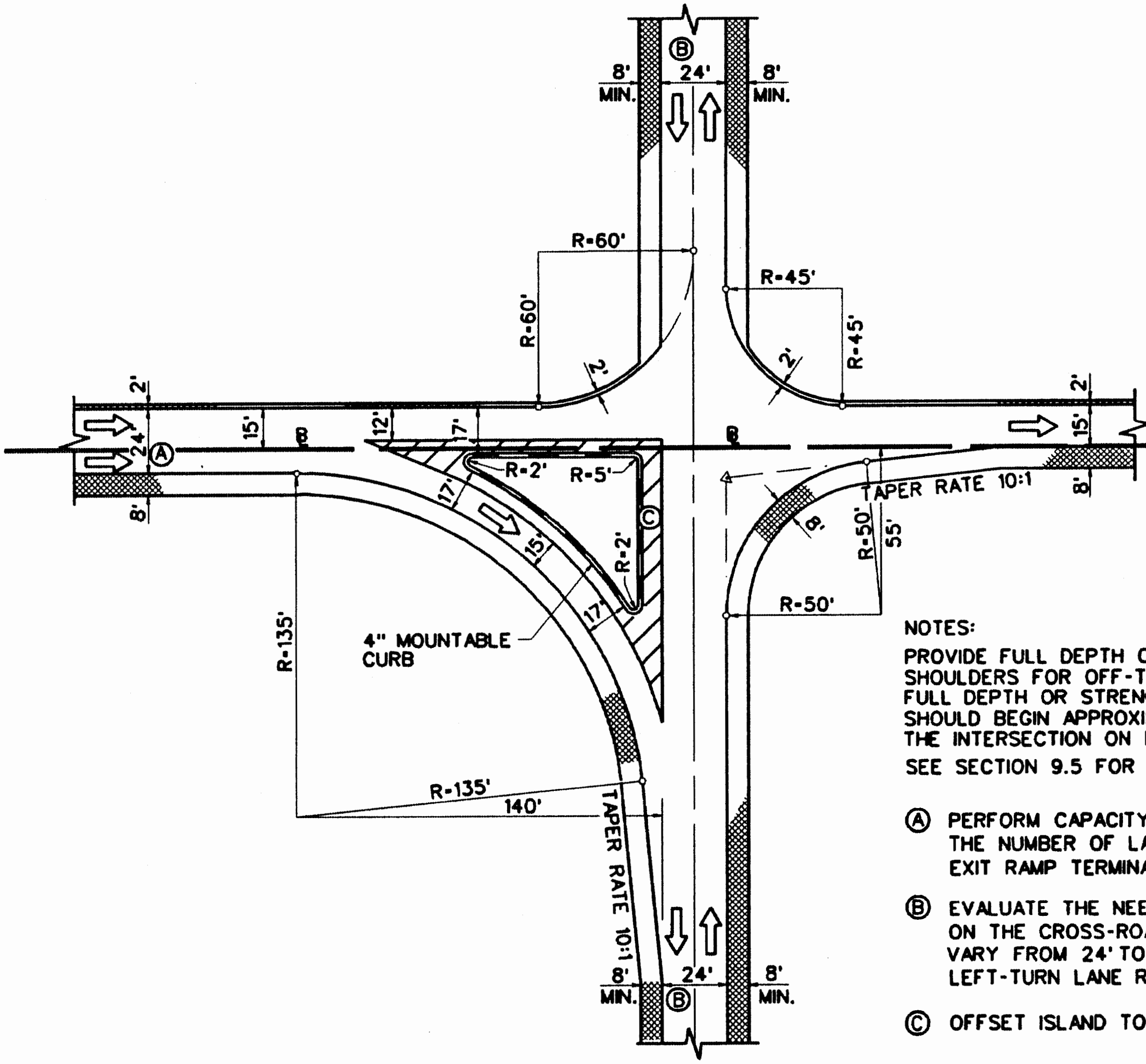
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 24' TO 40' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

RAMP TERMINAL AT 2-LANE CROSSROAD (RURAL)
(WB-67 Design Vehicle)

Figure 10.8M



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

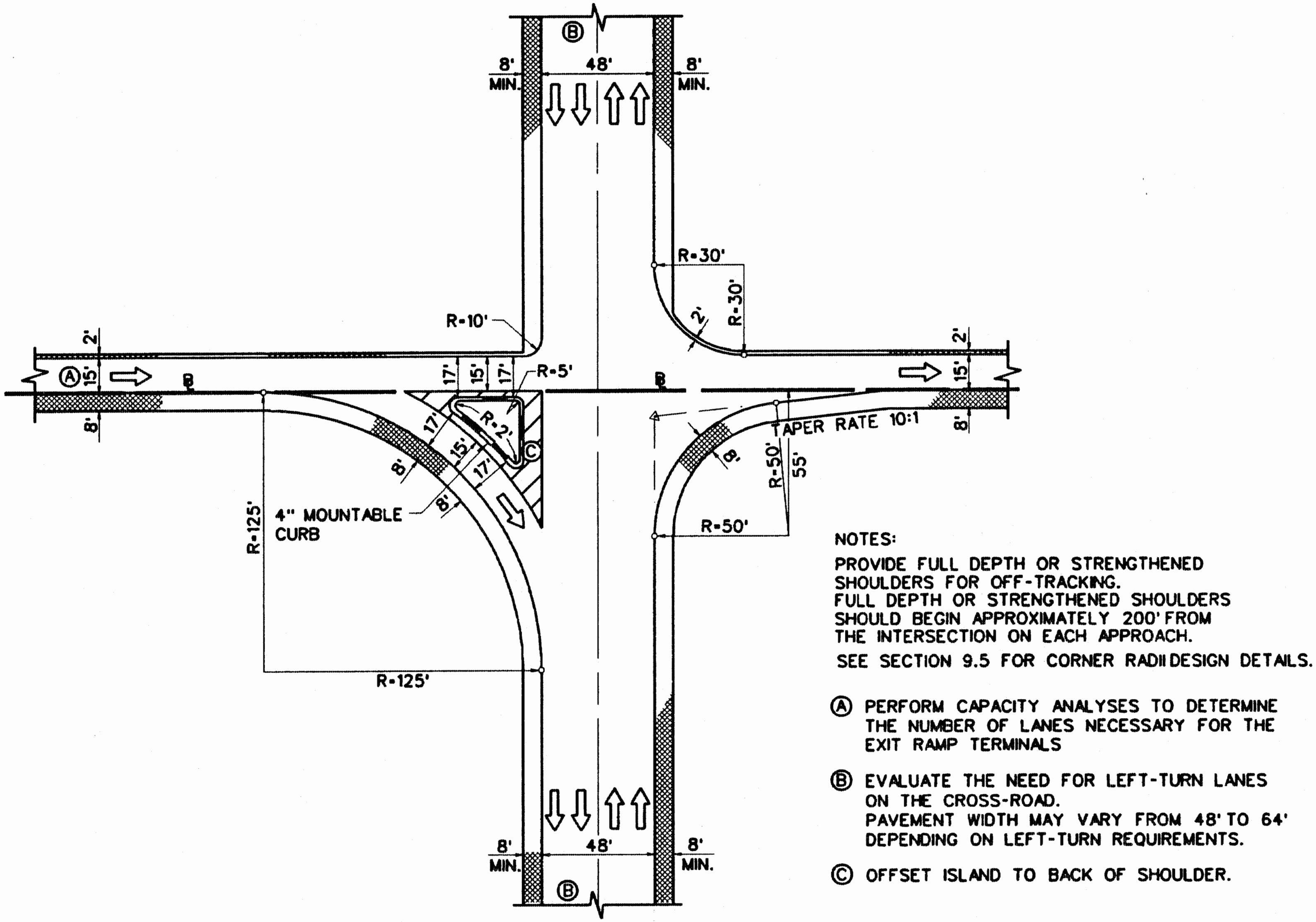
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 24' TO 40' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

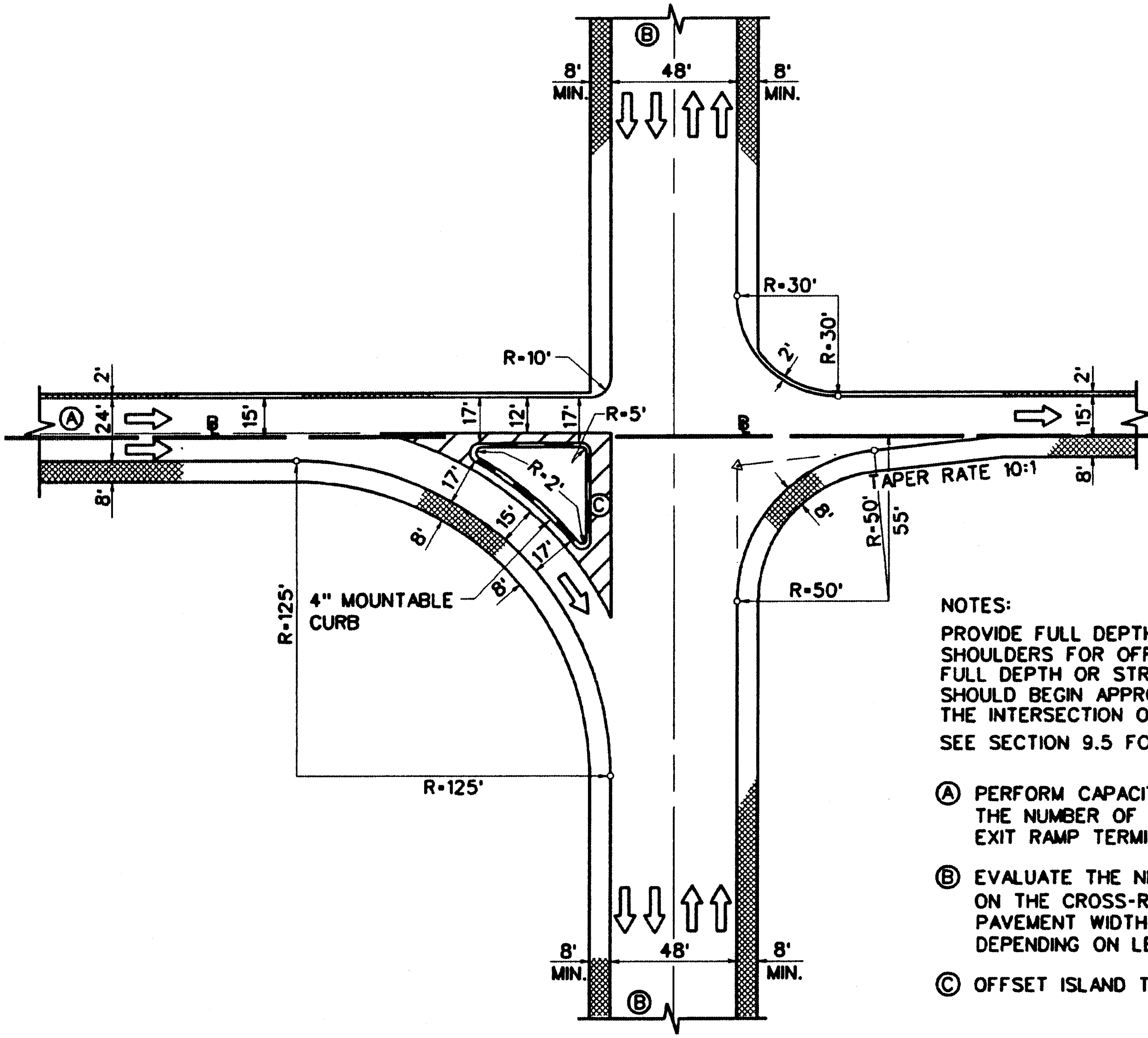
RAMP TERMINAL AT 2-LANE CROSSROAD (RURAL)
(WB-67 Design Vehicle)

Figure 10.8N



RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (RURAL)
(WB-67 Design Vehicle)

Figure 10.8O



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

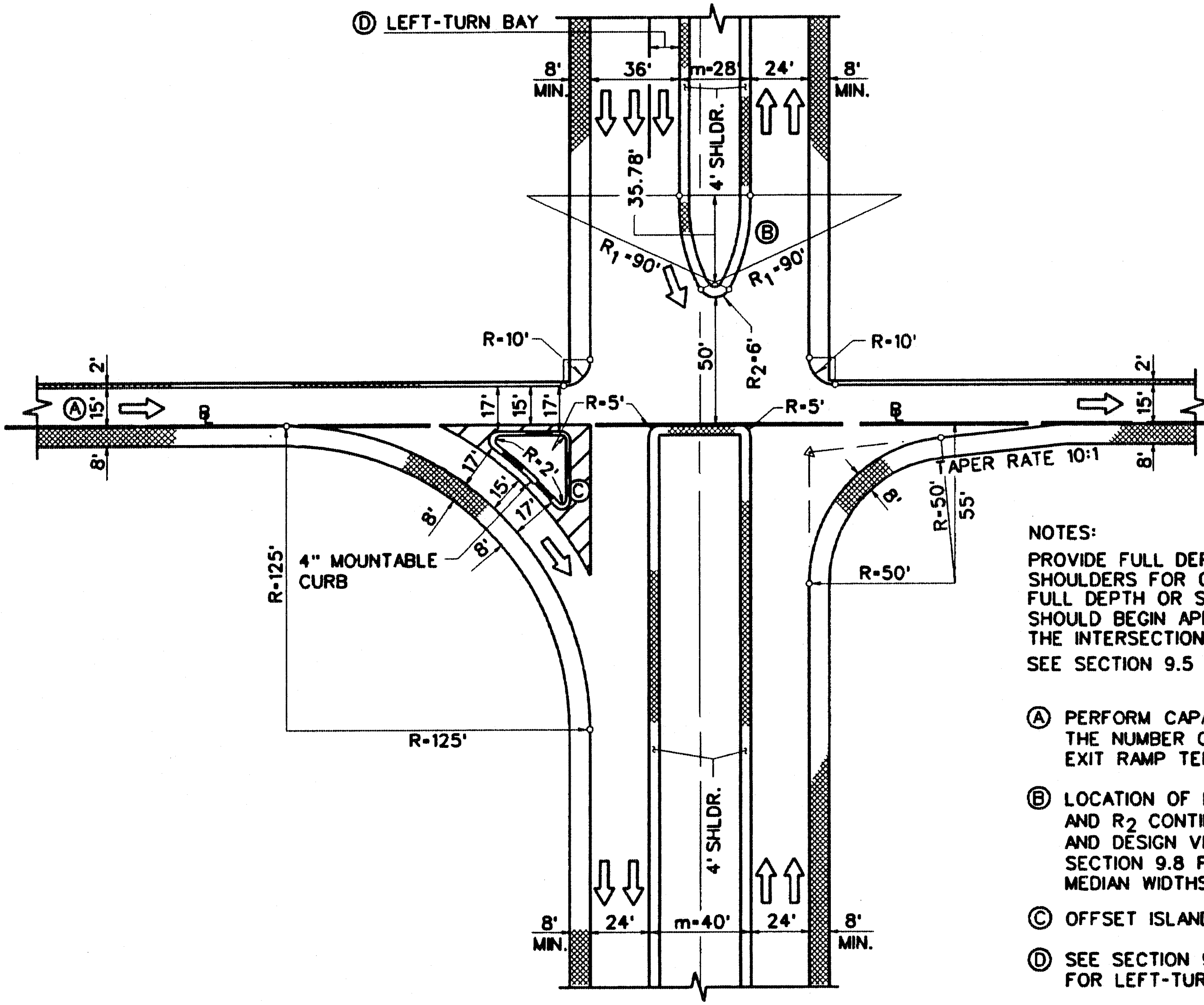
Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD.
PAVEMENT WIDTH MAY VARY FROM 48' TO 64' DEPENDING ON LEFT-TURN REQUIREMENTS.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (RURAL)
(WB-67 Design Vehicle)

Figure 10.8P



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

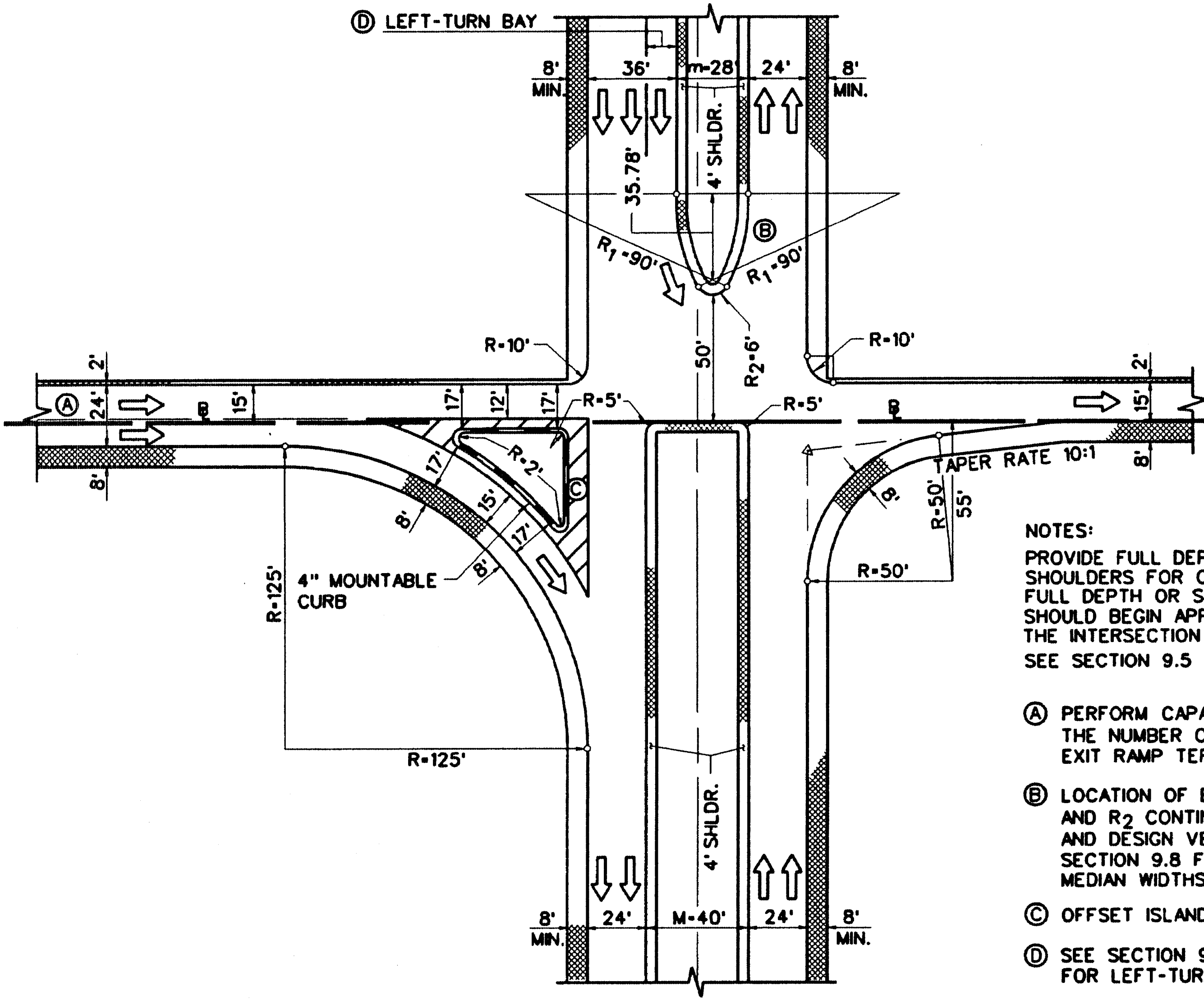
Ⓑ LOCATION OF BULLET NOSE AND RADIUS R_1 AND R_2 CONTINGENT UPON MEDIAN WIDTH AND DESIGN VEHICLE TURNING PATH. SEE SECTION 9.8 FOR DESIGN CRITERIA FOR MEDIAN WIDTHS OTHER THAN 40'.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

Ⓓ SEE SECTION 9.3 FOR DESIGN CRITERIA FOR LEFT-TURN LANES.

RAMP TERMINAL AT 4-LANE DIVIDED CROSSROAD (RURAL)
(WB-67 Design Vehicle)

Figure 10.8Q



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.
SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

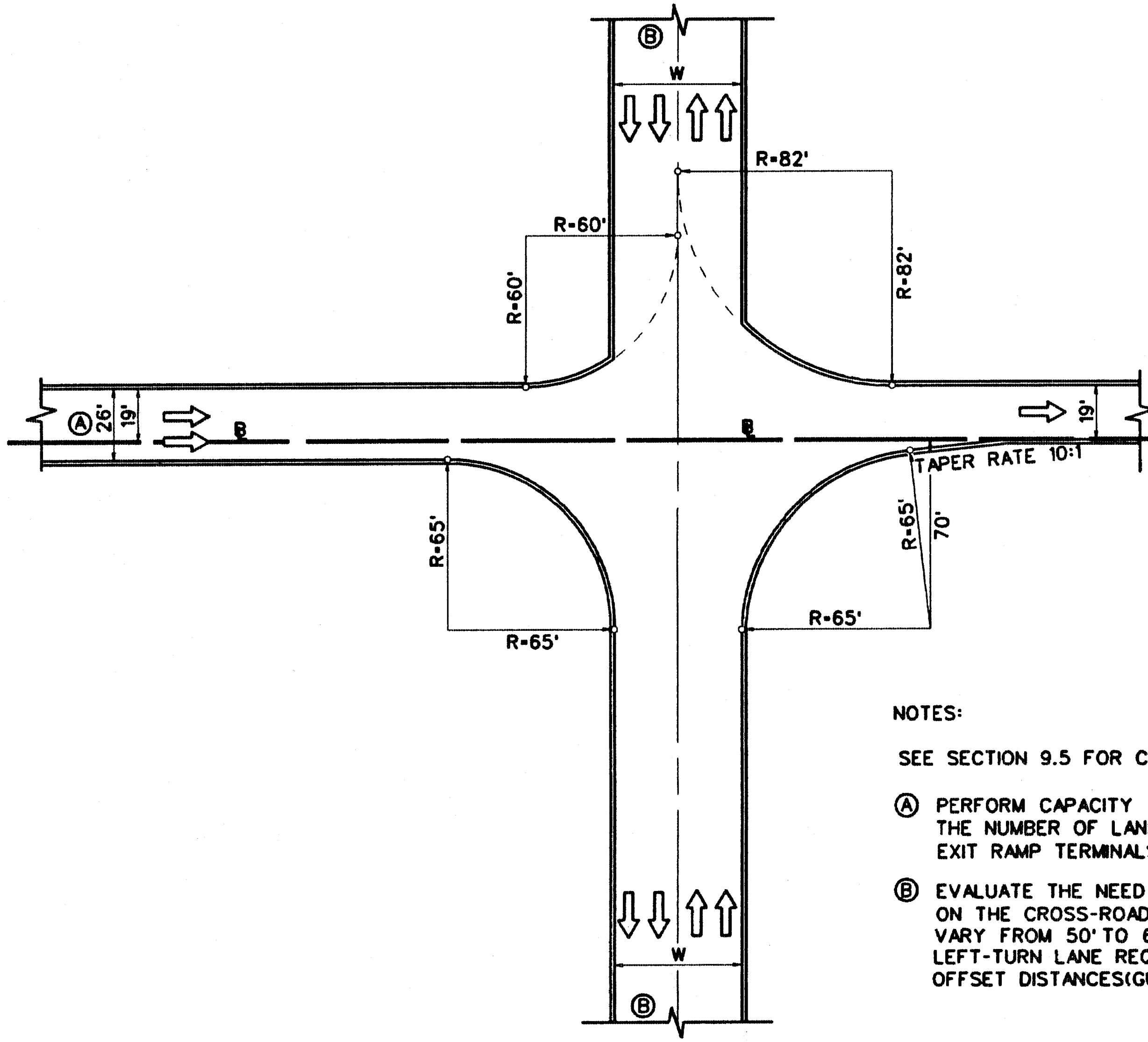
Ⓑ LOCATION OF BULLET NOSE AND RADIUS R_1 AND R_2 CONTINGENT UPON MEDIAN WIDTH AND DESIGN VEHICLE TURNING PATH. SEE SECTION 9.8 FOR DESIGN CRITERIA FOR MEDIAN WIDTHS OTHER THAN 40'.

Ⓒ OFFSET ISLAND TO BACK OF SHOULDER.

Ⓓ SEE SECTION 9.3 FOR DESIGN CRITERIA FOR LEFT-TURN LANES.

RAMP TERMINAL AT 4-LANE DIVIDED CROSSROAD (RURAL)
(WB-67 Design Vehicle)

Figure 10.8R



NOTES:

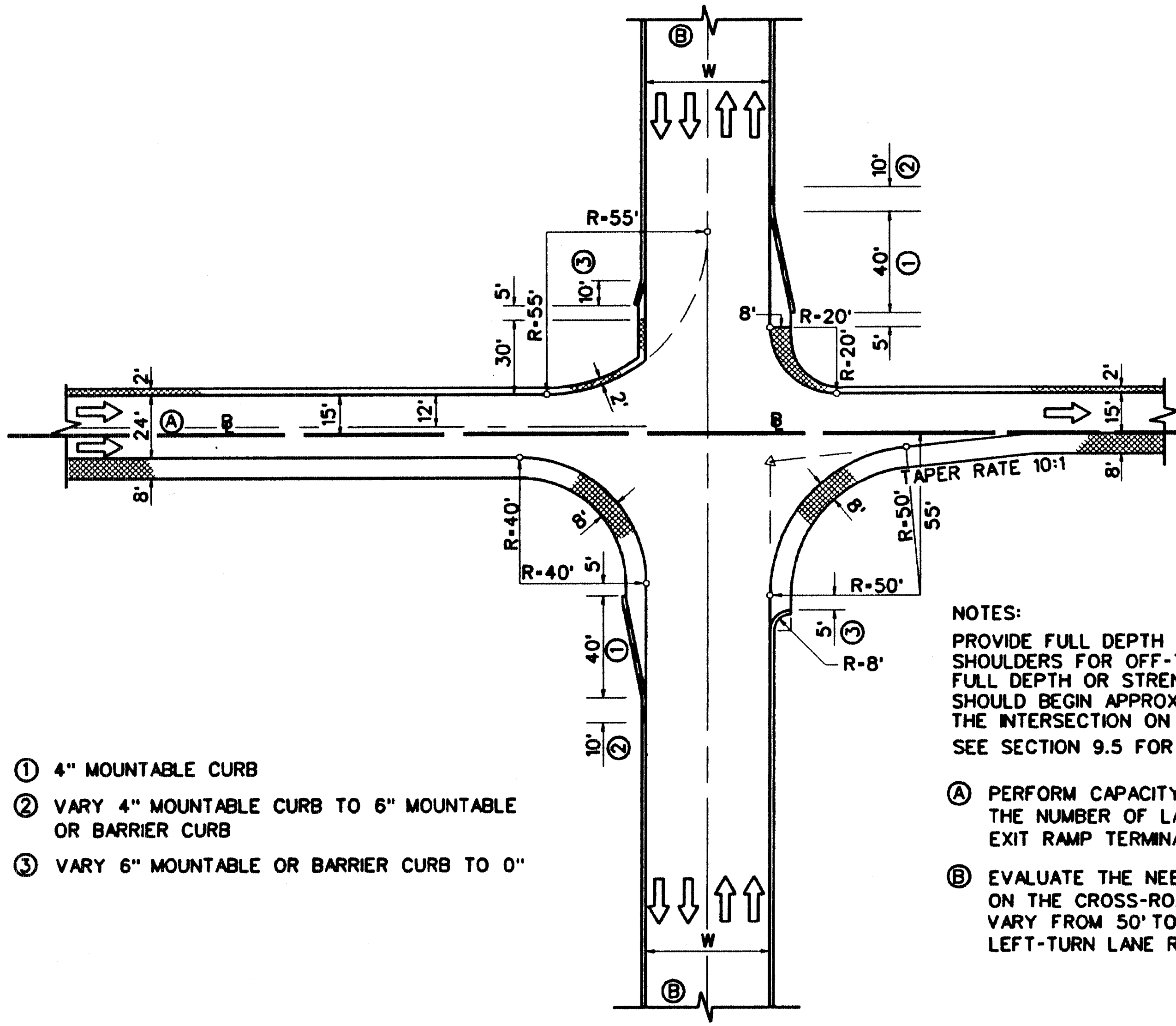
SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH "W" MAY VARY FROM 50' TO 68' DEPENDING ON LEFT-TURN LANE REQUIREMENTS AND CURB OFFSET DISTANCES(GUTTER WIDTH).

RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (URBAN)
(WB-67 Design Vehicle)

Figure 10.8S



① 4" MOUNTABLE CURB

② VARY 4" MOUNTABLE CURB TO 6" MOUNTABLE OR BARRIER CURB

③ VARY 6" MOUNTABLE OR BARRIER CURB TO 0"

NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE CROSS-ROAD. PAVEMENT WIDTH MAY VARY FROM 50' TO 68' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

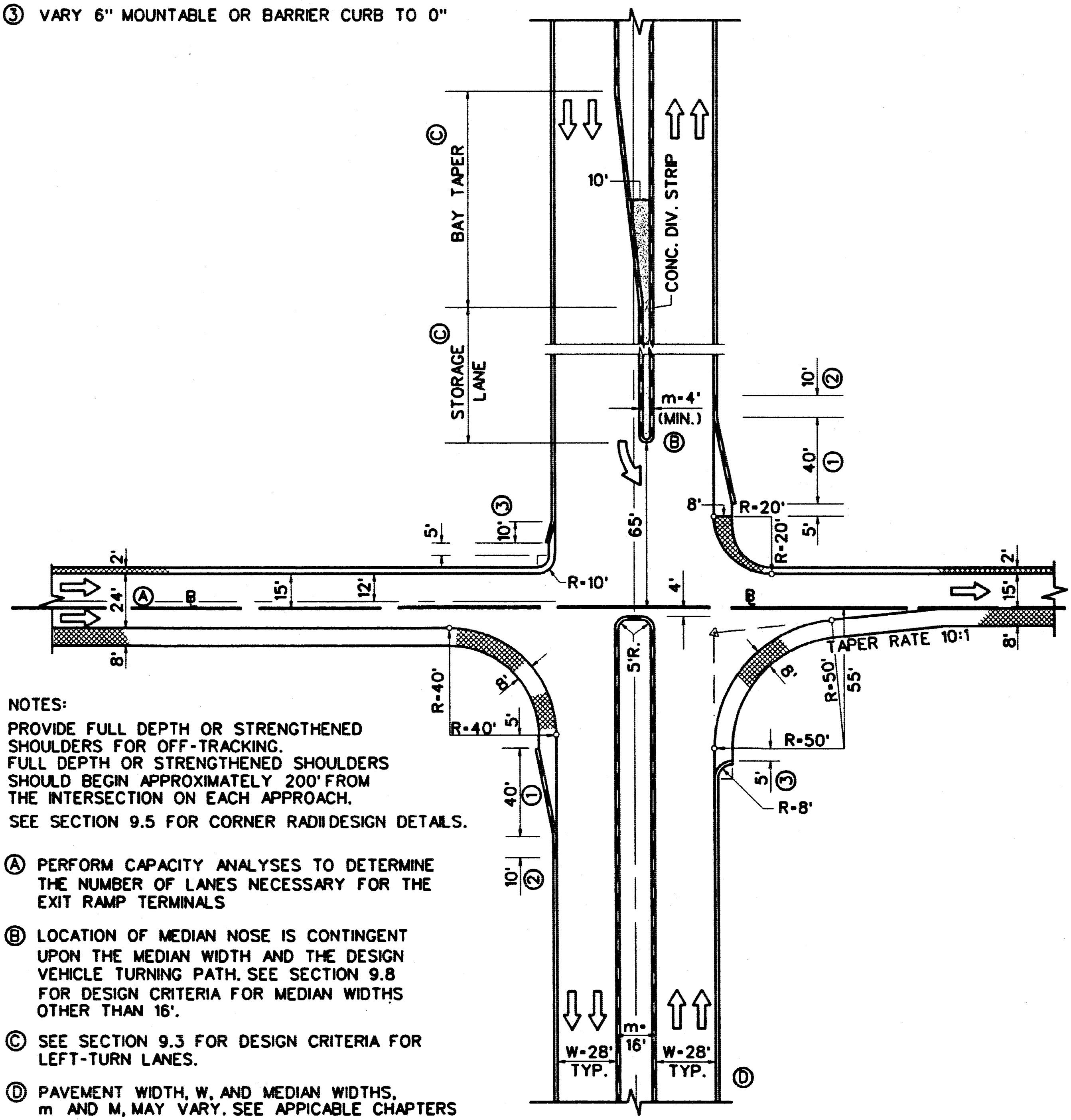
RAMP TERMINAL AT 4-LANE UNDIVIDED CROSSROAD (URBAN)
(WB-67 Design Vehicle)

Figure 10.8T

① 4" MOUNTABLE CURB

② VARY 4" MOUNTABLE CURB TO 6" MOUNTABLE OR BARRIER CURB

③ VARY 6" MOUNTABLE OR BARRIER CURB TO 0"



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.
FULL DEPTH OR STRENGTHENED SHOULDERS SHOULD BEGIN APPROXIMATELY 200' FROM THE INTERSECTION ON EACH APPROACH.

SEE SECTION 9.5 FOR CORNER RADII DESIGN DETAILS.

Ⓐ PERFORM CAPACITY ANALYSES TO DETERMINE THE NUMBER OF LANES NECESSARY FOR THE EXIT RAMP TERMINALS

Ⓑ LOCATION OF MEDIAN NOSE IS CONTINGENT UPON THE MEDIAN WIDTH AND THE DESIGN VEHICLE TURNING PATH. SEE SECTION 9.8 FOR DESIGN CRITERIA FOR MEDIAN WIDTHS OTHER THAN 16'.

Ⓒ SEE SECTION 9.3 FOR DESIGN CRITERIA FOR LEFT-TURN LANES.

Ⓓ PAVEMENT WIDTH, W, AND MEDIAN WIDTHS, m AND M, MAY VARY. SEE APPICABLE CHAPTERS FOR WIDTHS.

RAMP TERMINAL AT 4-LANE DIVIDED CROSSROAD (URBAN)
(WB-67 Design Vehicle)

Figure 10.8U

NOTES:

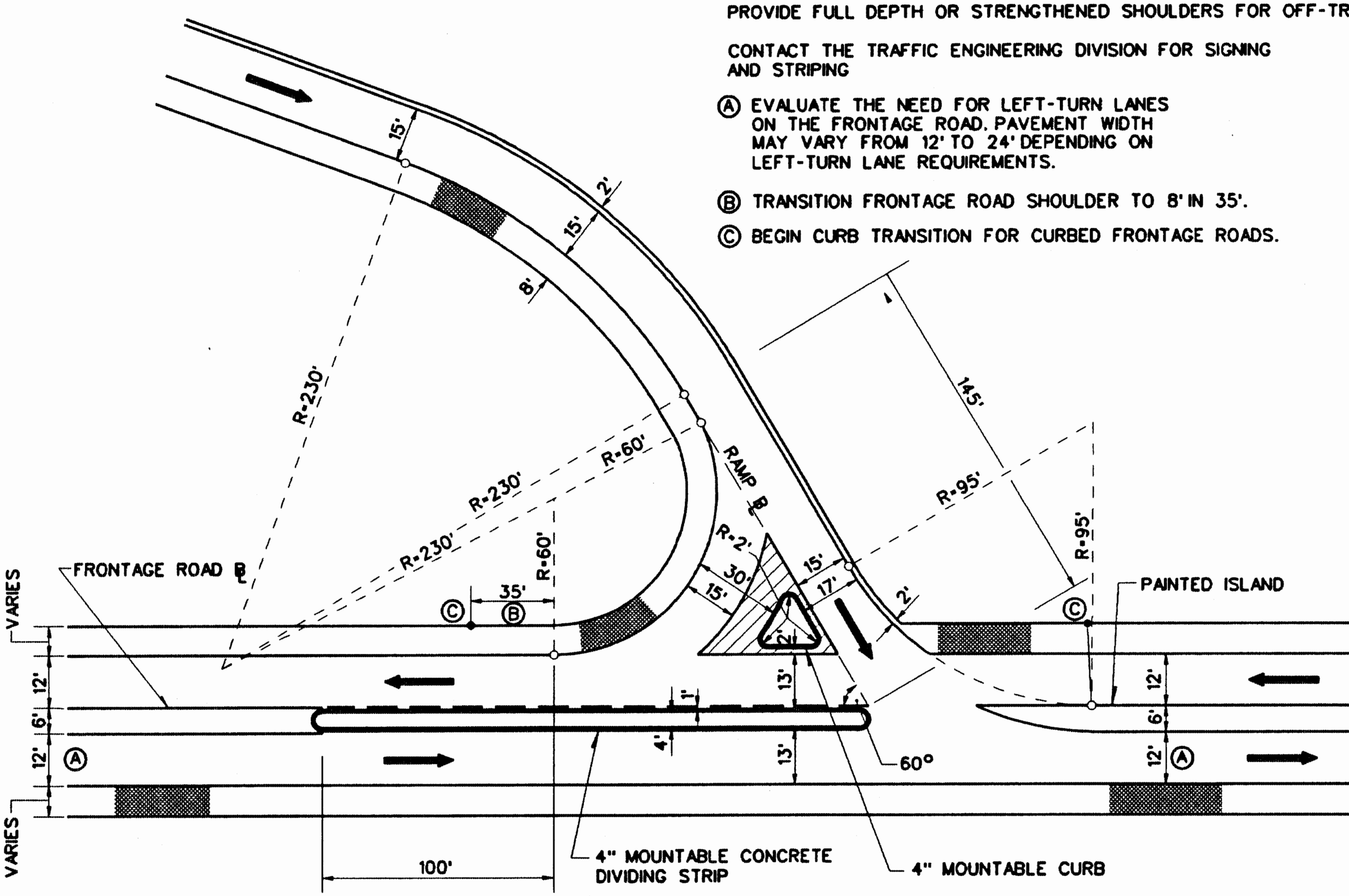
PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.

CONTACT THE TRAFFIC ENGINEERING DIVISION FOR SIGNING AND STRIPING

Ⓐ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE FRONTAGE ROAD. PAVEMENT WIDTH MAY VARY FROM 12' TO 24' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

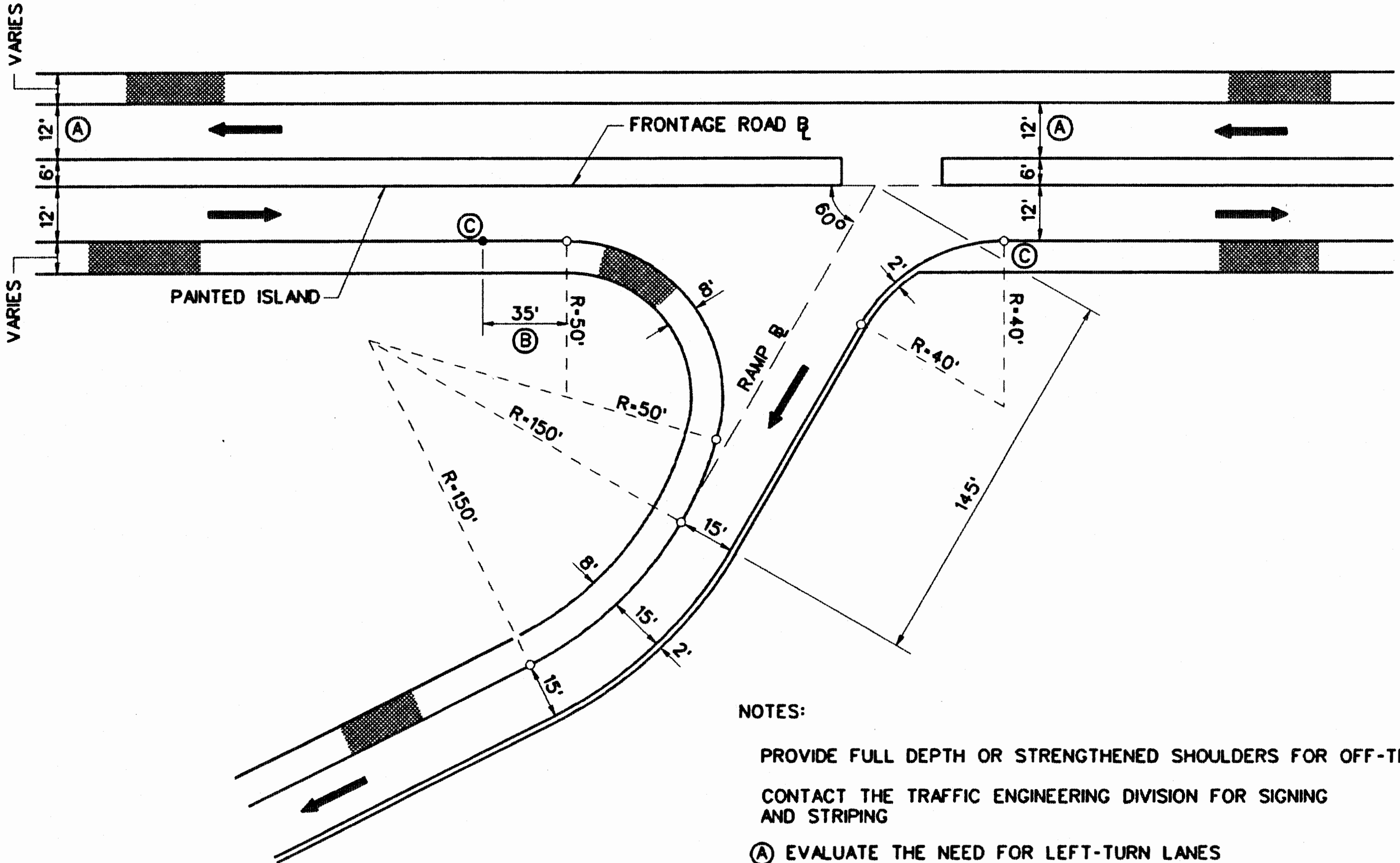
Ⓑ TRANSITION FRONTAGE ROAD SHOULDER TO 8' IN 35'.

Ⓒ BEGIN CURB TRANSITION FOR CURBED FRONTAGE ROADS.



BUTTONHOOK EXIT TERMINAL TO FRONTAGE ROAD
(WB-67 Design Vehicle)

Figure 10.8V



NOTES:

PROVIDE FULL DEPTH OR STRENGTHENED SHOULDERS FOR OFF-TRACKING.

CONTACT THE TRAFFIC ENGINEERING DIVISION FOR SIGNING AND STRIPING

Ⓐ EVALUATE THE NEED FOR LEFT-TURN LANES ON THE FRONTAGE ROAD. PAVEMENT WIDTH MAY VARY FROM 12' TO 24' DEPENDING ON LEFT-TURN LANE REQUIREMENTS.

Ⓑ TRANSITION FRONTAGE ROAD SHOULDER TO 8' IN 35'.

Ⓒ BEGIN CURB TRANSITION FOR CURBED FRONTAGE ROADS.

BUTTONHOOK ENTRANCE TERMINAL TO FRONTAGE ROAD
(WB-67 Design Vehicle)

Figure 10.8W

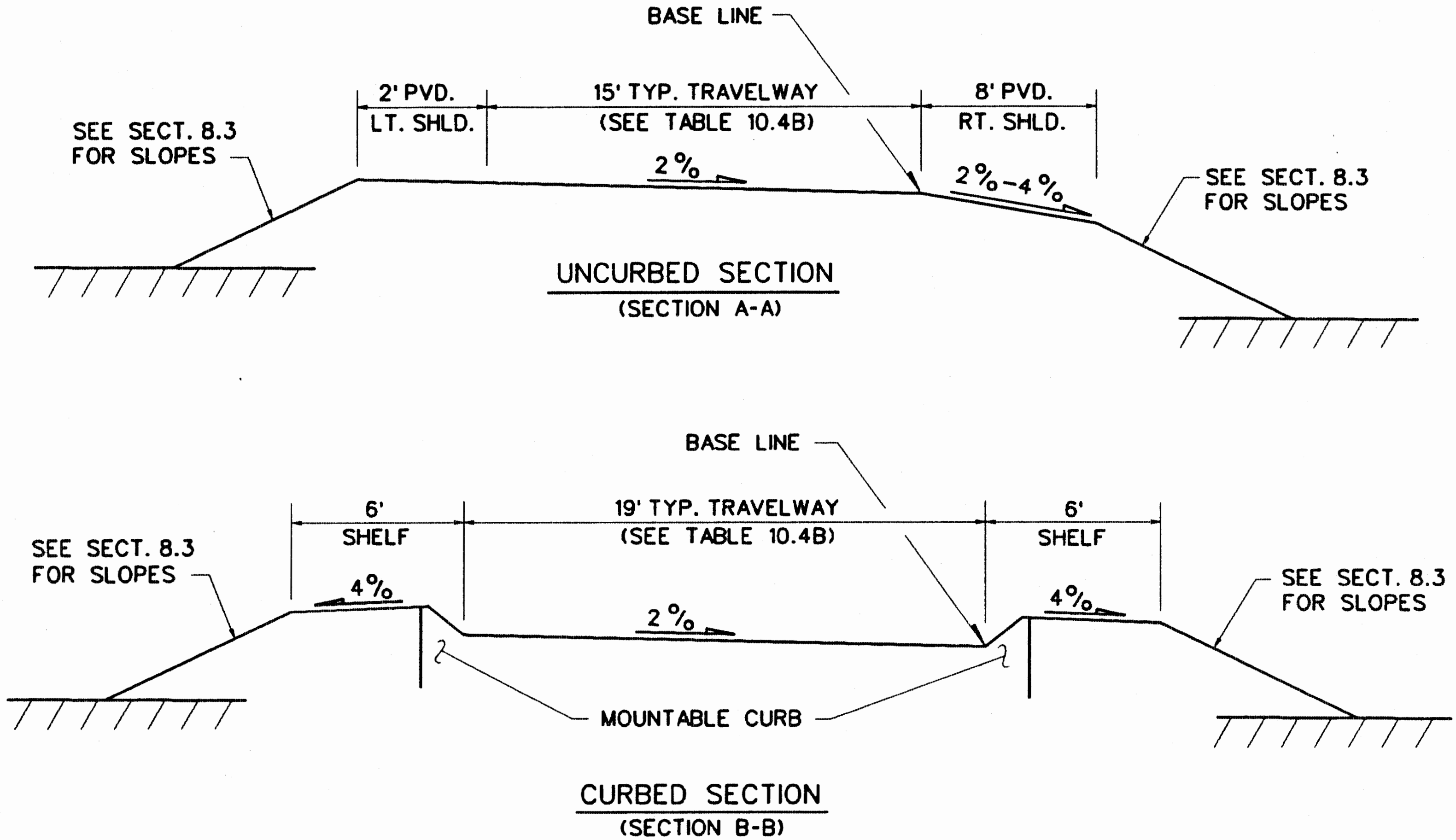
10.9 TYPICAL SECTION FIGURES

This Section presents the typical ramp section figures used by ODOT. Table 10.9A summarizes the figures in Section 10.9.

Table 10.9A

TYPICAL RAMP SECTION FIGURES

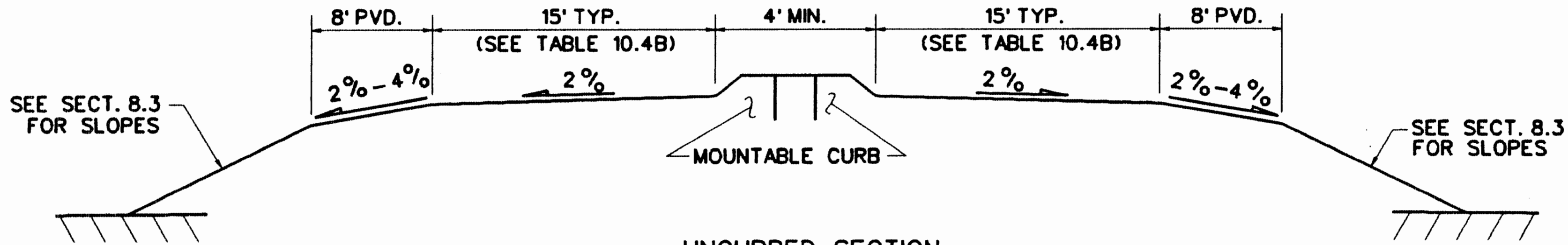
Figure Number	Figure Title
Figure 10.9A	Typical Ramp Sections (Tangent)
Figure 10.9B	Typical Ramp Sections (Adjacent On/Off Ramps)



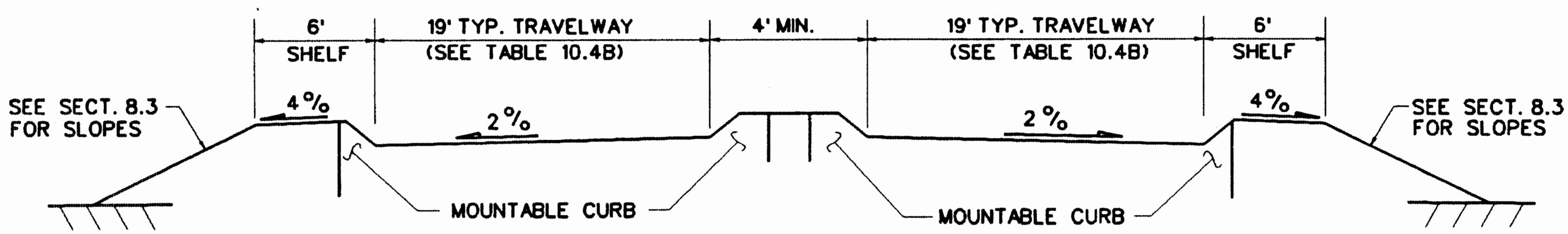
Note: Typical sections are for ramp proper away from either terminal. Section will vary at the at-grade intersection or freeway/ramp junction.

TYPICAL RAMP SECTIONS
(Tangent)

Figure 10.9A



UNCURBED SECTION
(SECTION C-C)



CURBED SECTION
(SECTION D-D)

Notes:

1. Typical sections are for ramp proper away from either terminal. Section will vary at the at-grade intersection or freeway/ramp junction.

2. Designer may determine a median barrier is warranted to separate on/off ramps.

TYPICAL RAMP SECTIONS
(Adjacent On/Off Ramps)

Figure 10.9B

10.10 REFERENCES

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

2. *Interstate System Accident Research Study II*, FHWA, 1975.

3. *Highway Capacity Manual*, Special Report 209, Transportation Research Board, 1985.

4. *A Policy on Geometric Design of Rural Highways*, AASHTO, 1965.

5. *Procedure for Analysis and Design of Weaving Sections -- A User's Guide*, Jack Leisch, October 1985.

REGISTERED PROFESSIONAL ENGINEER
MARK ALAN MURPHY
17346
OKLAHOMA

Chapter Eleven

Roadside Safety

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Chapter Eleven

ROADSIDE SAFETY

11.1 DEFINITIONS

1. Roadside Clear Zones. The distance beyond the edge of travel lane that should be clear of any non-traversable hazards or fixed objects.

2. 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 (e.g., rivers).

3. Parallel Slopes. Cut and fill slopes for which the toe runs approximately parallel to the flow of traffic.

4. Transverse Slopes. Cut and fill slopes for which the toe runs approximately perpendicular to the flow of traffic. Transverse slopes are typically formed by intersections between the mainline and driveways, median crossovers or side roads.

5. 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.

6. 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. For most embankment heights, if a fill slope is between 3:1 (inclusive) and 4:1 (exclusive), it is considered a non-recoverable parallel slope.

7. 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. A roadside barrier may be required on a critical parallel slope. Section 11.3 provides additional information.

8. Fore Slope. The side slope created by connecting the graded shoulder at the hinge point to the ditch bottom, downward and outward.

9. Back Slope. The side slope created by the connection of the ditch bottom, upward and generally outward, to the natural ground.

10. Fill Slope. The side slope created by connecting the graded shoulder at hinge point to the ditch bottom or natural ground line, downward and outward.

11. Cut Slope. The side slope created by going directly upward and outward from the shoulder edge to the natural ground.

12. Run/Rise Ratio or Slope Value. The relative steepness on a slope normal to the edge of the traveled way expressed as

a ratio of run to rise. For example, a 4:1 back slope rises 1 ft for each 4-ft run (lateral offset); likewise, a 4:1 fill slope falls 1 ft for each 4-ft run (lateral offset).

13. Toe of (Fill) Slope. The intersection of the fill slope or fore slope with the natural ground or ditch bottom, before the recommended rounding is applied.

14. Top of (Cut) Slope. The intersection of the back slope with the natural ground, before the recommended rounding is applied.

15. Barrier Warrant. A criterion that identifies an area of concern which should be shielded by a traffic barrier, if judged to be practical.

16. Roadside Barrier. A longitudinal barrier used to shield hazards located within an established clear zone. Roadside barriers include guardrail, concrete barriers, etc.

17. Median Barrier. A longitudinal barrier used to prevent an errant vehicle from crossing the portion of a divided highway separating the traveled ways for traffic in opposite directions.

18. Impact Attenuator (Crash Cushion). A traffic barrier used to safely shield fixed objects or other hazards from approximately head-on impacts by errant vehicles.

19. 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.

20. Shy Distance. Distance from the edge of the 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. This is considered a minimum of 2 ft on low-speed urban streets.

21. 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 vehicle's path at impact.

22. Experimental System. A roadside barrier, end terminal or crash cushion which has performed satisfactorily in full-scale crash tests, but has not yet been installed in sufficient quantity and/or has not been exposed to traffic long enough to evaluate its in-service performance adequately.

23. Operational System. A roadside barrier, end terminal or crash cushion which has performed satisfactorily in full-scale crash tests and has demonstrated satisfactory in-service performance.

24. Barrier Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Barrier curbs may range in height between 6 inches and 12 inches with a face steeper than 1 horizontal to 3 vertical.

25. Mountable Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Mountable curbs have a height of 6 inches or less with a face no steeper than 1 horizontal to 3 vertical.

11.2 ROADSIDE CLEAR ZONES

11.2.1 General Application

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.

Table 11.2A presents clear zone distances for design. When using the recommended clear zone distances, the designer should consider the following:

1. Project Scope of Work. The clear zone distances in Section 11.2 apply to all freeway projects and to new construction/ reconstruction projects on non-freeways. Chapter Thirteen presents the 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, 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. Figure 11.2A presents a schematic for the various possibilities.

5. Highway Design Adjustments. The recommended clear zone distance should be adjusted for the highway design speed, slope condition and traffic volumes, as indicated in Table 11.2A. See Section 11.2.4 for adjustments on horizontal curves.

11.2.2 Parallel Slopes (Uncurbed)

Table 11.2A presents the Department's criteria for clear zones on cut and fill slopes which run parallel to the highway. The following discusses the application of the table:

1. Recoverable Fill Slopes. For parallel fill slopes 4:1 and flatter (Figure 11.2A(a)), the recommended clear zone distance can be determined directly from Table 11.2A.

2. Non-Recoverable Fill Slopes. For parallel fill slopes steeper than 4:1 but 3:1 or flatter (Figure 11.2A(b)), the recommended clear zone includes a distance beyond the toe of the slope. Use the following procedure to determine the clear zone:

   a. The slope beyond the toe of the non-recoverable fill slope will probably be 6:1 or flatter. Determine the clear zone for a 6:1 or flatter slope from Table 11.2A for the applicable design speed and traffic volume.

   b. To determine the recommended distance beyond the toe, subtract the shoulder width (or the distance from the edge of the travel lane to the slope break) from the distance in Step #2a.

Table 11.2A

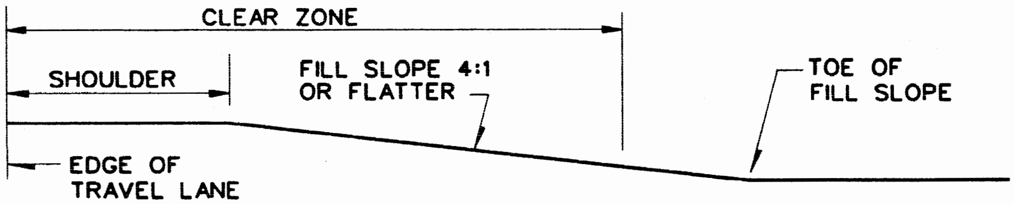
CLEAR ZONE DISTANCE (ft)
(New Construction/Reconstruction)

Design Speed	Design ADT	Fill Slopes			Cut Slopes		
		6:1 or flatter	5:1 to 4:1	3:1	3:1	4:1 to 5:1	6:1 or flatter
40 mph or less	Under 750 750-1500 1500-6000 Over 6000	7-10 10-12 12-14 14-16	7-10 12-14 14-16 16-18	See Procedure in Section 11.2.2	7-10 10-12 12-14 14-16	7-10 10-12 12-14 14-16	7-10 10-12 12-14 14-16
45-50 mph	Under 750 750-1500 1500-6000 Over 6000	10-12 14-16 16-18 20-22	12-14 16-20 20-26 24-28		8-10 10-12 12-14 14-16	8-10 12-14 14-16 18-20	10-12 14-16 16-18 20-22
55 mph	Under 750 750-1500 1500-6000 Over 6000	12-14 16-18 20-22 22-24	14-18 20-24 24-30 26-32*		8-10 10-12 14-16 16-18	10-12 14-16 16-18 20-22	10-12 16-18 20-22 22-24
60 mph	Under 750 750-1500 1500-6000 Over 6000	16-18 20-24 26-30 30-32*	20-24 26-32* 32-40* 36-44*		10-12 12-14 14-18 20-22	12-14 16-18 18-22 24-26	14-16 20-22 24-26 26-28
65-70 mph	Under 750 750-1500 1500-6000 Over 6000	18-20 24-26 28-32* 30-34*	20-26 28-36* 34-42* 38-46*		10-12 12-16 16-20 22-24	14-16 18-20 22-24 26-30	14-16 20-22 26-28 28-30

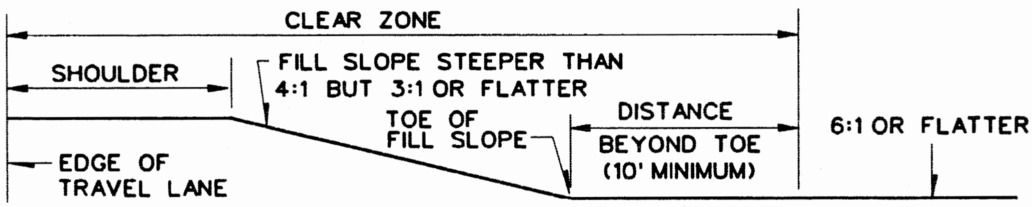
Source: (1)

Notes:
1. All distances are measured from the edge of the travel lane.
2. See discussion in Section 11.2.2 (Comment #2) for application of clear zone criteria on non-recoverable fill slopes.
3. See discussion in Section 11.2.3 for application of clear zone criteria across ditch sections.
4. These values apply to tangent sections of highway. For horizontal curve adjustments on horizontal curves, see Table 11.2B.
5. For clear zones, the "Design ADT" will be the total ADT on two-way roadways and the directional ADT on one-way roadways (e.g., ramps and one roadway of a divided highway).

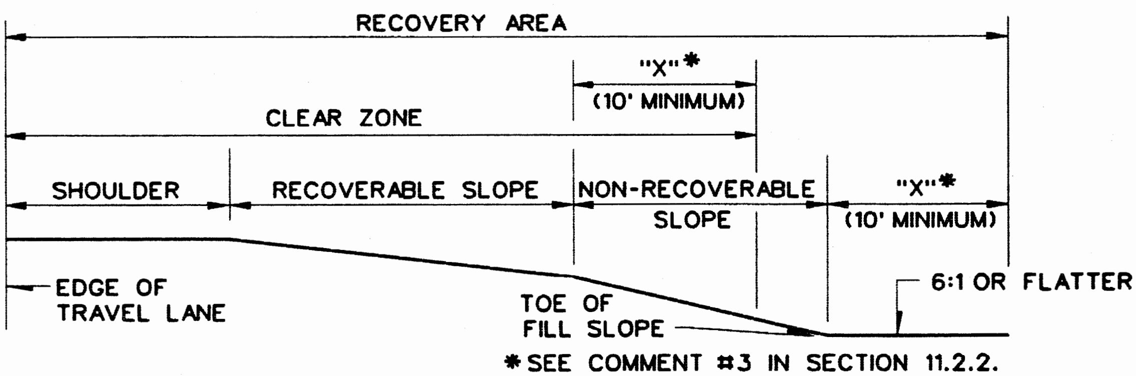
* Where a site specific investigation indicates a high probability of continuing accidents or such occurrences are indicated by accident history, the designer may provide clear zone distances greater than 30 ft as indicated. Clear zones may be limited to 30 ft for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance. When the clear zone distance is reduced to 30 ft as covered above, a Level 2 exception documentation must be developed. Also see note 6 on Table 11.2B.



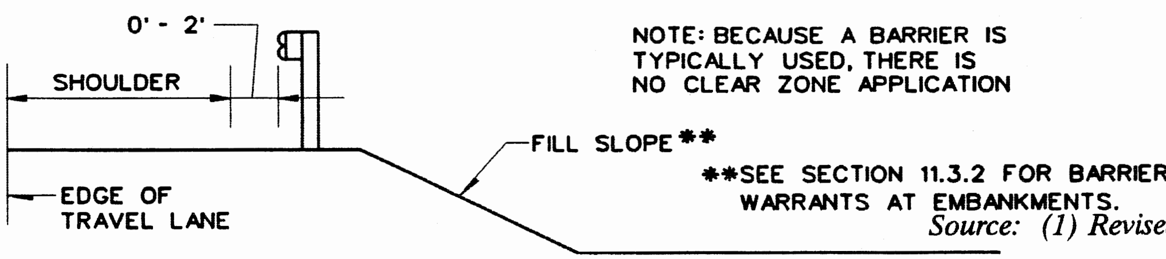
RECOVERABLE PARALLEL SLOPE (a)



NON-RECOVERABLE PARALLEL SLOPE (b)



BARN-ROOF PARALLEL SLOPE (c)



Source: (1) Revised

CRITICAL PARALLEL SLOPE (d)

CLEAR ZONE APPLICATION FOR FILL SLOPES
(Uncurbed)

Figure 11.2A

c. If the distance in Step #2b is greater than 10 ft, this distance will be the desirable clear zone beyond the toe; 10 ft will be the minimum. If the distance in Step #2b is less than 10 ft, the minimum clear distance will be 10 ft beyond the toe.

3. <u>Barn-Roof Fill Slope (Recoverable/Non-Recoverable</u>. Barn-roof fill slopes may be designed with a recoverable slope leading to a non-recoverable slope (Figure 11.2A(c)). The distance from the slope break to the clear zone (noted as "x" on Figure 11.2(A)(c)) should be applied as an addition outside the intersection of the non-recoverable slope and the 6:1 or flatter slope. This addition should be a minimum of 10 ft wide. The procedure for determining the clear zone is similar to that described for non-recoverable fill slopes in Comment #2; i.e., a clear area beyond the toe of slope (10-ft minimum) will be needed where the clear zone extends beyond the break between the recoverable and non-recoverable slopes.

4. <u>Barn-Roof Fill Slope (Recoverable/Recoverable</u>. 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. <u>Critical Fill Slope</u>. Fill slopes steeper than 3:1 are critical (Figure 11.2A(d)). These typically require a barrier and, therefore, there is no clear zone application.

The following examples illustrate how to determine the clear zone distance for various parallel slopes.

* * * * * * * * * * *

<u>Example 11-1 (Recoverable Fill Slope)</u>

Given: Fill Slope — 4:1
Design Speed — 60 mph
Design ADT — 7000

Problem: Determine the recommended clear zone distance.

Solution: From Table 11.2A, the clear zone distance should be 36′ - 44′. Note that this distance will apply regardless of the shoulder width. However, as indicated in a footnote to the table, the clear zone distance may be limited to 30′ based on specific site conditions to provide a more practical design.

<u>Example 11-2 (Barn-Roof Fill Slope)</u>

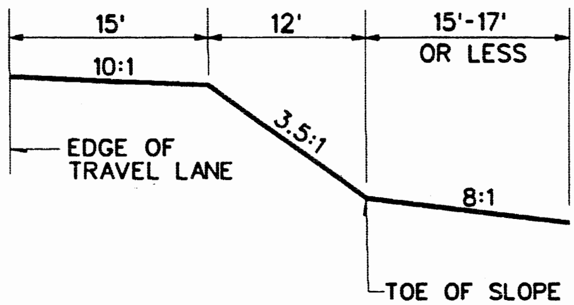
Given: Fill Slope — See Figure 11.2B
Design Speed — 60 mph
Design ADT — 7000

Problem: Determine the recommended clear zone distance.

Solution: The procedure in Section 11.2.2 for non-recoverable fill slopes is used as follows:

2a. From Table 11.2A, the clear zone for a fill slope 6:1 or flatter is 30′ - 32′.

2b. The recommended clear distance beyond the toe of the non-recoverable



Source: (1) Revised

NON-RECOVERABLE FILL SLOPE (Example 11-2)

Figure 11.2B

slope (3.5:1) is (30′ - 32′) minus 15′ yields (15′ - 17′).

2c. The desirable clear distance beyond the toe is 15′ - 17′; the minimum distance is 10′.

* * * * * * * * * *

11.2.3 Cut Slopes (Uncurbed)

Ditch sections, as illustrated in Figure 11.2C, are typically constructed in roadside cuts without curbs. 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 11.2A to determine the clear zone based on the ditch fore slope.

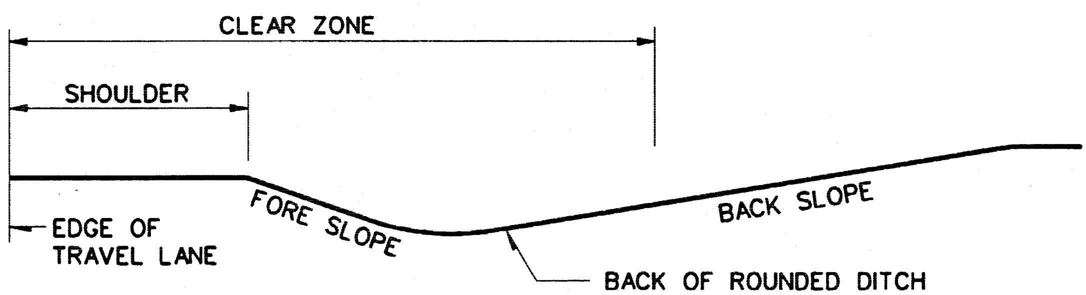
2. Check Location of the Toe of Back Slope. Based on the distance from Step #1, determine if the toe of the back slope is within the clear zone. The toe of back slope is defined as the point at which the ditch rounding ends and the (uniform) back slope begins. If the toe is at or beyond the clear zone, 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 below will apply.

3. Check for Roadside Hazards on Back Slope. If the toe of the back slope is within the clear zone distance from Step #1 above, a clear zone should be provided on the back slope. This clear zone will be a distance beyond the toe of back slope as follows:

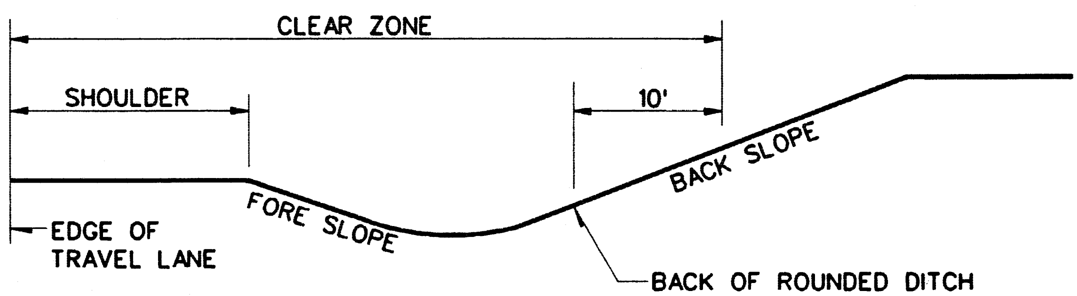
   a. When the back slope is 6:1 or flatter (Figure 11.2C(a)), treat the back slope as level and use the clear zone based on the fore slope rate to determine the clear zone limit on the back slope.

   b. When the back slope is steeper than 6:1 but 3:1 or flatter (Figure 11.2C(b)), assume the vehicle cannot make it up to the top of the back slope, if the slope is at least 10-ft wide. The initial 10 ft beyond the outside limit of ditch rounding or the distance in Step #3a, whichever is less, should be clear of roadside hazards. Any obstacles beyond this point would be considered outside of the clear zone.

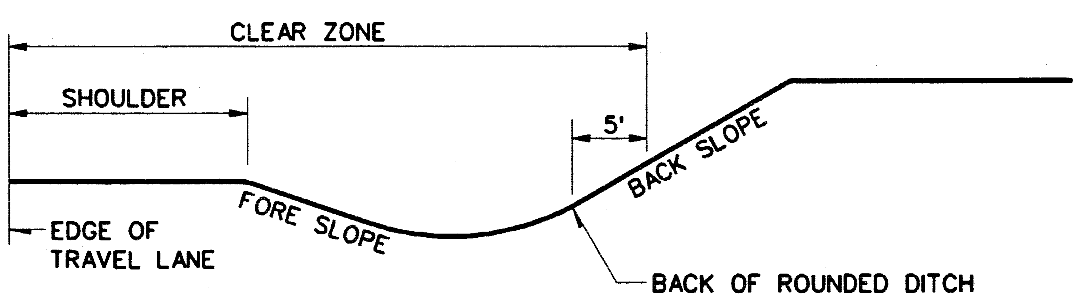
   c. When the back slope is steeper than 3:1 (Figure 11.2C(c)), the initial 5 ft beyond the outside limit of ditch rounding should be clear of roadside hazards.



BACK SLOPE 6:1 OR FLATTER (a)



BACK SLOPE STEEPER THAN 6:1 BUT 3:1 OR FLATTER (b)



BACK SLOPE STEEPER THAN 3:1 (c)

CLEAR ZONE APPLICATION FOR CUT SLOPES
(Uncurbed)

Figure 11.2C

d. In rock cuts with steep back slopes, no clear zone is required beyond the toe of back slope. However, the rock cut should be relatively smooth to minimize the hazards of vehicular impact.

4. Check Ditch Traversability. The designer should evaluate the traversability of the ditch cross section. See Section 11.3.7.1

* * * * * * * * * *

Example 11-3 (Ditch Section)

Given: ADT = 7000
V = 60 mph
Tangent Roadway
Fore Slope = 6:1
Ditch Width = 8′
Back Slope = 4:1
Toe of back slope is 25′ from edge of travel lane.
See Figure 11.2D

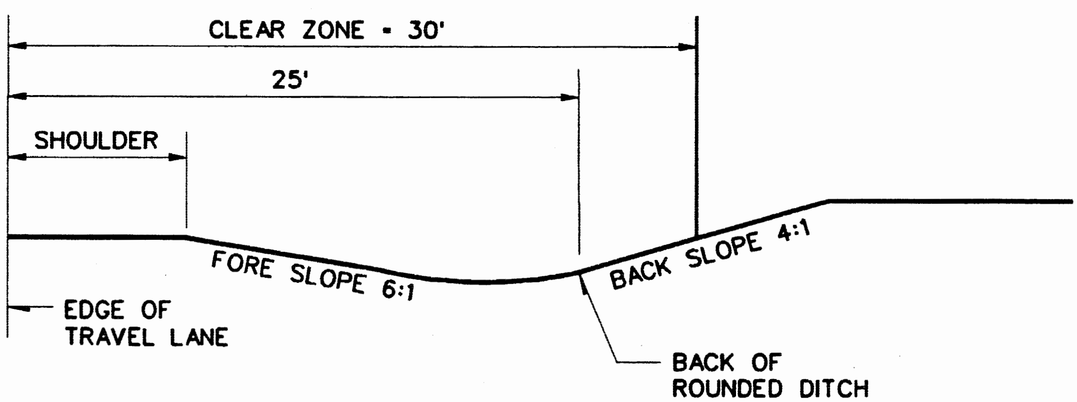
Problem: Determine the clear zone application across the ditch section.

Solution: Using the procedure in Section 11.2.3:

1. Check Fore Slope. Table 11.2A yields a clear zone of 30′ 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. Check for Roadside Hazards on Back Slope. With a 4:1 back slope, the criteria in Step #3b in the procedure will apply. Based on these criteria, the initial 5′ beyond the toe of back slope should be clear of roadside hazards. This provides a 30′ clear zone for the roadside.



CLEAR ZONE AT DITCH SECTION
(Example 11-3)

Figure 11.2D

4. Check Ditch Traversability. As discussed in Section 11.3, the ditch in this example is a traversable ditch and, therefore, is not a roadside hazard.

* * * * * * * * * * *

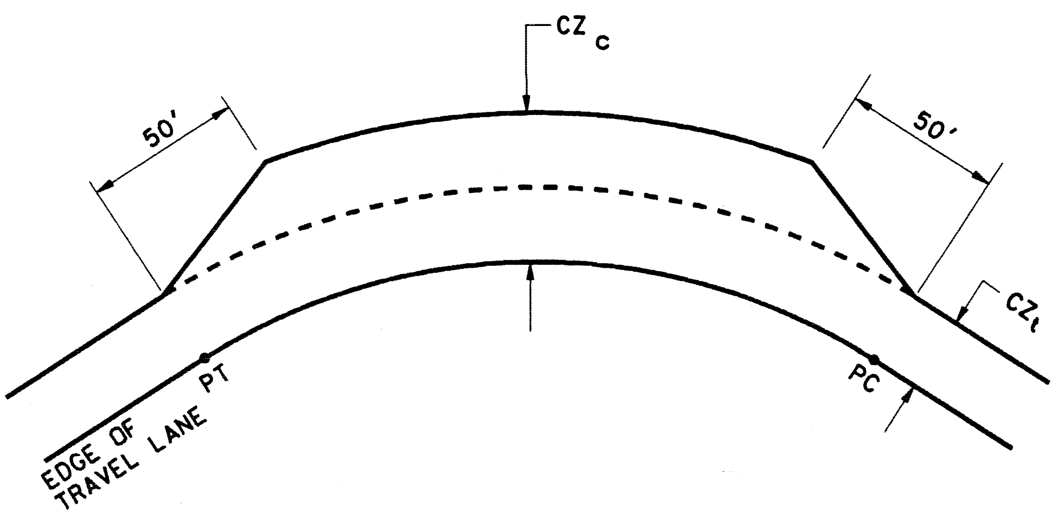
11.2.4 Horizontal Curves

On the outside of horizontal curves, run-off-the-road vehicles may travel a farther distance from the travel lane before regaining control of the vehicle. The designer may choose to modify the clear zone distance obtained from Table 11.2A for horizontal curvature. These modifications are normally considered where accident history indicates a need or where a specific site investigation shows a definitive accident potential.

Where adjustments will be applied, Figure 11.2E illustrates the application on a curve. Table 11.2B provides recommended factors for clear zones on horizontal curves.

11.2.5 Curbed Sections

Figure 11.2F illustrates the clear zone application where curbed sections are used. In urban areas, the minimum clear zone distance is 1.5 ft from the face of curb. However, if practical, a 5-ft to 10-ft clear zone distance should be provided, especially at intersections and driveway entrances, as measured from the face of curb.



CZ_t = clear zone on tangent section
CZ_c = clear zone on horizontal curve

HORIZONTAL CURVE ADJUSTMENTS

Figure 11.2E

Table 11.2B

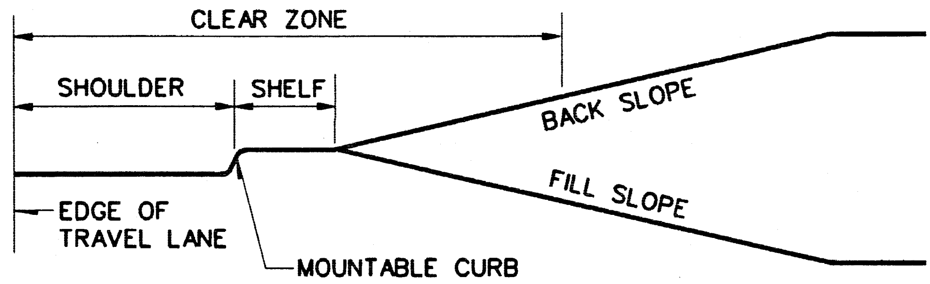
CLEAR ZONE ADJUSTMENT FACTORS FOR HORIZONTAL CURVES (K_{CZ})

DEGREE OF CURVE	DESIGN SPEED (mph)						
	40	45	50	55	60	65	70
2.0	1.08	1.10	1.12	1.15	1.19	1.22	1.27
2.5	1.10	1.12	1.15	1.19	1.23	1.28	1.33
3.0	1.11	1.15	1.18	1.23	1.28	1.33	1.40
3.5	1.13	1.17	1.22	1.26	1.32	1.39	1.46
4.0	1.15	1.19	1.25	1.30	1.37	2.44	
4.5	1.17	1.22	1.28	1.34	1.41	1.49	
5.0	1.19	1.24	1.31	1.37	1.46		
6.0	1.23	1.29	1.36	1.45	1.54		
7.0	1.26	1.34	1.42	1.52			
8.0	1.30	1.38	1.48				
9.0	1.34	1.43	1.53				
10.0	1.37	1.47					
15.0	1.54						

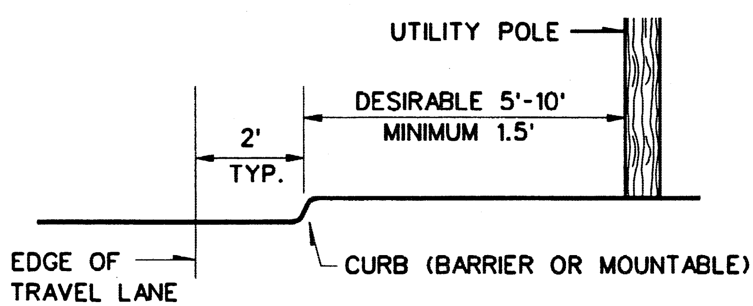
Source: (1)

Notes:
1. Adjustments apply to the outside of a horizontal curve.
2. Curves flatter than 2° do not require adjustments.
3. The applicable clear zone distance on a horizontal curve is calculated by: $CZ_c = (K_{cz})(CZ_t)$

   where: CZ_c = clear zone on a curve, ft
   K_{cz} = curve correction factor
   CZ_t = clear zone on a tangent section from Section 11.2.
4. For curves intermediate in the table, use a straight-line interpolation.
5. See Figure 11.2E for the application of CZ_c to the roadside around a curve.
6. Curve correction criteria may be modified or reduced as covered in note on Table 11.2A, but reduction or elimination of horizontal curve correction distances (down to 30′) must be documented as a Level 2 exception.



RURAL CURBED SECTION (a)



URBAN CURBED SECTION (b)

Note: Where mountable curb is used in urban areas, the clear zone will desirably be determined as indicated in (a) for a rural section.

CLEAR ZONE APPLICATION
(Curbed Section)

Figure 11.2F

11.2.6 Rural Collectors and Local Roads

On non-State highways functionally classified as rural collectors and rural local roads, where the design speed is 40 mph or less, the minimum clear zone will be 10 ft, regardless of the side slopes or traffic volumes. See *County Roads Design Guidelines Manual* for additional information.

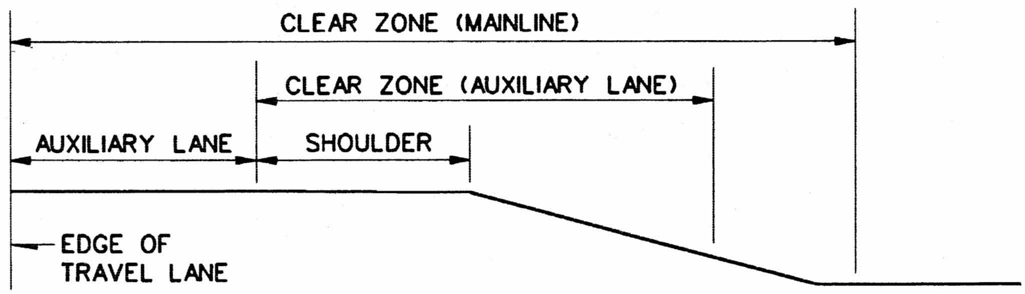
11.2.7 Auxiliary Lanes

As discussed in Section 8.1, auxiliary lanes are defined as any lanes beyond the basic through travel lanes which are intended for use by vehicular traffic for specific functions. This includes turn lanes at intersections, truck-climbing lanes, weaving lanes, etc. The clear zone for auxiliary lanes will be determined as follows:

1. Turn Lanes at Intersections. Where the intersection is uncurbed, clear zones will be determined based on the design speed and traffic volumes associated with the through travel lanes; i.e., the presence of the turn lane is ignored when determining clear zones. Where the intersection is curbed, the criteria in Section 11.2.5 will apply; i.e., the minimum clear zone is 1.5 ft from the face of curb, and the desirable clear zone is 5 ft to 10 ft from the edge of the turn lane.

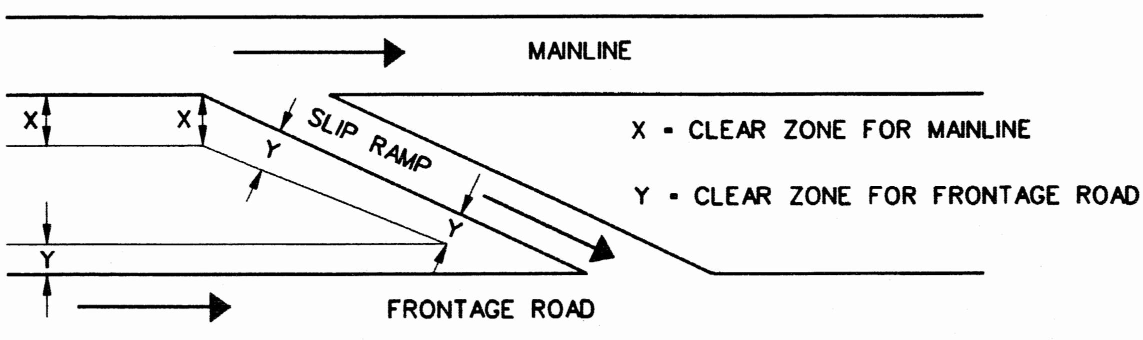
2. Auxiliary Lanes Adjacent to Mainline. Figure 11.2G(a) illustrates the clear zone application for climbing lanes, acceleration/deceleration lanes, weaving lanes, etc. Two independent clear zone determinations are necessary. First, the designer calculates the clear zone from the edge of the through traveled way based on the total traffic volume, including the auxiliary lane volume. Second, the designer calculates the clear zone from the edge of the auxiliary lane based on the traffic volume in the auxiliary lane. The clear zone distance which extends the farthest will apply.

3. Variable Design Speeds. Figure 11.2G(b) illustrates a roadway with a variable design speed — a slip ramp from a mainline to a one-way frontage road. As illustrated in the figure, variable clear zones will apply based on the variable design speeds.



Note: Use larger of clear zone measured from edge of travel lane or from edge of auxiliary lane. See Section 11.2.7. This applies to higher volume, higher speed auxiliary lanes (e.g., truck-climbing lanes, acceleration lanes). For turn lanes at intersections, see Section 11.2.7.

AUXILIARY LANES ADJACENT TO MAINLINE (a)



SLIP RAMPS (b)

CLEAR ZONE APPLICATION
(Auxiliary Lanes)

Figure 11.2G

11.3 ROADSIDE BARRIER WARRANTS

11.3.1 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 include:

1. eliminate the hazard (flatten embankment, remove rock outcroppings, etc.);

2. relocate the hazard;

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 travelway, nature of the hazard, costs for remedial action and accident experience. Where used, the designer should ensure that any roadside barrier installations are considered early in the project design.

As an alternative to a subjective evaluation, the designer may elect to use an approved cost-effective analysis for roadside safety applications. One example is ROADSIDE, which is presented in Appendix A of Reference (1).

11.3.2 Embankments

The severity of the roadside embankment depends upon the rate of fill slope and the height of fill. For all highways, Figure 11.3A should be used to determine if a barrier is warranted. For low embankment heights, the criteria allow fill slopes steeper than 3:1 to remain unshielded. A barrier is not required for areas outside of the shaded region, unless there are roadside hazards within the clear zone as calculated from Section 11.2.

11.3.3 Roadside Hazards

Section 11.2 presents the recommended clear zone distances for various highway conditions. These distances should be free of any fixed objectives and non-traversable hazards based on the relative severity between impacting guardrail and impacting the hazard. A few examples of roadside hazards include:

1. non-breakaway sign supports, non-breakaway luminaire supports, traffic signal poles and railroad signal poles;

2. concrete footings, etc., extending more than 4 inches above the ground;

3. bridge piers and abutments at underpasses, bridge parapet ends and pedestrian rail ends;

4. retaining walls and culvert headwalls;

5. trees with diameter greater than 4 inches (at maturity);

6. rough rock cuts;

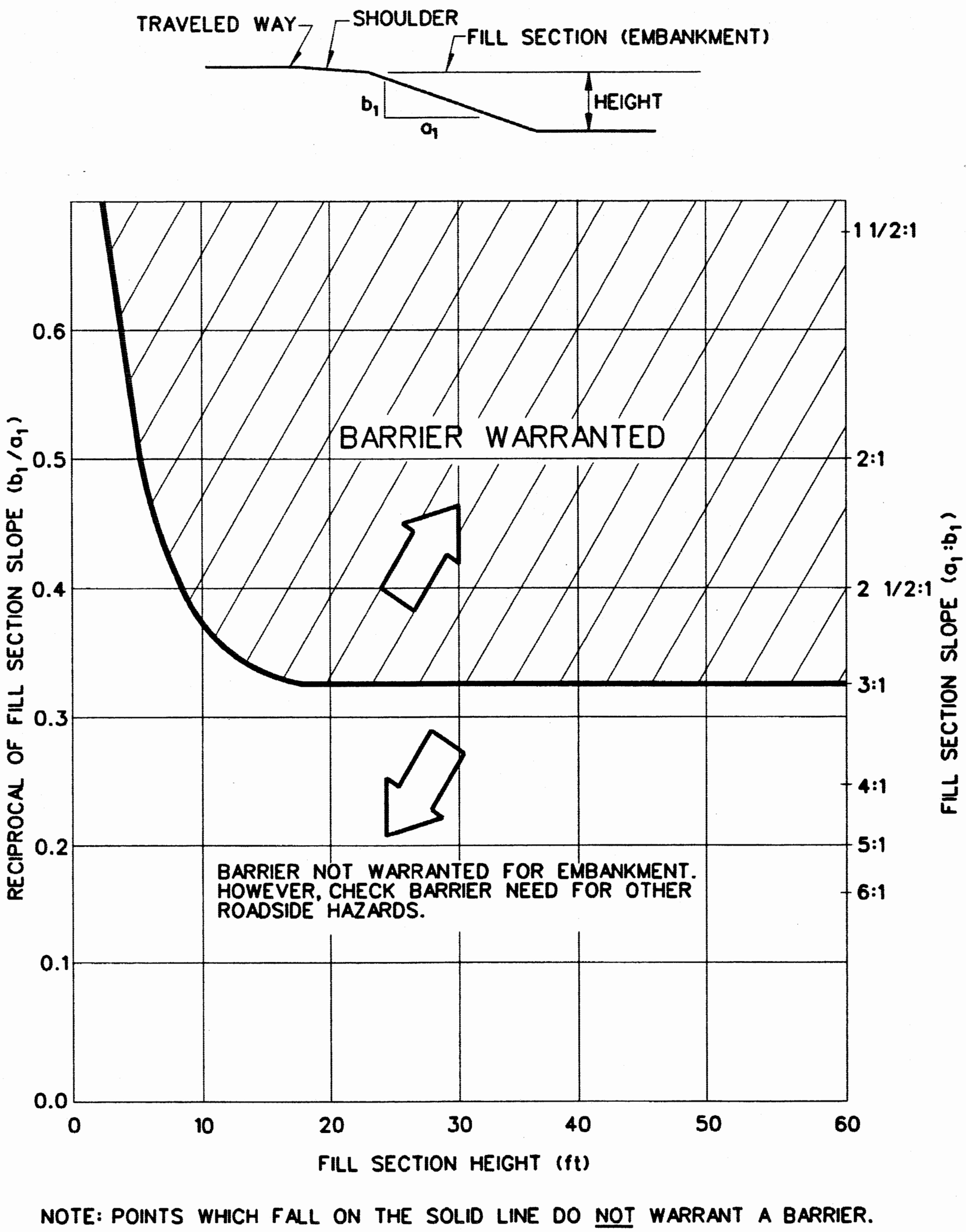
7. large boulders;

8. mailboxes;

9. streams or permanent bodies of water;

10. non-traversable ditches; and

11. utility poles or towers.



Source: (1)

BARRIER WARRANTS FOR EMBANKMENTS

Figure 11.3A

Once the designer has concluded that a barrier is warranted, the first attempt should be to remove or relocate the hazard or to make the hazard breakaway. If these are 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 edge of the clear zone.

11.3.4 Bridge Rails/Parapets

11.3.4.1 Approaches

Barrier protection is normally warranted on all approach ends to bridge rails or parapets, and it is normally warranted on the trailing ends of two-way roadways. No roadside barrier is needed on the trailing end of a one-way roadway, unless a barrier is warranted for other reasons (e.g., fill slopes steeper than 3:1). Figure 11.3B presents the criteria for barrier warrants for bridge rails or parapets.

11.3.4.2 Pedestrian Rail

If a sidewalk is placed on a bridge, it may be warranted to provide the typical 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. Principal Arterials. A separate pedestrian rail is required on all bridges with sidewalks on all high-speed principal arterials with partial control of access.

2. Other Facilities. On all other facilities with sidewalks on bridges, the need for a separate pedestrian rail will be considered on a case-by-case basis. The following factors will be evaluated:

   a. design speed;

   b. pedestrian volumes;

   c. 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 Bridge Division will be responsible for the final decision on when to use a pedestrian rail in combination with a typical bridge rail.

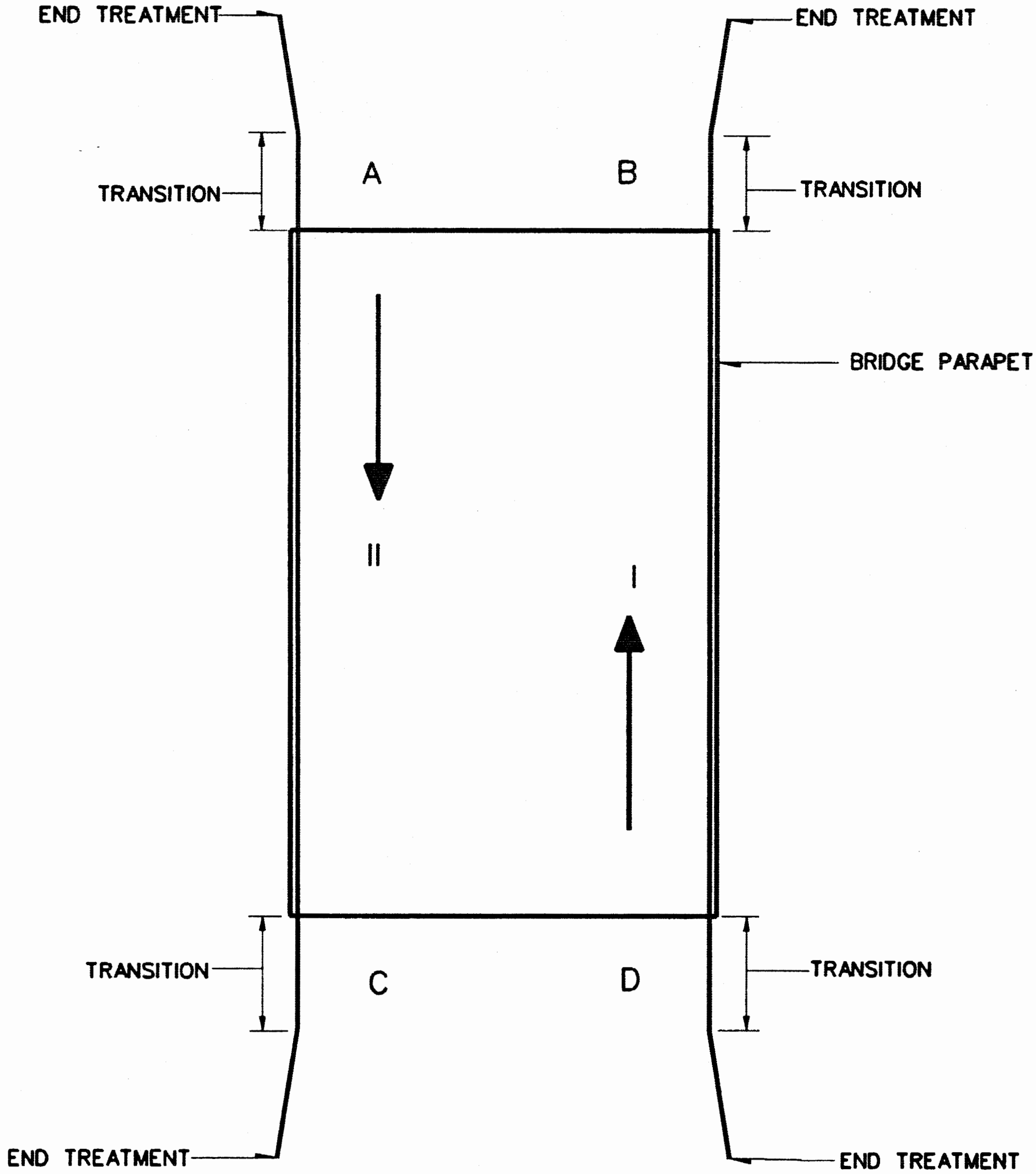
11.3.5 Transverse Slopes

Where the highway mainline intersects a driveway, side road or median crossing, a slope transverse to the mainline will be present. See Figure 11.3C. In general, transverse slopes should be as flat as practical. The following will apply:

1. Multilane Divided Highways. On all freeways and other multilane divided highways with $V \geq 50$ mph and high traffic volumes, the transverse slopes should be 10:1 or flatter.

2. Other Facilities. The transverse slopes should be 6:1 or flatter. However, on urban facilities ($V \leq 45$ mph) and on rural facilities ($V \leq 40$ mph), transverse slopes steeper than 6:1 may be considered.

If these criteria cannot be met practically, a roadside barrier should be considered. The decision to use a barrier will be made on a

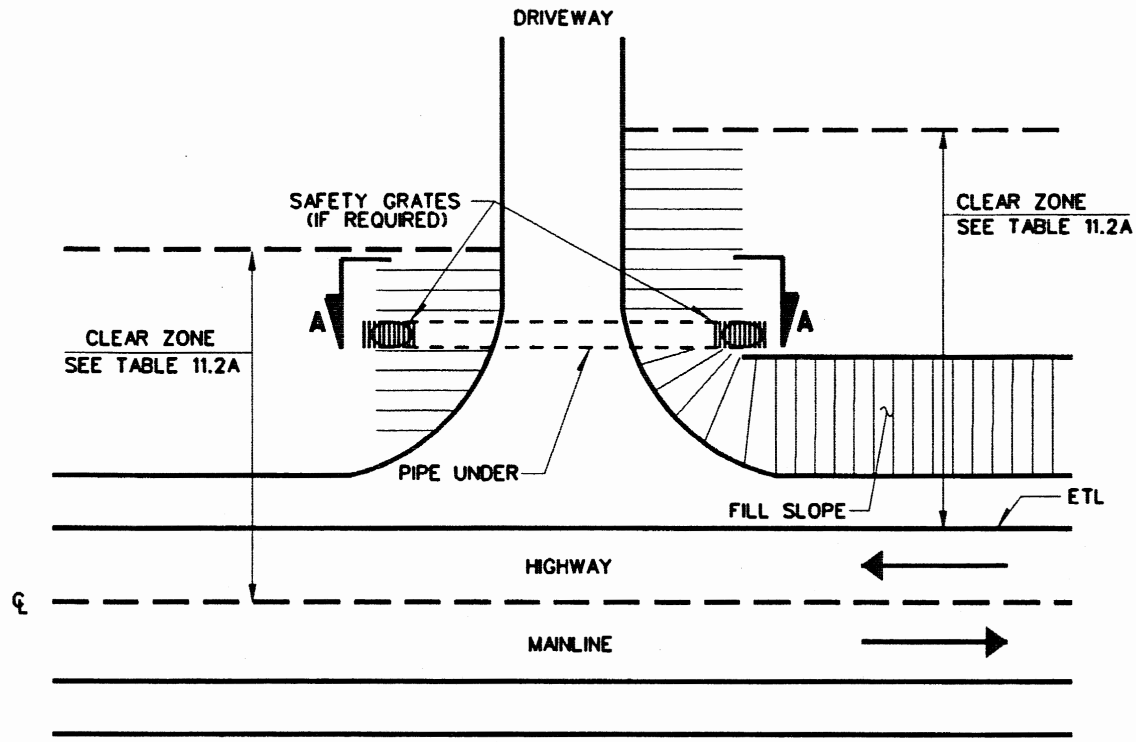


TRAFFIC DIRECTION	APPROACH BARRIER REQUIRED AT
I and II	A B C D
II Only	A B
I Only	C D

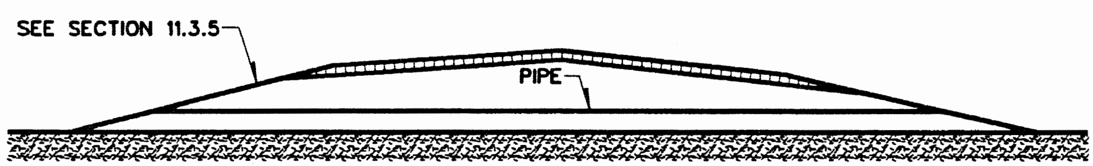
Source: (2)

BARRIER WARRANTS AT BRIDGES

Figure 11.3B



NOTE: ON A ONE-WAY FACILITY, THE SAFETY GRATE ON THE DOWNSTREAM SIDE OF THE DRIVEWAY IS NOT REQUIRED.



SECTION A-A

TRANSVERSE SLOPES
ON A TWO-LANE, TWO-WAY ROADWAY

Figure 11.3C

case-by-case basis considering costs, traffic volumes, severity of the proposed transverse slope and other relevant factors.

11.3.6 Rock Cuts

As indicated in Section 11.3.3, rough rock cuts located within the clear zone may be considered a roadside hazard. The following will apply to their treatment:

1. Hazard Identification. There is no precise method to determine whether or not a rock cut is sufficiently "ragged" to be considered a roadside hazard. This will be a judgment decision based on a case-by-case evaluation.

2. Debris. A roadside hazard may be identified based on known or potential occurrences of rock debris encroaching onto the roadway.

3. Barrier Warrant. If the rock cut or rock debris is within the clear zone, a barrier may be warranted.

4. Barrier Type. Where a barrier is used, a full-section concrete median barrier will typically be used up to a suggested maximum offset of 15 ft from the edge of travel lane.

11.3.7 Roadside Drainage Features

Effective drainage is one of the most critical elements in the design of a highway or street. However, drainage features should be designed and constructed considering their consequences on run-off-the-road vehicles. Ditches, curbs, culverts and drop inlets are common drainage system elements that should be designed, constructed and maintained considering both hydraulic efficiency and roadside safety.

In general, the following options, listed in order of preference, are applicable to all drainage features:

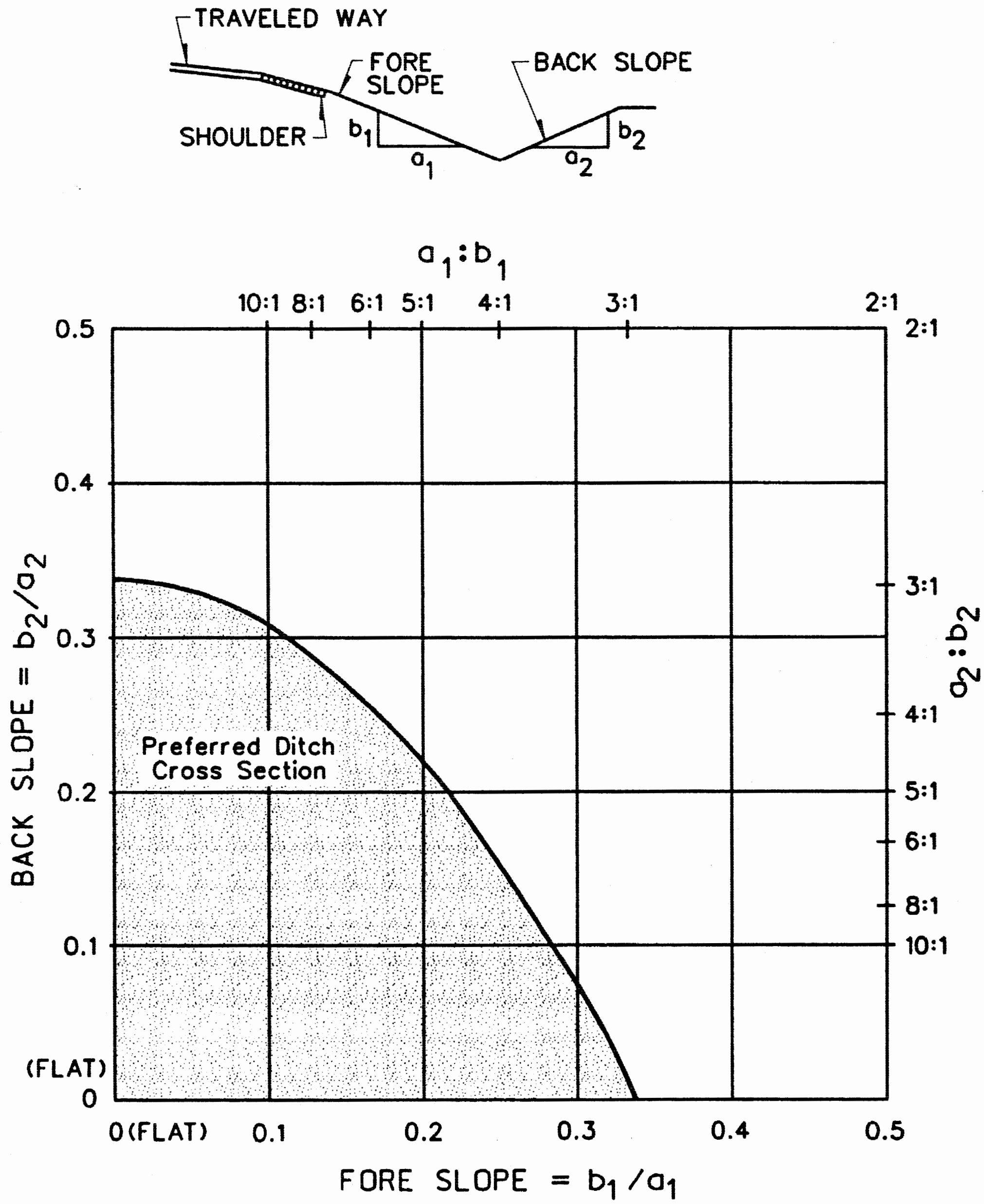
1. Design or modify drainage structures so that they are traversable or present a minimal hazard to an errant vehicle.

2. If a major drainage feature cannot effectively be redesigned or relocated, shielding by a traffic barrier should be considered if the feature is in a vulnerable location.

11.3.7.1 Roadside Ditches

Figure 11.3D and E present preferred fore slope and back slope combinations for basic ditch configurations. Cross sections which fall in the shaded region of each of the figures are considered traversable. Ditch sections which fall outside the shaded region are considered less desirable and their use should be limited where high-angle encroachments can be expected, such as on the outside of relatively sharp curves.

The designer should recognize that the traversability criteria in Figures 11.3D and E are predicated on impact speeds of 60 mph at a 25° angle of impact. Although no specific criteria are available, steeper fore slopes and/or back slopes or narrower ditch widths would be safely traversable at lower design speeds. Chapters Eight and Twelve present ODOT's criteria for the dimensions of roadside ditches based on functional classification and design speed. In general, these ditch sections meet the traversability criteria in Figures 11.3D and E for the higher speed facilities; the ditch section criteria for the lower speed facilities are judged to be reasonable for those highway types.

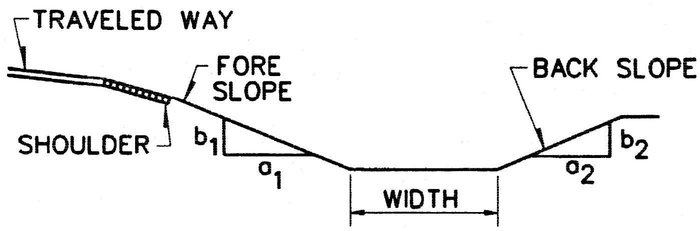


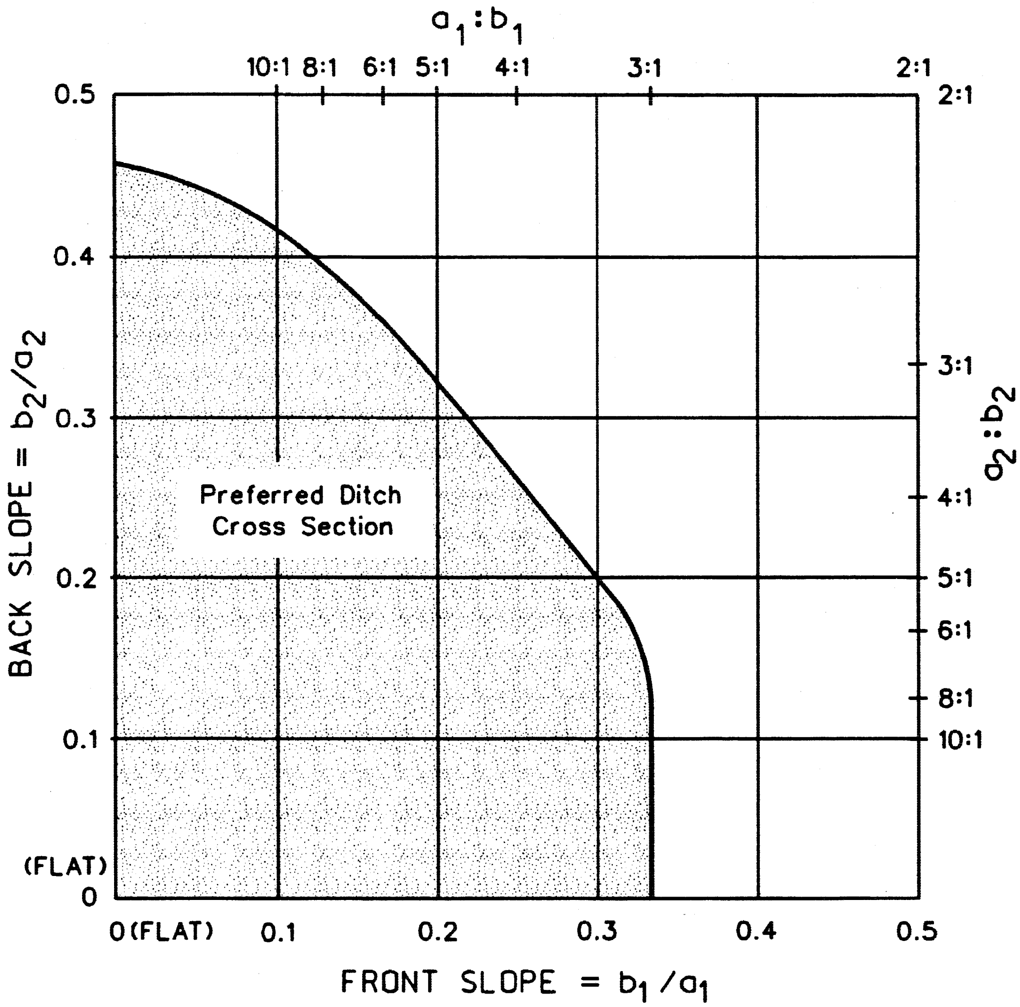
Note: This chart is applicable to all V-ditches, rounded ditches with a bottom width less than 8 feet, and trapezoidal ditches with bottom widths less than 4 feet.

Source: (1)

PREFERRED CROSS SECTIONS FOR DITCHES (With Abrupt Slope Changes)

Figure 11.3D





Note: This chart is applicable to all rounded ditches with a bottom width of 8 feet or more and to trapezoidal ditches with bottom widths equal to or greater than 4 feet.

Source: (1)

PREFERRED CROSS SECTIONS FOR DITCHES
(With Gradual Slope Changes)

Figure 11.3E

11.3.7.2 Curbs

Curbs are typically used for drainage control. In general, the use of curbs with a barrier is discouraged and, specifically, curbs higher than 4 inches should not be used with a barrier on new construction facilities with design speeds 50 mph or higher. Existing curb installations higher than 4 inches may remain if the installation otherwise meets ODOT criteria. Section 11.5 provides additional details for relative placement of curbs and guardrail. Section 8.1 and the *ODOT Standard Drawings* provide information on the different types of curbs used by ODOT and the criteria for their placement.

11.3.7.3 Cross Drainage Structures

If a slope is generally traversable, the preferred treatment for any cross drainage structure is to extend (or shorten) it to intercept the roadway embankment and to match the inlet or outlet slope to the embankment slope. This results in an extremely small "target" to hit, reduces erosion problems and simplifies mowing operations. For small culverts, no other treatment is required. For cross drainage structures, a small pipe culvert is defined as a single round pipe with a 30-inch or less diameter or multiple round pipes each with a 30-inch or less diameter. Extending or shortening culverts and their fill slopes, however, to locate the inlets/outlets beyond the clear zone is not recommended if such treatments introduce discontinuities in an otherwise traversable slope. See *ODOT Standard Drawings*.

Single structures, drop inlet openings, or end treatments wider than 30 inches shall be made traversable for passenger size vehicles by using bar grates or pipes to reduce the clear opening width. The *ODOT Standard Drawings* provide additional details in the design of bar grates. To maintain hydraulic efficiency, it may be necessary to apply bar grates to flared wingwalls, flared end sections or culvert extensions that are larger in size than the main barrel.

For intermediate sized pipes and culverts with inlets and outlets which cannot readily be made traversable, the pipe should be extended so that the obstacle is located outside of the clear zone. For major drainage structures which are costly to extend and with end sections which cannot be made traversable (e.g., 5-ft high box culverts), shielding with a barrier may be warranted.

11.3.7.4 Parallel Drainage Structures

Parallel drainage culverts are those which are oriented parallel to the main flow of traffic. They are typically used under driveways, field entrances, access ramps, intersecting side roads and median crossovers. As with cross drainage structures, the designer's primary concern should be to design generally traversable slopes and to match the culvert opening with adjacent slopes. Section 11.3.5 provides ODOT's criteria for transverse slope rates.

Parallel drainage structures should desirably match the selected side slope and be safely treated if practical when they are located in a vulnerable position relative to the main road traffic outside the clear zone. 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. The *ODOT Standard Drawings* provide additional details in the design of pipes and bars. Generally, single pipes with diameters of 24 inches or less will not require a grate. However, when a multiple-pipe installation is involved, consideration of a grate for the smaller pipes may be appropriate.

Parallel drainage structures may be closely spaced because of frequent driveways and intersecting roads. In such locations, it may be desirable to convert the open ditch into a storm drain and backfill the areas between adjacent driveways. This treatment will eliminate the ditch section as well as the transverse embankments with pipe inlets and outlets.

11.4 ROADSIDE BARRIERS

11.4.1 Barrier Types

Table 11.4A presents the roadside barrier types which are approved for use by ODOT. The table summarizes the hardware requirements for each system. The designer should reference the *ODOT Standard Drawings* for more detailed information. The following sections briefly describe each system and its typical usage.

11.4.1.1 "W" Beam Guardrail

The "W" beam system with heavy posts is a semi-rigid system. In general, this guardrail system is the preferred selection on non-freeway highways and on rural freeways. A major objective of the heavy post system is to prevent a vehicle from "snagging" on the posts. This is achieved by using blockouts to offset the posts from the longitudinal beam and by establishing 6′-3″ as the maximum allowable post spacing.

ODOT has approved the use of two guardrail "W" beam systems. One uses 6″ x 8″ wood posts, and the other uses either W6 x 8.5 or W6 x 9.0 steel posts. The selection of which system to use on a project is the Contractor's option.

11.4.1.2 Modified Thrie Beam

The modified thrie beam is a semi-rigid system with an 18-inch longitudinal W-beam section. The deeper longitudinal member of the thrie beam decreases the probability of an impacting vehicle overriding or underriding the barrier. The notch at the base of the blockout is designed to bend in upon impact. This dynamic reaction allows the longitudinal beam to remain nearly vertical, thereby enhancing barrier performance.

Based on crash tests, the modified thrie beam can accommodate vehicles ranging in size from an 1,800-pound vehicle to a 32,000-pound intercity bus. Because of its capacity to contain many large-vehicle impacts, the designer should consider using the modified thrie beam at locations which have experienced or may experience a high number of large-vehicle impacts. The modified thrie beam should also be considered where there is a concern for snagging (e.g., at bridge parapets, bridge piers, etc.).

There are two modified thrie beam systems available. One uses 6″ x 8″ wood posts, and the other uses either W6 x 8.5 or W6 x 9.0 steel posts. The selection of which system to use on a project is the Contractor's option. Whichever post system is selected, the blockout plate must always be steel.

11.4.1.3 Concrete Median Barrier (CMB)

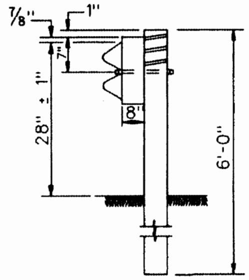
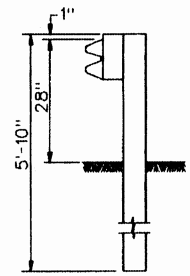
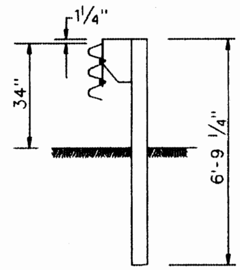
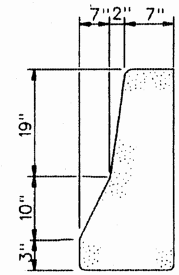
The CMB should be considered on the roadside to shield rigid objects where no deflection distance is available. The CMB is typically used on urban freeways wherever a barrier is required. If a rigid object is not continuous (e.g., bridge piers), the designer may use a half-section CMB. To provide the necessary lateral support, backfill should be provided behind the half-section CMB, or the CMB should be tied to a concrete surface with reinforcing steel. If this is not practical, use the full-section CMB. See *ODOT Standard Drawings* and Table 11.4A.

11.4.1.4 Vertical Wall

A vertical concrete wall may be appropriate at some locations. This includes, for example, in the median of a divided facility where rectangular-shaped bridge piers are used.

Table 11.4A

ROADSIDE BARRIERS

				
System	"W" Beam Guardrail (Wood Posts)	"W" Beam Guardrail (Steel Posts)	Modified Thrie Beam Guardrail (Steel Posts)	Concrete Median Barrier***
Post Spacing	6'-3"	6'-3"	6'-3"	N/A
Deflection Distance	4'*	4'*	3'*	0
Post Type	6" x 8" Treated Wood Post	W6 x 8.5 Steel Post or W6 x 9.0 Steel Post	W6 x 8.5 Steel Post** or W6 x 9.0 Steel Post	N/A
Beam Type	Steel "W" Section, 12 ga.	Steel "W" Section, 12 ga.	Steel Thrie Beam, 12 ga.	N/A
Offset Brackets (Blockouts)	6" x 8" x 14" Treated Block	W6 x 8.5 x 14" Steel Block or W6 x 9.0 x 14" Steel Block	W6 x 8.5 x 21½" Steel Block or W6 x 9.0 x 21½" Steel Block	N/A

See *ODOT Standard Drawings*.

* Clear distance measured from the face of the rail. Decreasing the post spacing to 3'-1½" will decrease the deflection distance by 1 ft.
** A 6" x 8" treated wood post may also be used.
*** Where the half-section CMB is used (as shown in figure), backup support is required.

11.4.2 Barrier Selection

The barrier performance-level requirements must be considered when selecting an appropriate roadside barrier. Traditionally, most barriers have been developed and tested for passenger cars and offer marginal protection when struck by heavier vehicles at high speeds and other than flat angles of impact. Therefore, if passenger vehicles are the primary concern, the "W" beam system will normally be selected. However, locations with poor geometrics, high traffic volumes and speeds, high-accident experience, and/or a significant volume of heavy trucks and buses may warrant a higher performance level barrier (e.g., the modified thrie beam or CMB). This is especially important if barrier penetration by a vehicle is likely to have serious consequences.

The dynamic deflection must also be considered in barrier selection. Table 11.4A provides the deflection distances for the various systems. If the deflection distance is not available, the railing system should be stiffened or a CMB should be used.

Another consideration in selecting the barrier type depends on maintenance of the system. Although the "W" beam can often sustain second hits, it will need to be repaired with some frequency. In areas of restricted geometry, high speeds, high traffic volumes and/or where railing repair creates hazardous conditions for both the repair crew and for motorists using the roadway, a rigid barrier should be considered. The CMB also allows better control of roadside vegetation, and it provides a more convenient means to transition into bridge piers. For these reasons, the CMB is typically used on urban freeways when a barrier is required.

Table 11.4B summarizes the advantages and disadvantages of the roadside barriers used by ODOT and provides their typical usage.

Table 11.4B

ROADSIDE BARRIER SELECTION

SYSTEM	ADVANTAGES	DISADVANTAGES	TYPICAL USAGE
W-Beam Guardrail	1. Lowest initial cost. 2. High level of familiarity by maintenance personnel. 3. Can safely accommodate wide range of impact conditions for passenger cars. 4. Relatively easy installation. 5. Remains functional after moderate collisions.	1. Cannot accommodate impacts by large vehicles at other than flat angles of impact. 2. At high-impact locations, will require frequent maintenance. 3. Susceptible to vehicular underride and override. 4. Susceptible to vehicular snagging (without rub rail).	1. Non-freeways. 2. Rural freeways.
Modified Thrie Beam	1. Lower initial cost than CMB. 2. Improved underride/override protection. 3. Improved performance for large vehicle impacts. 4. Relatively easy installation. 5. Remains functional after moderate collisions.	1. Higher initial cost than W-beam guardrail. 2. Cannot accommodate severe impacts by large vehicles. 3. Some lack of familiarity by maintenance personnel.	1. Special sites only.
Concrete Median Barrier	1. Can accommodate most vehicular impacts without penetration. 2. Little or no deflection distance required behind barrier. 3. Little or no damage sustained for most vehicular impacts; therefore, least need for maintenance. 4. No vehicular underride/override potential or snagging potential.	1. High initial cost. 2. Can induce vehicular rollover. 3. For given impact conditions, highest occupant decelerations; therefore, least forgiving of barrier systems. 4. Reduced performance where offset between travel lane and barrier exceeds 15′.	1. Urban freeways. 2. Where high traffic volumes are present. 3. Where high volumes of large vehicles are present. 4. Where snagging is a concern.

End Treatment.
2 lane: use GET's on all sides

4 lane Div:
Approach - GET
Exit - Turn down
(Type B)

11.5 ROADSIDE BARRIER LAYOUT

11.5.1 Length of Need

A roadside barrier must be extended a sufficient distance upstream from the hazard to safely protect a run-off-the-road vehicle. Otherwise, the vehicle could travel behind the guardrail 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 hazard.

Many factors combine to determine the appropriate length of need for a given roadside condition. Figure 11.5A illustrates the variables that must be considered in designing a roadside barrier to shield a hazard effectively. Once the appropriate variables have been selected, the required length of need can be calculated from the following formulas:

Flared Design

$$X = \frac{L_H + \frac{b}{a}(L_1) - L_2}{\frac{b}{a} + \frac{L_H}{L_R}} \qquad (11\text{-}1)$$

$$Y = L_H - \frac{L_H}{L_R}(X) \qquad (11\text{-}2)$$

Unflared Design

$$X = \frac{L_R(L_H - L_2)}{L_H} \qquad (11\text{-}3)$$

$$Y = L_2 \qquad (11\text{-}4)$$

where:

X, Y = coordinates of end of guardrail need.

a/b = guardrail flare (e.g., 15:1) (see Table 11.5A).

L_C = recommended clear zone.

L_H = distance from edge of travel lane to back of hazard. The lateral extent of the hazard (L_H), is the distance from the edge of the travel lane to the far side of the hazard if the hazard is a fixed object, – if inside clear zone – or to the outside edge of the clear zone, L_C, if the hazard is an embankment or a fixed object that extends beyond the clear zone.

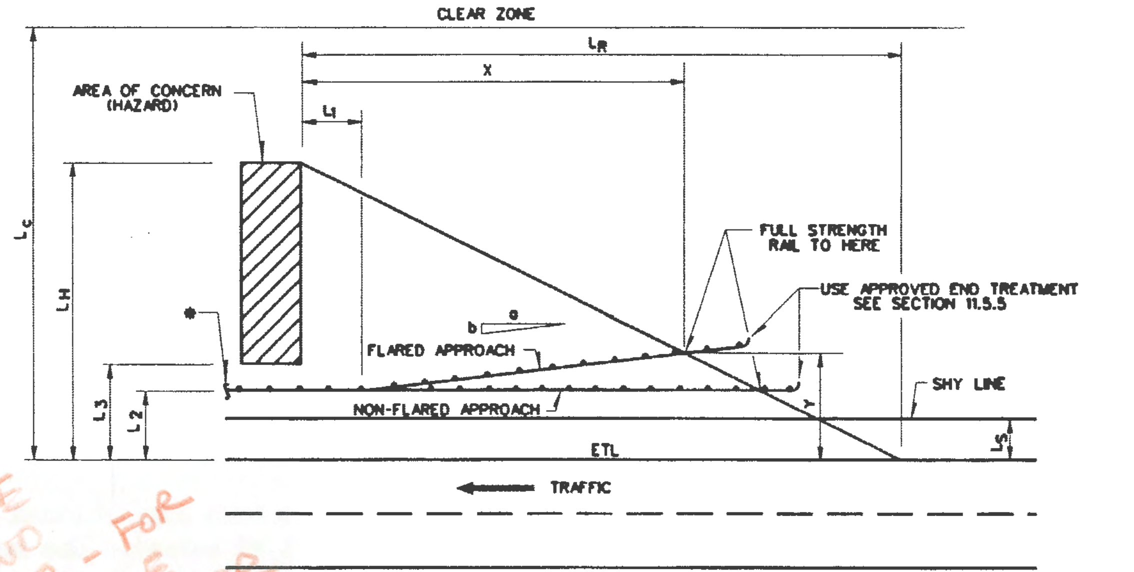
L_S = shy line offset, or distance at which guardrail is no longer perceived as an obstacle by a driver (see Table 11.5A).

L_R = runout length (see Table 11.5A).

L_1 = distance from hazard to where guardrail flare begins.

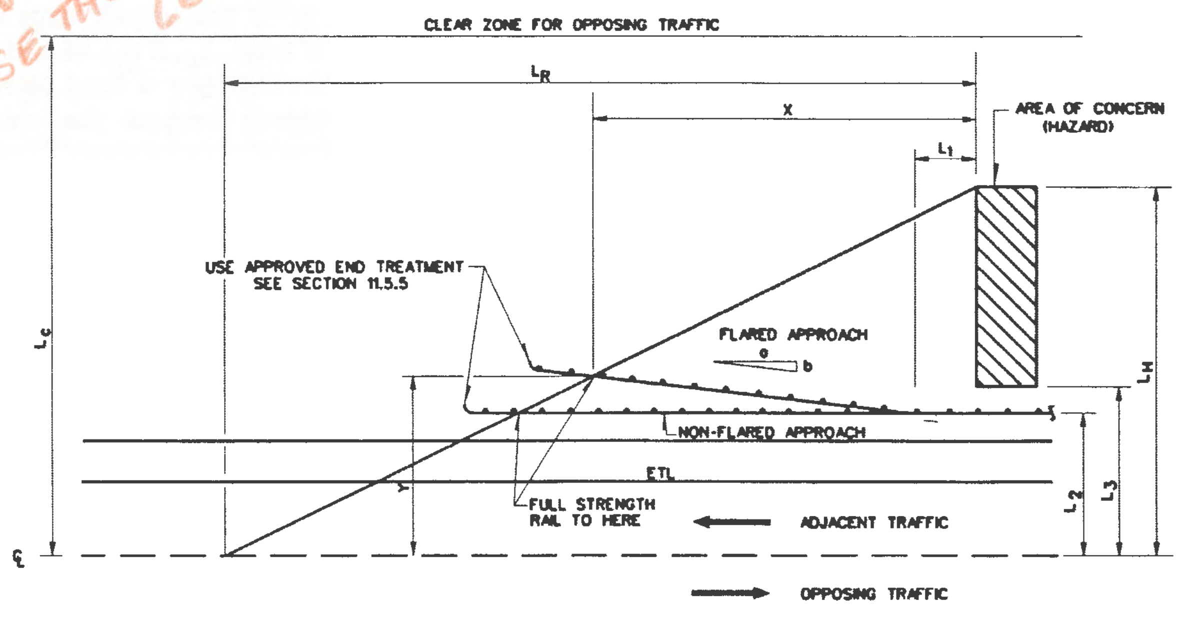
L_2 = distance from edge of travel lane to guardrail.

L_3 = distance from edge of travel lane to front of hazard. (L_3 - L_2) must equal or exceed deflection distance.



APPROACH TREATMENT FOR ONCOMING TRAFFIC (a)

* COMPARE THESE TWO LENGTHS - FOR LEAVING SIDE CHOOSE THE HIGHER LENGTH



APPROACH TREATMENT FOR OPPOSING TRAFFIC (b)

* THE DISTANCE BEYOND THE HAZARD SHOULD BE 50' OR AS DETERMINED BY A LENGTH-OF-NEED CALCULATION FOR OPPOSING TRAFFIC.

Note: X and Y are shown for the flared approach.

Source: (1) Revised

BARRIER LENGTH OF NEED

Figure 11.5A

Table 11.5A

DESIGN ELEMENTS FOR BARRIER LENGTH OF NEED

Design Speed (mph)	Design Traffic Volume (ADT)				L_S Shy Line Offset (ft)	Flare Rates		
	Over 6000	2000-6000	800-2000	Under 800		Inside of Shy Line	Outside of Shy Line	
	L_R(ft)	L_R(ft)	L_R(ft)	L_R(ft)			Guardrail	CMB
70	480	440	400	360	10.0	30:1	15:1	20:1
65	440	400	365	330	9.0	28:1	14:1	18:1
60	400	360	330	300	8.0	26:1	13:1	17:1
55	360	325	295	270	7.25	24:1	12:1	16:1
50	320	290	260	240	6.5	21:1	11:1	14:1
45	280	255	230	210	5.75	19:1	10:1	13:1
40	240	220	200	180	5.0	17:1	9:1	11:1
35	210	190	170	160	4.25	15:1	8:1	10:1
30	170	160	140	130	3.5	13:1	7:1	8:1
25	140	130	120	110	2.75	12:1	7:1	8:1
20	110	100	90	80	2.0	10:1	7:1	8:1

Source: (1) Revised

* * * * * * * * * * *

__Example 11-4 (Roadside Obstacle)__

Given: The following site conditions apply to a roadside obstacle (see Figure 11.5B):

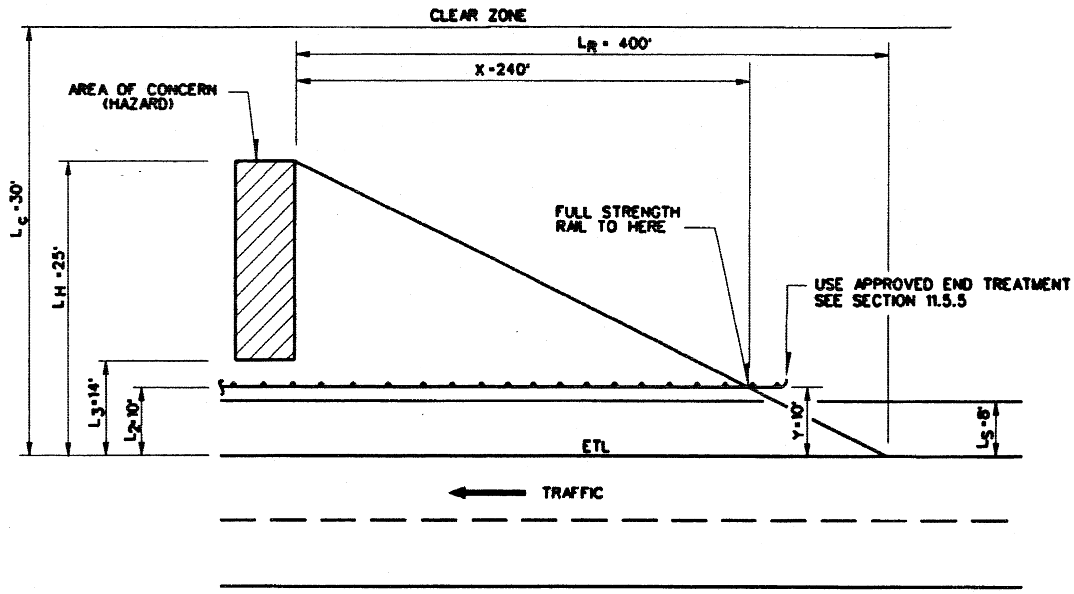
ADT = 7000
V = 60 mph
Slope = 6:1 fill slope
Tangent Roadway
Shoulder Width = 8′
L_H = 25′ (to back of hazard)

Problem: Determine the details of barrier location. Use the W-beam system with an unflared layout. $L_1 = 0$.

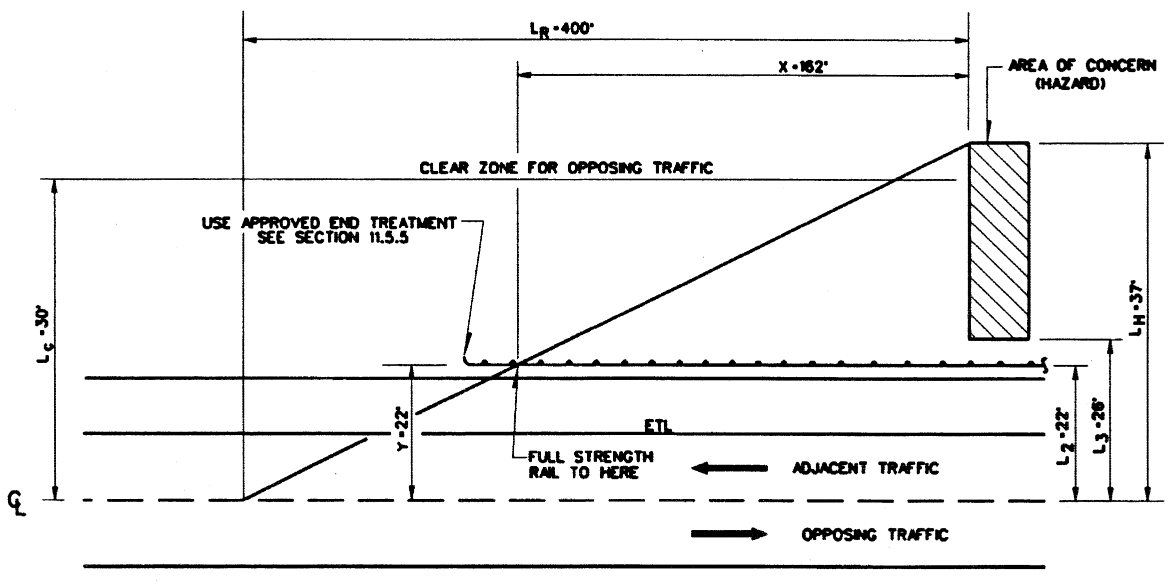
Solution: The following steps apply:

1. __Determine Clear Zone__. From Table 11.2A, the clear zone is 30′. No adjustment for horizontal curvature is necessary. Therefore, L_C = 30′. The hazard is within the clear zone and, therefore, L_H = 25′.

2. __Determine Barrier Lateral Location (L_2)__. From Table 11.5A, L_S = 8.0′ (the shy line offset). As discussed in Section 11.5.2, the barrier lateral location should be at L_S or the shoulder width, whichever is greater. Desirably, the shoulder will be widened by 2′ and the barrier face will be placed at the edge of the widened shoulder. For this example, use L_2 = 10′.



APPROACH TREATMENT FOR ONCOMING TRAFFIC (a)



APPROACH TREATMENT FOR OPPOSING TRAFFIC (b)

BARRIER LENGTH OF NEED FOR ROADSIDE OBSTACLE
(Example 11-4)

Figure 11.5B

3. Determine "L_3". From Table 11.4A, the deflection distance for the W-beam guardrail is 4′. $L_3 = L_2 + 4' = 14'$. Therefore, the front of the hazard can be no closer than 14′ from the edge of travel lane.

4. Determine "L_R". From Table 11.5A, $L_R = 400'$.

5. Determine Length of Need Coordinates (X,Y). For the approaching traffic, Equations 11-3 and 11-4 yield the following for the unflared design:

$$X = \frac{400\ (25-10)}{25} = 240'$$

$$Y = 10'$$

For the opposing traffic and assuming 12′ travel lanes, the following adjustments are made:

$$L_C = 30'$$

$$L_H = 37'$$

(Note: The back of the hazard is outside of the clear zone for opposing traffic; however, the entire hazard may be protected for a small additional cost.)

$$L_2 = 10' + 12' = 22'$$

$$L_R = 400'$$

$$X = \frac{400\ (37-22)}{37} = 162'$$

$$Y = 22'$$

Discussion: A flared barrier will considerably shorten the required length of guardrail.

Assuming $L_1 = 0$ and $a/b = 15{:}1$, Equations 11-1 and 11-2 yield the following:

Approach	Opposing
X = 116′	X = 95′
Y = 18′	Y = 29′

Example 11-5 (Bridge Rail Approach)

Given: The following site conditions apply to a bridge rail approach (see Figure 11.5C):

ADT = 4000
V = 55 mph
Slope = 3:1 fill slope
Embankment Height = 10′
Tangent Roadway
Shoulder Width = 10′
Graded Shoulder Width = 15′

Problem: Determine the details of barrier location. Use the W-beam system with an unflared layout.

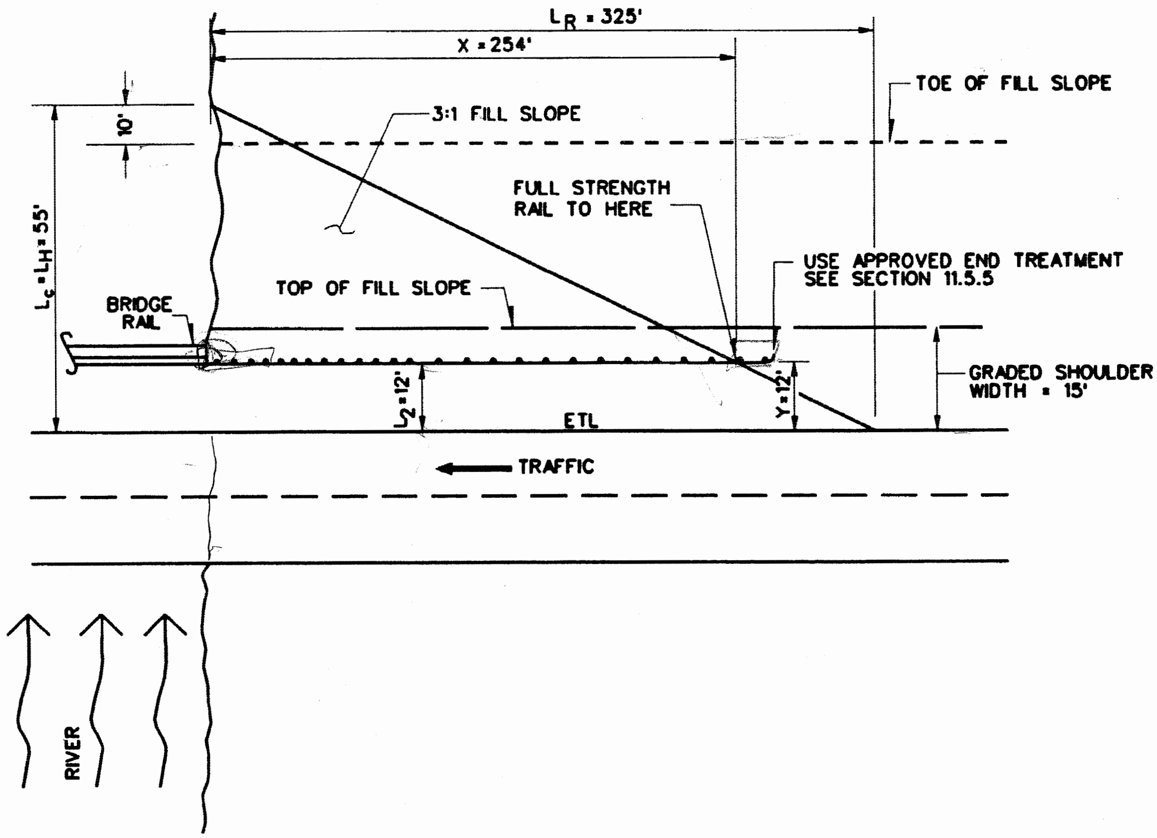
Solution: The following steps apply:

1. Determine Clear Zone. Based on the discussion in Section 11.2, the clear zone will extend a minimum of 10′ beyond the toe of the 3:1 fill slope. Therefore, the minimum L_H from the edge of travel lane is the sum of the graded shoulder width plus the additional barrier offset plus the lateral extent of fill slope plus 10′:

$$L_H = 15' + (3 \times 10') + 10' = 55'$$

The hazards within the clear zone are the bridge rail end and the river.

2. Determine Barrier Lateral Location (L_2). From Table 11.5A, $L_S = 7.25'$ (the shy line offset). As discussed in Section 11.5.2, the barrier lateral location should be at L_S or the shoulder width, whichever



BARRIER LENGTH OF NEED FOR BRIDGE RAIL
(Example 11-5)

Figure 11.5C

is greater. Desirably, the shoulder will be widened by 2′ and the barrier will be placed at the edge of the widened shoulder. For this example, use L_2 = 12′.

3. Determine "L_3". Not applicable.

4. Determine "L_R". From Table 11.5A, L_R = 325′.

5. Determine Length of Need Coordinates (X,Y). For the approaching traffic, Equations 11-3 and 11-4 yield the following for the unflared design:

$$X = \frac{325\ (55 - 12)}{55} = 254'$$

$$Y = 12'$$

Example 11-6 (Embankments)

Given: The following site conditions apply to an embankment (see Figure 11.5D)

ADT = 5000
V = 60 mph
Slope = 4:1 fill slope constantly transitioning to a 2:1 fill slope
Embankment Height = 20′ @ 3:1 slope rate
No roadside hazards beyond toe of slope.
Tangent Roadway
Shoulder Width = 8′
Graded Shoulder Width = 12′

1. Determine Clear Zone. Based on the discussion in Section 11.3, a fill slope does not become a roadside hazard until its steepness exceeds 3:1. Therefore, in the absence of any roadside hazard beyond the toe of fill slope, the lateral extent of hazard in this example is at the toe of the 3:1 fill slope. L_H, then, is the sum of the shoulder width plus the additional barrier offset plus the lateral extent of the 3:1 fill slope:

$$L_H = 8' + 4' + (3 \times 20') = 72'$$

2. Determine Barrier Lateral Location (L_2). From Table 11.5A, L_S = 8.0′ (the shy line offset). As discussed in Section 11.5.2, the barrier lateral location should be at L_S or the shoulder width, whichever is greater. Desirably, the shoulder will be widened by 2′ and the barrier will be placed at the edge of the widened shoulder. For this example, use L_2 = 10′.

3. Determine "L_3". Not applicable.

4. Determine "L_R". From Table 11.5A, L_R = 360′.

5. Determine Length of Need Coordinates (X,Y). For the approaching traffic, Equations 11-3 and 11-4 yield the following for the unflared design:

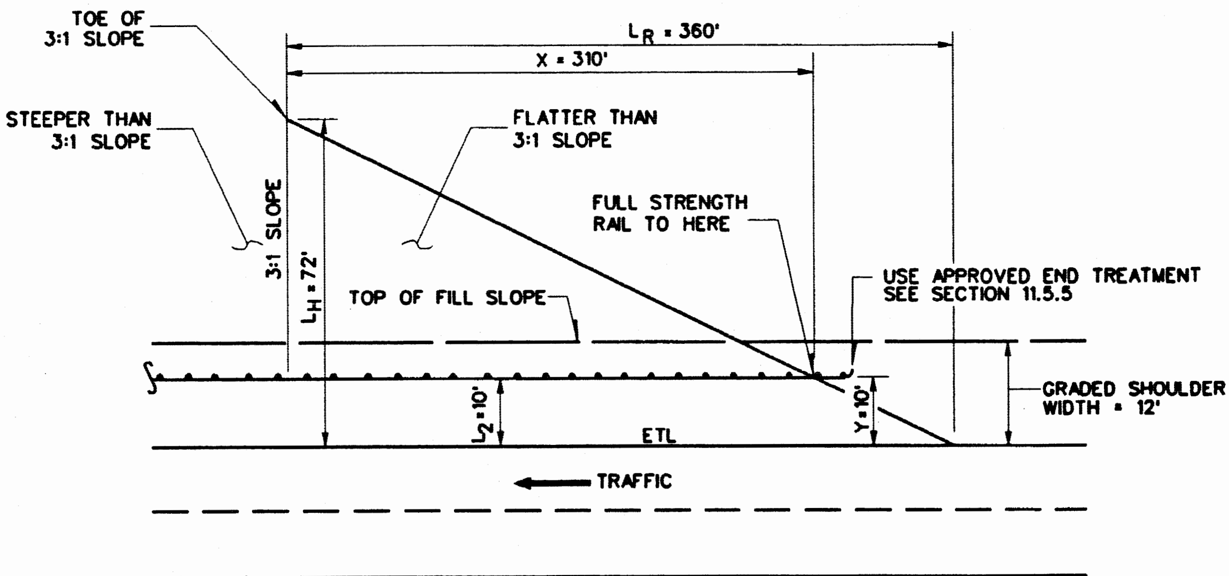
$$X = \frac{360\ (72 - 10)}{72} = 310'$$

$$Y = 10'$$

* * * * * * * * * * *

11.5.2 Lateral Placement

Roadside barriers should normally be placed as far as practical from the edge of the travel lane. Such placement gives an errant motorist the best chance of regaining control of the vehicle without impacting the barrier. It also provides better sight distance, particularly at nearby intersections. These factors should be considered when determining barrier lateral placement:



BARRIER LENGTH OF NEED AT EMBANKMENT
(Example 11-6)

Figure 11.5D

1. Deflection: The dynamic deflection distance of the barrier, as measured from the face of the rail, should not be violated. Table 11.4A provides the deflection distances for the types of roadside barriers used by ODOT.

2. Post Support. At a minimum, 2 ft should be provided between the back of the barrier post and the slope break (PI) in a fill slope to provide adequate soils support for the post.

3. Shy Distance. Drivers tend to "shy" away from continuous longitudinal obstacles along the roadside, such as guardrail. Therefore, the lateral barrier offset should desirably be based on the criteria in Table 11.5A.

4. Shoulder Widening. When barrier is placed next to the shoulder, desirably the shoulder width should be increased by an additional 2 ft. In addition, the area between the edge of the travel lane and to a point 2 ft behind the barrier should be paved, even if the normal shoulder is unpaved. See *ODOT Standard Drawings* for additional information on guardrail placement and shoulder base widening.

Figure 11.5E illustrates these criteria in the lateral placement of a roadside barrier.

11.5.3 Placement Behind Curbs

If practical, roadside barriers should not be placed in conjunction with mountable or barrier curbs. Where this is necessary, the following will apply (see Figure 11.5F):

1. Roadside Barrier/Curb Orientation. Typically, the face of the barrier should be in line with the face of the curb (i.e., at the gutter line). At a maximum, the barrier face should typically be no more than 9 inches from the face of the curb. The height of the guardrail must be measured from the pavement surface (e.g., where curbs are on bridges).

2. Lateral Placement. Table 11.5B presents criteria to determine proper barrier placement behind curbs.

Table 11.5B

BARRIER PLACEMENT BEHIND CURBS

Design Speed (mph)	Barrier Curb	Mountable Curb
≤35	NA	NA
40	8.5′	8.5′
45	9.0′	9.0′
≥50	Note (2)	Note (2)

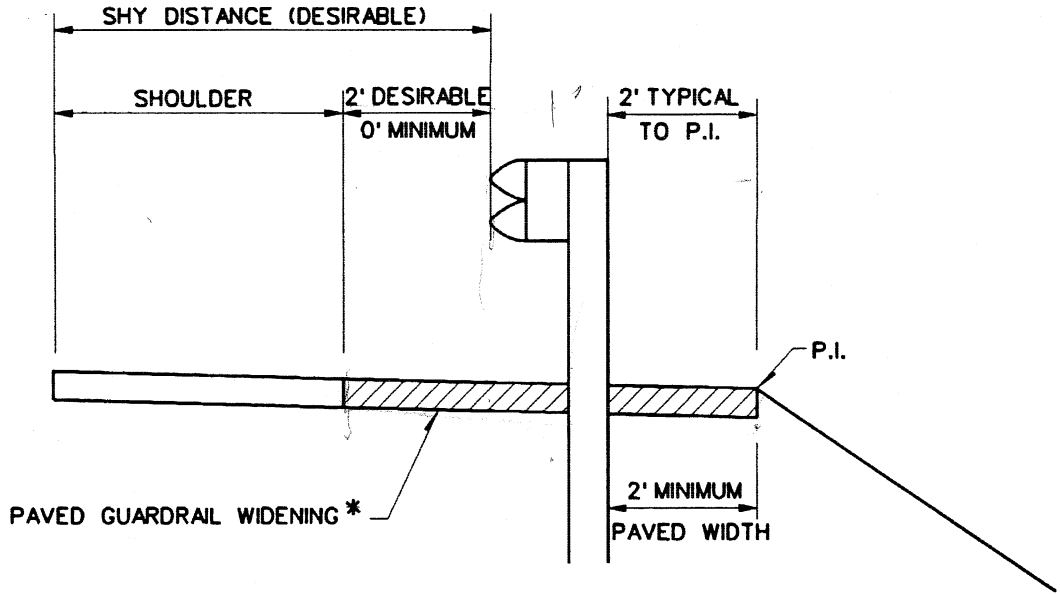
Source: (2) Revised

Notes: 1. *Value in table represent distance beyond which it is acceptable to place a barrier. See Figure 11.5F.*

2. *For design speeds ≥50 mph, curbs should not be used with a barrier on new construction.*

11.5.4 Placement on Slopes

Slopes in front of a barrier should be 10:1 or flatter. This also applies to the areas in front of the flared section of guardrail and to the area approaching the terminal ends. In some cases, it may be acceptable to use slopes steeper than 10:1. See the discussion in Reference (1).



*WHERE EXISTING SHOULDERS ARE UNPAVED,
THEY WILL TYPICALLY BE PAVED IN FRONT OF BARRIER.

NOTE: SEE ODOT STANDARD DRAWINGS FOR ADDITIONAL DETAILS.

Source: (1) Revised

LATERAL PLACEMENT OF BARRIER

Figure 11.5E

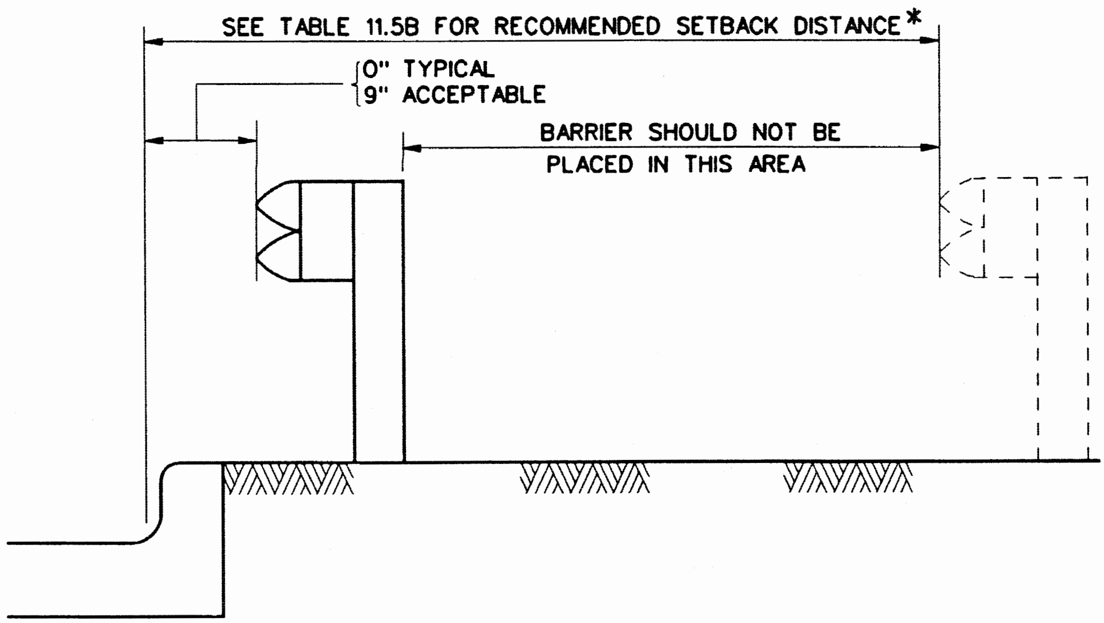


*TABLE 11.5B IS NOT APPLICABLE FOR DESIGN SPEEDS OF 35 MPH OR LESS. AT THESE SPEEDS, ANY BARRIER PLACEMENT IS ACCEPTABLE.

Source: (1) Revised

PLACEMENT OF BARRIER RELATIVE TO CURBS

Figure 11.5F

11.5.5 Terminal Treatments

Barrier terminal sections present a potential roadside hazard for run-off-the-road vehicles. However, they are also critical to the proper structural performance of the barrier system. Therefore, the designer must carefully consider the selection and placement of the terminal end. As described in the *ODOT Standard Drawings*, the following terminal treatments are used by ODOT:

1. Anchor Unit Type A. This guardrail anchor provides a gradual height transition up to the full rail height. The *ODOT Standard Drawings* provide details on the design and placement of this anchorage unit.

2. Anchor Unit Type B. This guardrail anchor is typically used on the trailing end of a barrier on a one-way facility. It provides the structural capacity required at the terminus of the guardrail.

3. Anchored in Back Slope. Wherever practical, the use of this anchor is encouraged. It essentially eliminates the possibility for an errant vehicle to strike the terminal end or to run behind the terminal.

4. Special Terminal End Treatments. There are several types of special end treatments available in the market (e.g., SENTRE, CAT). Selection should be based upon manufacturer's recommendations.

5. Bridge Connections. The *ODOT Standard Drawings* provide details for anchorage systems used at bridge connections.

No specific guidance is presented on when to use Anchorage Unit Type A (#1) or the special end treatments available (#4). On Federal-aid projects, the selection of end treatments will be made between ODOT and FHWA on a project-by-project and site-by-site basis.

11.5.6 Transitions

Guardrail transitions are necessary to join two systems with different structural and/or dynamic characteristics. For example, this occurs when guardrail approaches a bridge parapet or CMB installation. The *ODOT Standard Drawings* provide details for anchor units and standard guardrail. See Section 7.8 of Reference (1) for additional discussion.

11.5.7 Minimum Length/Gaps

Short runs of barrier have limited value, and they should be avoided. As a rule of thumb, a barrier should have at least 100 ft of standard guardrail section exclusive of terminal sections and/or transition sections. Likewise, short gaps between runs of barrier are undesirable. Therefore, gaps of less than 200 ft between barrier termini should be connected into a single run. Exceptions may be necessary for access.

11.6 MEDIAN BARRIERS

11.6.1 Warrants

The following summarizes the Department's criteria:

1. Freeways. Figure 11.6A 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 medians not within the optional or warranted area, if a significant number of crossover accidents have occurred.

2. 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 will reduce the likelihood of a crossover accident. Therefore, 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 11.6A can be used for some guidance.

11.6.2 Types

Table 11.6A presents the types of median barriers which are approved for use by ODOT. The following briefly describes each type:

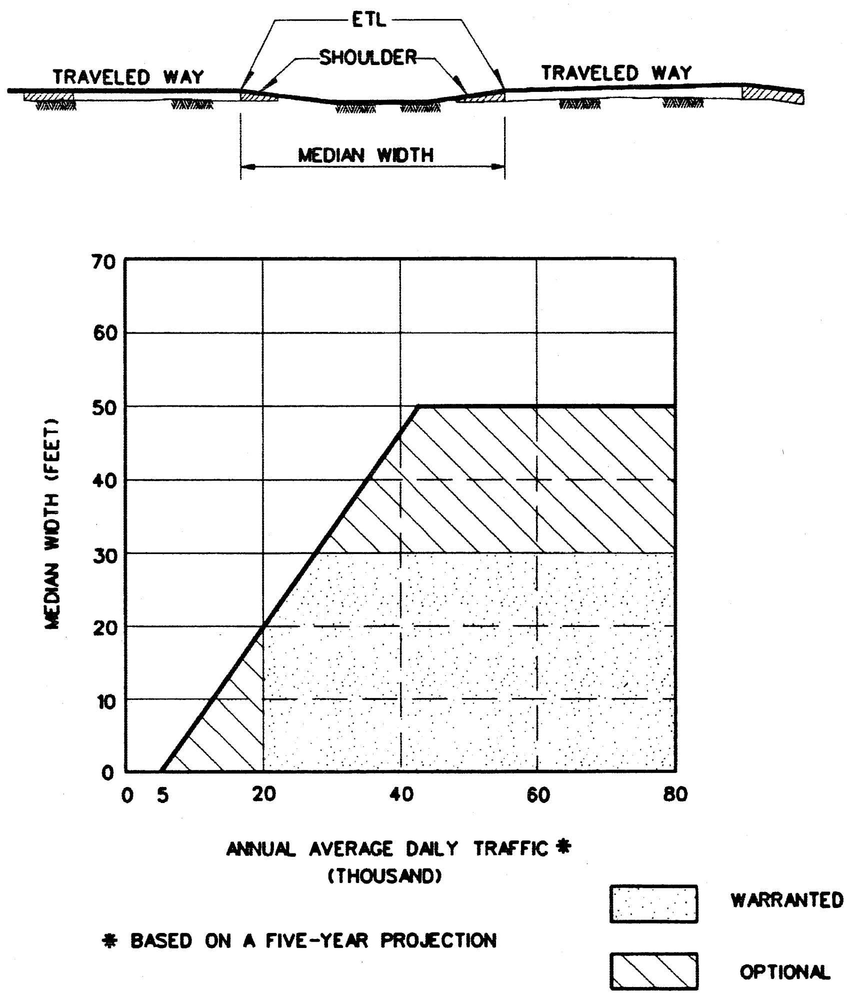
1. "W" Beam Median Barrier. The "W" beam median barrier with heavy posts is a semi-rigid system. Its performance is similar to the "W" beam guardrail system. This median barrier is most applicable to medians with intermediate width. Another application of the W-beam median barrier is for the separation of adjacent on/off ramps at interchanges.

   ODOT has approved the use of two "W" beam median barriers. One uses 6" x 8" timber posts, and the other uses either W6 x 8.5 or W6 x 9.0 steel posts. The selection of which system to use on a project is the Contractor's option.

2. Modified Thrie Beam Median Barrier. The modified thrie beam median barrier on heavy steel posts is a semi-rigid system. Its performance is similar to the thrie beam guardrail system. As with the modified thrie beam guardrail, the thrie beam median barrier is only used in special situations where it is judged to be a better selection.

3. Concrete Median Barrier. The concrete median barrier (CMB) is a rigid system which will rarely deflect upon impact. A half-section CMB may be necessary where the median barrier must divide to go around a fixed object in the median (e.g., bridge piers). In this situation, the obstacle is typically encased within concrete to create a level surface from CMB face to CMB face.

4. Tall CMB. The standard 32-inch CMB 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 undesirable results. Therefore, on some freeways it may be warranted to install the "tall" CMB, which is 42 inches high. The barrier maintains the standard safety shape of the 32-inch CMB, but it is extended an additional 10 inches.



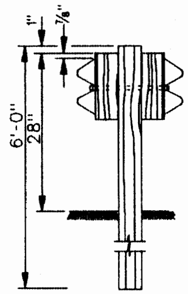
Source: (1)

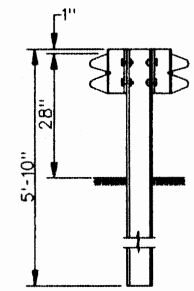
MEDIAN BARRIER WARRANTS

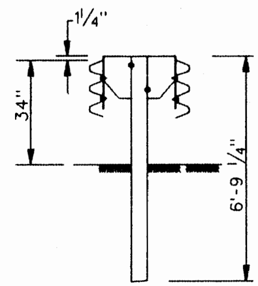
Figure 11.6A

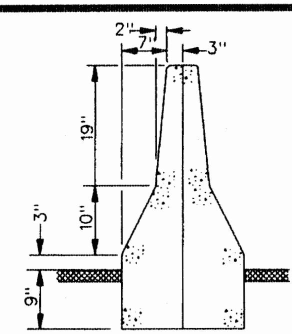
Table 11.6A

MEDIAN BARRIERS









System	"W" Beam Median Barrier (Wood Posts)	"W" Beam Median Barrier (Steel Posts)	Modified Thrie Beam Median Barrier (Steel Posts)	Concrete Median Barrier**
Post Spacing	6'-3"	6'-3"	6'-3"	N/A
Deflection Distance*	4'	4'	3'	0
Post Type	6" x 8" Treated Wood Post	W6 x 8.5 Steel Post or W6 x 9.0 Steel Post	W6 x 8.5 Steel Post*** or W6 x 9.0 Steel Post	N/A
Beam Type	Steel "W" Section, 12 ga.	Steel "W" Section, 12 ga.	Steel Thrie Beam, 12 ga.	N/A
Offset Brackets (Blockouts)	6" x 8" x 14" Treated Block	W6 x 8.5 x 14" Steel Block or W6 x 9.0 x 14" Steel Block	W6 x 8.5 x 21½" Steel Block or W6 x 9.0 x 21½" Steel Block	N/A

See *ODOT Standard Drawings*.

* Clear distance measured from the face of the rail. Decreasing the post spacing to 3'-1½" will decrease the deflection distance by 1 ft.
** See text for discussion on 42" CMB.
*** A 6" x 8" treated wood post may also be used.

However, this restricts sight distance around horizontal curves and restricts vision for authorized vehicles (e.g., police) who wish to view the opposing lanes.

If there has been an adverse history of heavy-vehicle cross over accidents, the tall CMB should be considered.

11.6.3 Median Barrier Selection

ODOT 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 15 ft should desirably be avoided. Therefore, considering both maintenance and safety, this favors the use of the CMB on median widths up to about 30 ft and either the "W" beam or modified 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 is more likely to restrain and redirect heavy vehicles (trucks and buses) than the metal beam systems. Therefore, where there is a high volume of heavy vehicles, this may favor the CMB even on medians wider than 30 ft. Between the two metal beam systems, the modified 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. The CMB may also require a closed drainage system in the median, further increasing initial costs. However, the maintenance costs per impact on the CMB will probably be far less, which 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 on a substandard barrier. This observation favors the use of the CMB which normally sustains far less damage when impacted. Second, the maintenance operations for repairing damaged barrier can interrupt traffic operations. It is particularly undesirable to close a traffic lane to repair a barrier. The consideration of maintenance operations generally favors the use of the CMB in narrow medians and/or on high-volume highways.

Table 11.6B summarizes the advantages and disadvantages of the median barriers used by ODOT and provides their typical usage.

11.6.4 Median Barrier Layout

Much of the information presented in Section 11.5 on roadside barrier layout also applies to median barriers (e.g., placement behind curbs). The following sections present criteria specifically for the design of median barriers:

Table 11.6B

MEDIAN BARRIER SELECTION

SYSTEM	ADVANTAGES	DISADVANTAGES	TYPICAL USAGE
W-Beam Median Barrier	1. Lowest initial cost. 2. High level of familiarity by maintenance personnel. 3. Can safely accommodate wide range of impact conditions for passenger cars. 4. Relatively easy installation. 5. Remains functional after moderate collisions.	1. Cannot accommodate impacts by large vehicles at other than flat angles of impact. 2. At high-impact locations, will require frequent maintenance. 3. Susceptible to vehicular underride and override. 4. Susceptible to vehicular snagging.	1. Wider medians. 2. Low to mid-range of traffic volumes.
Modified Thrie Beam Median Barrier	1. Lower initial cost than CMB. 2. Improved underride/override protection. 3. Improved performance for large vehicle impacts. 4. Relatively easy installation. 5. Remains functional after moderate collisions.	1. Higher initial cost than W-beam median barrier. 2. Cannot accommodate severe impacts by large vehicles. 3. Some lack of familiarity by maintenance personnel.	1. Special sites only.
Concrete Median Barrier	1. Can accommodate most vehicular impacts without penetration. 2. Little or no deflection distance required behind barrier. 3. Little or no damage sustained for most vehicular impacts; therefore, least need for maintenance. 4. No vehicular underride/override potential or snagging potential. 5. Light supports, sign supports, glare screens, etc. may be mounted on top.	1. High initial cost. 2. Can induce vehicular rollover. 3. For given impact conditions, highest occupant decelerations; therefore, least forgiving of barrier systems. 4. Reduced performance where offset between travel lane and barrier exceeds 15'.	1. Urban freeways. 2. Where high traffic volumes are present. 3. Where high volumes of large vehicles are present. 4. Where maintenance operations will require lane closure.

11.6.4.1 Sloped Medians

A median barrier should not be placed on a slope steeper than 10:1. Where the median slopes are steeper than 10:1, the designer should give special consideration to median barrier placement. Figure 11.6B illustrates three basic types of sloped medians. The following discusses barrier placement for each type (assuming a median barrier is warranted):

1. For Cross Section I, the designer should determine if the individual slopes warrant protection based on the criteria in Section 11.3. If both slopes warrant protection (Illustration 1), a roadside barrier should be placed at "b" and "d." If only one slope warrants protection, the median barrier should be placed to shield that slope. If neither slope warrants protection and both slopes are steeper than 10:1 (Illustration 2), a median barrier should be placed at "b" or "d," whichever is shielding the steeper slope. If the slopes are 10:1 or flatter (Illustration 3), the median barrier should be placed in the center of the median.

2. For Cross Section II, the slope in the median will determine the proper treatment. If the slope is steeper than 10:1 but flatter than 3:1 (Illustration 4), the median barrier should be placed at "b." If the median slope is 3:1 or steeper, a roadside barrier at "b" is the only necessary treatment. If the median slope is a roadside hazard (e.g., rough rock cut) (Illustration 5), a roadside barrier should be placed at both "b" and "d." If the median slope is 10:1 or flatter (Illustration 6), the median barrier should be placed in the center of the median.

3. For Cross Section III (Illustration 7), the redirective capacity of the median slope will determine the proper treatment. If the median slope is 3:1 or steeper and 3 ft or higher, no roadside or median barrier is necessary. If the median slopes are flatter than 3:1 and/or not 3-ft high, the median barrier should be placed at the apex of the cross section.

11.6.4.2 Flared/Divided Median Barriers

It may be necessary to intermittently divide a median barrier or to flare the barrier from one side to the other. A sloped median or a fixed object in the median may require this. The median barrier may be divided by one of these methods:

1. A fixed object may be encased by a CMB.

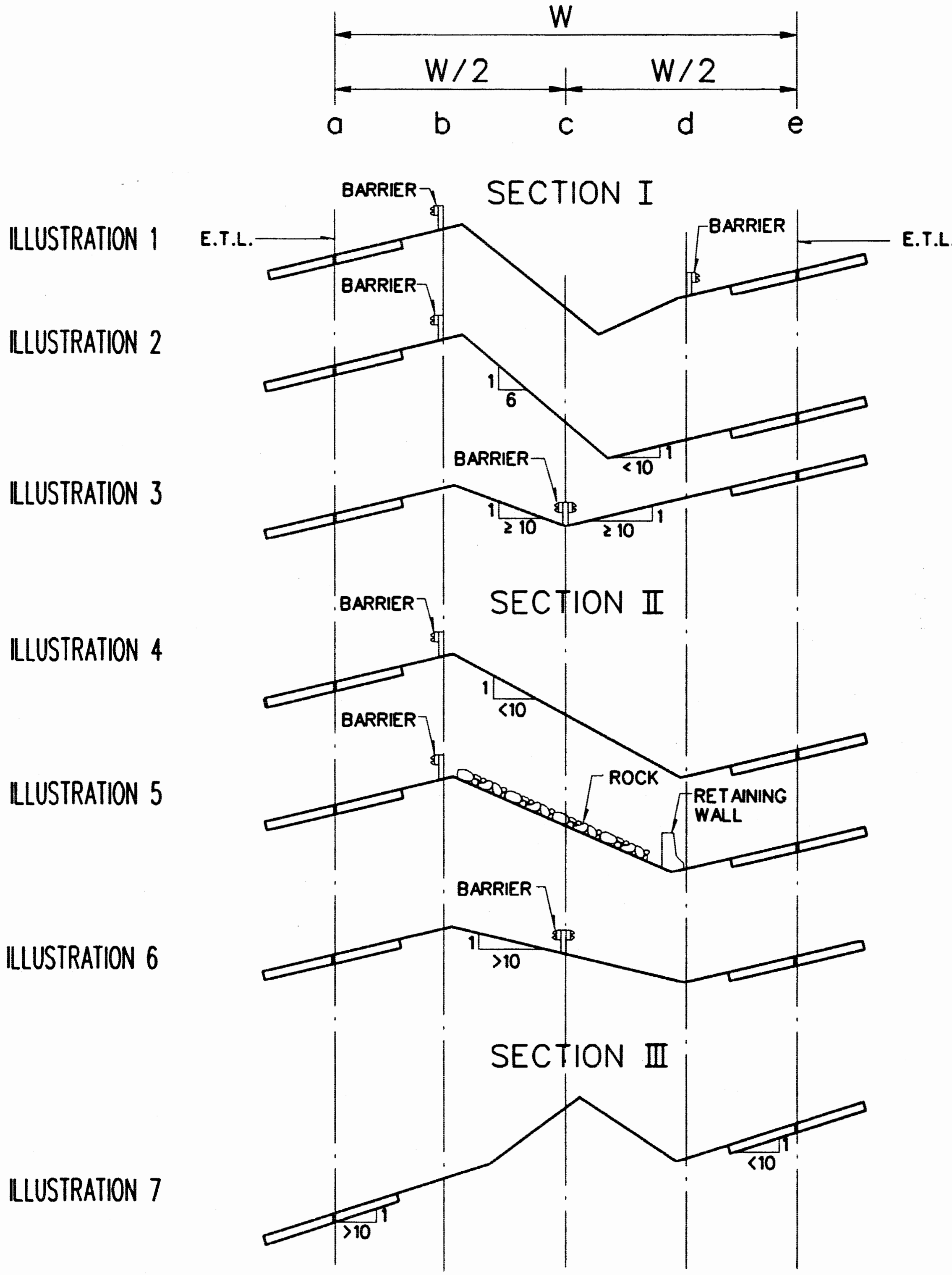
2. A half-section CMB may be used on both sides to shield a fixed object.

3. The metal beam median barriers can be split into two separate runs of 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 suggested flare rates. See Table 11.5A and the *ODOT Standard Drawings*.

11.6.4.3 Barrier-Mounted Obstacles

The designer may consider an additional factor when a CMB divides to pass on either side of an obstacle or when obstacles are mounted on top of a CMB (e.g., luminaire supports). 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 the obstacle on top of the CMB. Two potential counter-measures are to:



Source: (1)

MEDIAN BARRIER PLACEMENT
(Sloped Medians)

Figure 11.6B

1. provide a 2-ft deflection distance between the barrier and obstacle (e.g., bridge piers); or

2. use the 42-inch ("tall") CMB.

11.6.4.4 Terminal Treatments

As with roadside barrier terminals, median barrier terminals also present a potential roadside hazard for run-off-the-road vehicles. Therefore, careful consideration must be given to the selection and placement of the terminal end. If practical, the median barrier should be extended into a wider median area. The following terminal treatments are used by ODOT:

1. Special Terminal End Treatments (W-Beam, Thrie Beam). There are several types of special end treatments available for median metal beam rails (e.g., GREAT, CAT, BRAKEMASTER).

2. Impact Attenuator (CMB). For the terminal ends of concrete median barriers, impact attenuators are typically used (e.g., Sand Barrels, GREAT). See Section 11.7 for more information on impact attenuators.

No guidance is presented on when to use a specific anchorage type. On Federal-aid projects, the selection of end treatments will be made between ODOT and FHWA on a project-by-project and site-by-site basis.

11.6.5 Glare Screens

11.6.5.1 Warrants

Headlight glare from opposing traffic can be bothersome and distracting. Glare screens can be used in combination with median barriers to eliminate the problem. ODOT has not adopted specific warrants for the use of glare screens. The typical application, however, is on urban freeways with narrow medians and high traffic volumes. Another application is between on/off ramps at interchanges where the two ramps adjoin each other. Here, the sharp radii or curvature and the narrow separation may make headlight glare especially bothersome. The designer should consider the use of glare screens at these sites. A key element warranting their use is the number of public complaints ODOT may have received for a highway section.

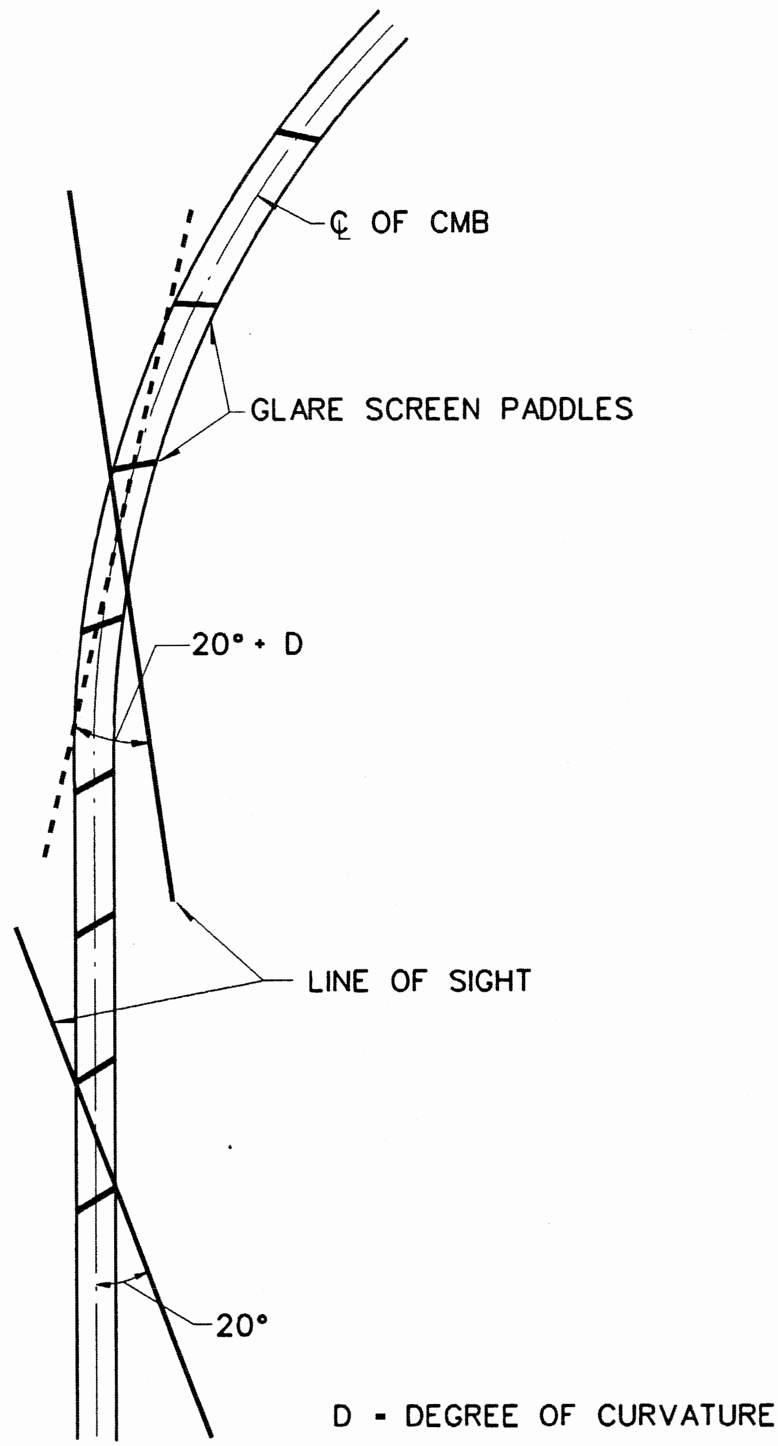
11.6.5.2 Design

Blocking headlight glare can be achieved in several ways:

1. Vegetation can be used; however, the designer should not introduce hazardous fixed objects in a narrow median.

2. Several commercial glare screens are available. One example is a series of plastic paddles. See the *ODOT Standard Drawings*.

Glare screens should be designed for a cutoff angle of 20°. This is the angle between the median centerline and the line of sight between two vehicles traveling in opposite directions. See Figure 11.6C. The glare screen should be designed to block the headlights of oncoming vehicles up to the 20° cutoff angle. On horizontal curves, the design



Source: (9) Revised

CUTOFF ANGLE FOR GLARE SCREENS

Figure 11.6C

cutoff angle should be increased to allow for the effect of the curvature on headlight direction. The criteria is:

Cutoff Angle = 20 + D

where: D = degree of curve

The designer should also evaluate the impact of a glare screen on horizontal sight distance on curves to the left. The screen could significantly reduce the available middle ordinate for stopping sight distance. See Section 6.5 for a discussion of sight distance at horizontal curves.

11.7 IMPACT ATTENUATORS (Crash Cushions)

11.7.1 General

Impact attenuators (crash cushions) 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. Impact attenuators are adaptable to many roadside hazard locations where longitudinal barriers cannot practically be used.

11.7.2 Warrants

Impact attenuator warrants are the same as barrier warrants. Once a hazard is identified, the designer should first attempt to remove, relocate or make the hazard break away. If the foregoing is impractical, then an impact attenuator should be considered.

Impact attenuators are most often installed to shield fixed-point hazards which are close to the travel lane. Examples include exit gore areas (particularly on structures), bridge piers, non-breakaway sign supports and median barrier ends. Impact attenuators are often preferable to guardrail to shield these hazards. Site conditions and costs will determine whether to use a barrier or impact attenuator.

11.7.3 Impact Attenuator Types

The following sections present the impact attenuators which are used by ODOT. A figure is presented for each system to illustrate a typical installation; however, each system can be designed for a wide range of performance and site conditions. Also, the designer should note that all of the operational systems are patented. See *ODOT Standard Drawings* for additional information on standard impact attenuator installations.

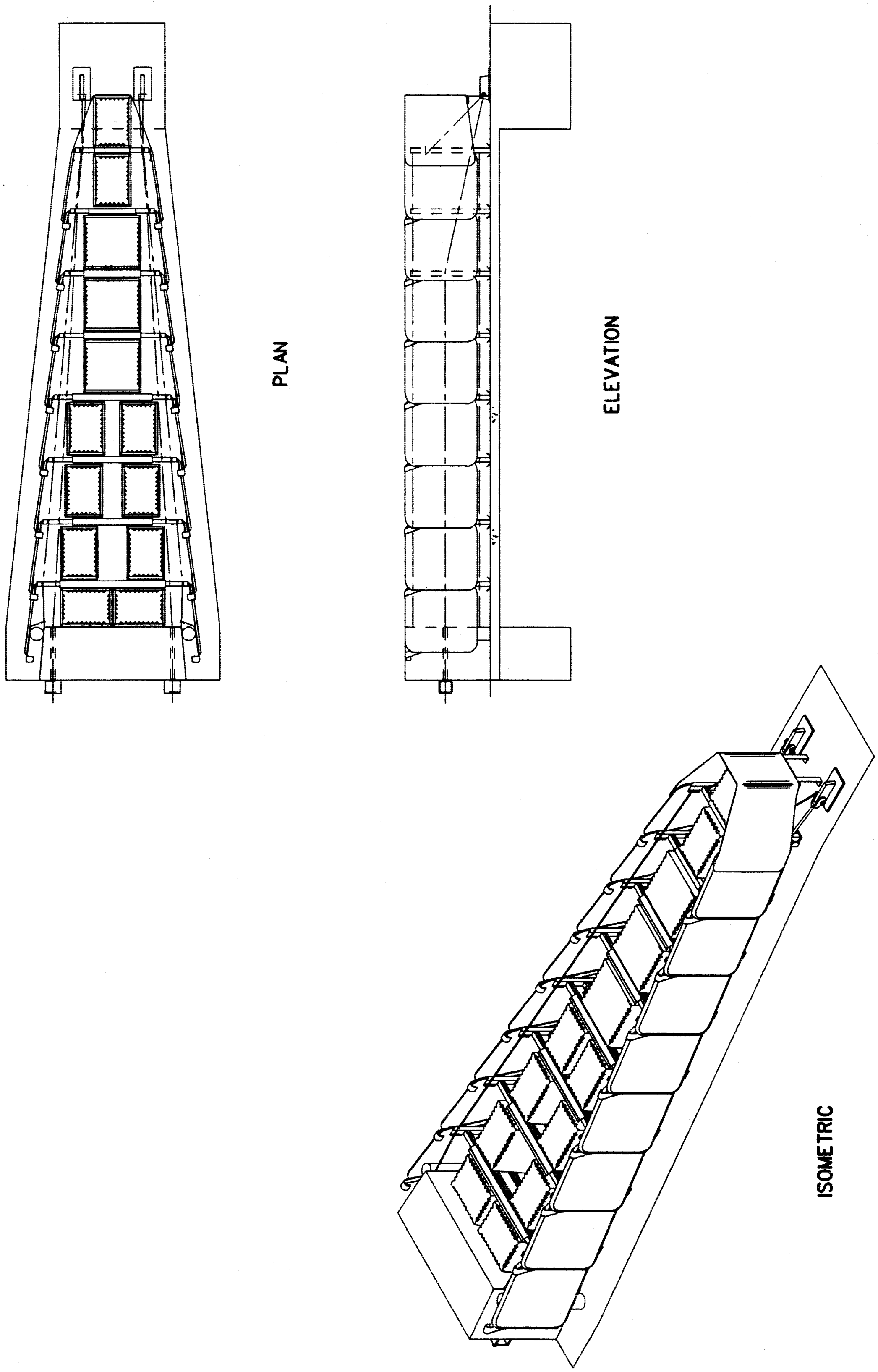
11.7.3.1 Hex Foam Sandwich

This impact attenuator is an array of bays filled with hex foam cartridges which gradually dissipate the vehicle's kinetic energy during impact. It requires a rigid back-up support and a vertical and lateral restraint with cables and diaphragms. Its side fender panels provide redirective capabilities. Figure 11.7A illustrates a typical Hex Foam Sandwich impact attenuator layout.

11.7.3.2 GREAT

The Guard Rail Energy-Absorbing Terminal (GREAT) uses hex foam cartridges similar to those in the Hex Foam Sandwich impact attenuator. Its redirective capabilities are achieved with triple corrugated structural plate beams (thrie beams) that telescope when impacted at the system's nose. The unit is restrained laterally by proof coil chains at the bottom and longitudinally by cables at the top for side impacts. A back-up structure and base pad are required.

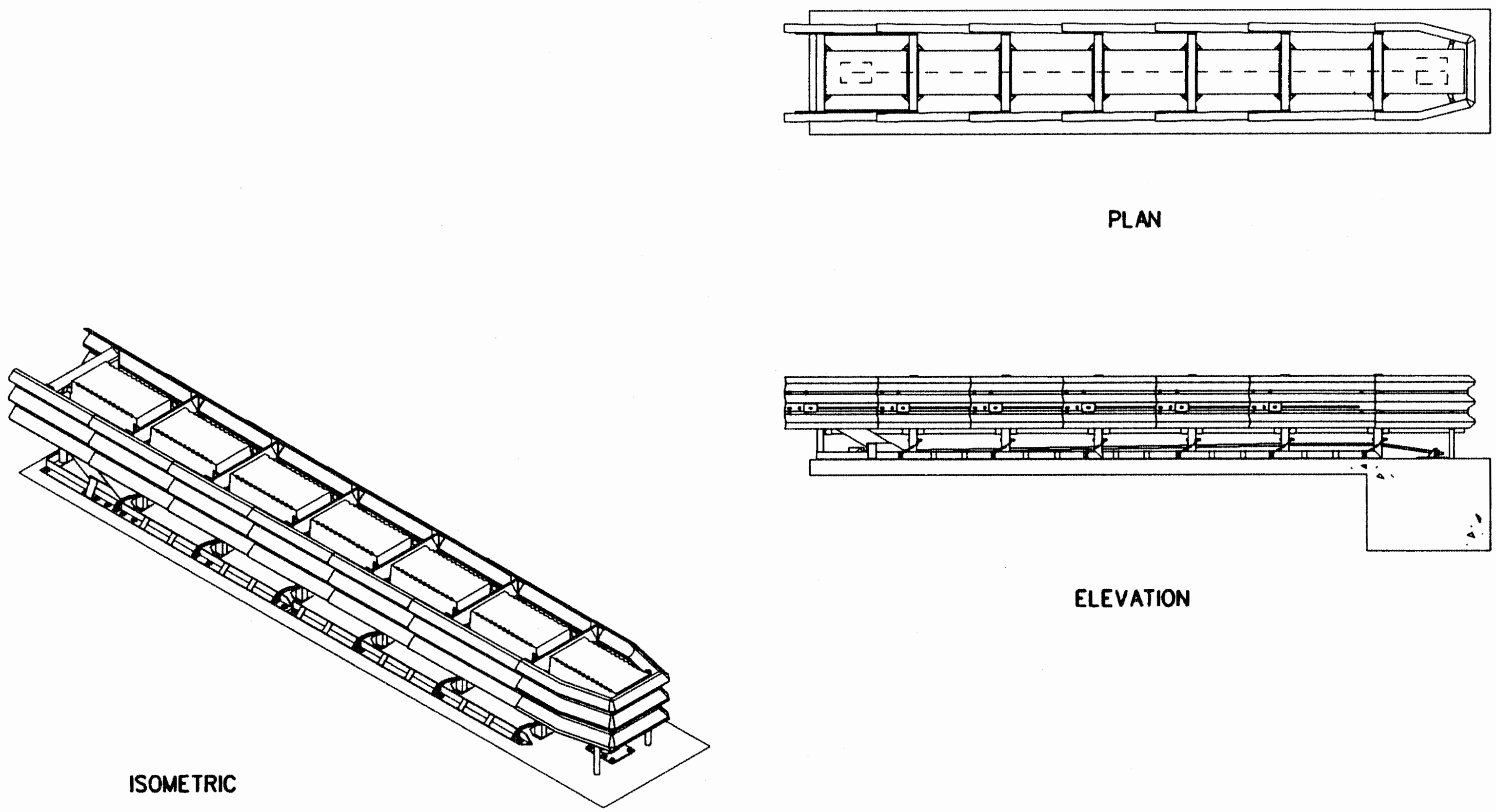
The GREAT's primary advantage is its adaptability to narrow obstacles where encroachments beyond the width of the obstacle cannot be tolerated. An example is the exposed end of a concrete median barrier in a narrow median. The number of bays can be varied to accommodate any design speed. It is available in standard widths of 2 ft, 2.5 ft and 3 ft. Figure 11.7B presents a typical GREAT layout.



Source: (12)

HEX-FOAM SANDWICH SYSTEM

Figure 11.7A



Note: Where traffic passes in opposing directions on either side of the GREAT, the corrugations will overlap differently.

Source: (12)

GREAT SYSTEM

Figure 11.7B

11.7.3.3 Sand Barrels (Inertial System)

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. This system has a low initial cost but relatively high maintenance cost. Therefore, sand barrel systems are best suited for low-impact locations (e.g., less than 3 hits a year). In addition, it may be the only practical impact attenuator for very wide hazards (e.g., a bridge pier in the center of a median).

The system requires no back-up support. However, the barrels have no redirective capability, and it generates considerable debris upon impact. The barrels should be placed within 1 ft of the hazard and are usually spaced 6 inches from module to module. The exterior modules must be laterally offset at least 30 inches from the corner of the hazard. Figure 11.7C presents a typical layout for a sand barrel system. Also, see the *ODOT Standard Drawings*.

11.7.3.4 Other Systems

There are several other impact attenuators available (e.g., the Connecticut Impact Attenuator System (CIAS), Hi-Dro Sandwich System). A proposed use of new or experimental systems must be coordinated with the Research and Development Division and Traffic Engineering Division. See Chapter Eighteen for additional guidance.

11.7.4 Impact Attenuator Selection

The selected impact attenuator must be compatible with the specific site characteristics. Normally, most of the operational systems will be adaptable to the site. Therefore, the designer must exercise judgment in impact attenuator selection considering safety, initial costs and maintenance.

Table 11.7A provides a summary of the advantages and disadvantages of the operational impact attenuators used by ODOT. Table 11.7B provides guidelines for impact attenuator selection based on the obstacle width and relative number of expected impacts. There are many other factors which will influence the selection of a impact attenuator for a given site, some of which are indicated in the Notes for Table 11.7B. Therefore, the designer should only use this table as a rough indicator to select a system.

The following indicates typical ODOT usage:

1. GREAT. This system is most often used at the end of a CMB, retaining wall, etc.

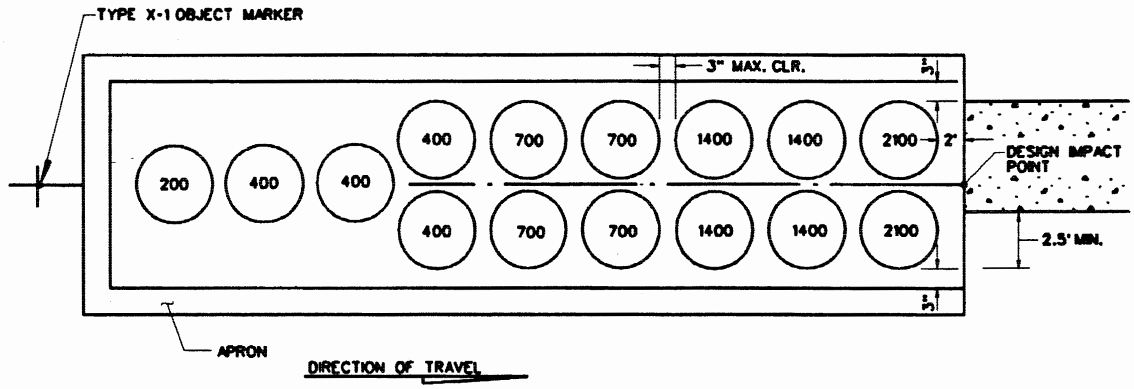
2. Sand Barrels. These are recommended for use off the highway where sand scatter and other debris will not adversely affect traffic. They are used to protect, for example, sign supports, single sign poles, bridge piers, etc.

11.7.5 Impact Attenuator Design

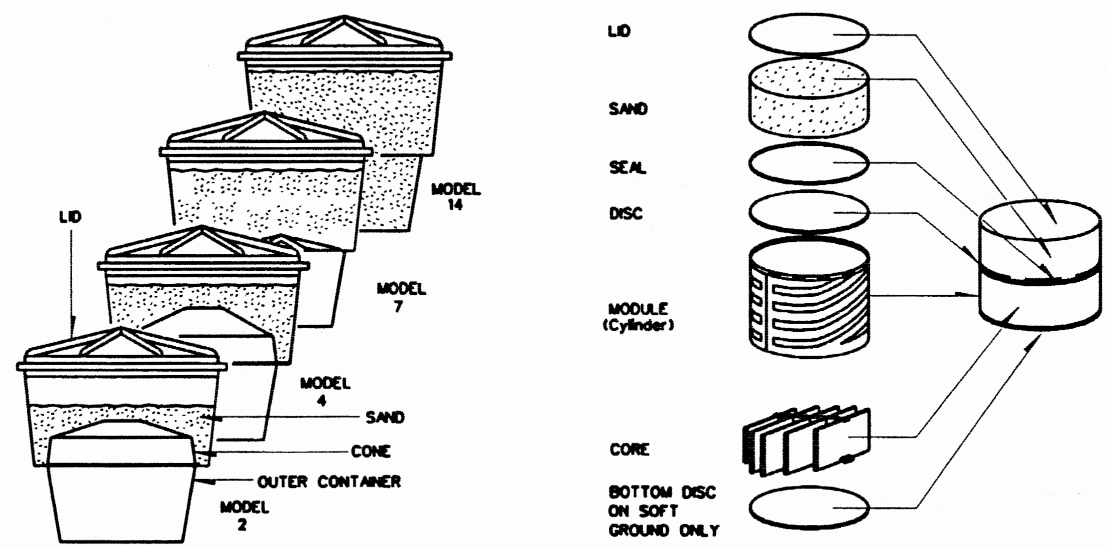
Once an impact attenuator has been selected, the designer must ensure that its design is compatible with the traffic and physical conditions at the site. The following sections will provide criteria for the basic input parameters into impact attenuator design. The designer should contact the manufacturer of the system for the detailed design of the impact attenuator.

11.7.5.1 Deceleration

For all safety appurtenances, acceptable vehicular deceleration is determined by the



NOTE: NUMBERS INDICATE WEIGHT OF SAND IN POUNDS IN EACH BARREL.
SEE ODOT STANDARD DRAWINGS FOR ADDITIONAL DETAILS.



ENERGITE III CONTAINER

FITCH CONTAINER

NOTE: "MODEL 2" CONTAINS 200 POUNDS OF SAND

Source: (12) Revised

SAND BARRELS

Figure 11.7C

Table 11.7A

IMPACT ATTENUATOR TYPES
(Advantages and Disadvantages)

SYSTEM	ADVANTAGES	DISADVANTAGES
Sand Barrels	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 and little or no "coffin corner" protection. 5. Considerable inventory of parts and space for replacements required. 6. Modules may "walk" when placed on structures.
Hex Foam Sandwich	1. Little or no debris after a hit. 2. Ease of maintenance after a hit. 3. Some attenuation capacity after a hit. 4. Relatively low maintenance cost to repair after a hit. 5. Coffin corner protection and side hit redirection.	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 3′ to 7.5′; special designs up to 15′.
GREAT	1.-5. Same as Hex Foam Sandwich. 6. Lightweight and, therefore, more suitable on structures. 7. Adaptable to 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′ to 3′.

Table 11.7B

GUIDELINES FOR IMPACT ATTENUATOR SELECTION

Width of Obstacle (W)	Relative No. of Expected Impacts	Suggested Priority of Selection	Notes
W < 3′	High	1. GREAT 2. Sand Barrels	(1), (3) (2), (4)
	Low	1. Sand Barrels 2. GREAT	(2), (4) (1), (3)
3′ ≤ W ≤ 7.5′	High	1. Hex Foam Sandwich 2. Sand Barrels	(1), (3) (2), (4)
	Low	1. Sand Barrels 2. Hex Foam Sandwich	(2), (4) (1), (3)
7.5′ < W ≤ 15′	High	1. Hex Foam Sandwich 2. Sand Barrels	(1), (3), (5) (2), (4)
	Low	1. Sand Barrels 2. Hex Foam Sandwich	(2), (4) (1), (3), (5)
W > 15′	All Sites	1. Sand Barrels	(2), (4)

Notes:

1. System requires a back-up structure, and the design of the back-up may or may not be compatible with the site conditions.

2. For rigid obstacles, system must be wider than the obstacle by at least 30 inches on either side. See Figure 11.7C.

3. System has redirective capabilities.

4. System has no redirective capabilities.

5. System requires special designs to accommodate this range of width.

occupant impact velocity as measured from full-scale crash tests. For design purposes, the maximum average deceleration level should not exceed 7.0 g's for attenuators. Wherever practical, the impact attenuator should be designed to meet this limit for deceleration. Only at very restricted sites will it be acceptable to exceed 7.0 g's.

11.7.5.2 Design Procedures

To determine the size and layout for the Hex Foam Sandwich, the GREAT System and the Sand Barrels, the initial vehicular speed must be selected. This should be the design speed with 40 mph as a minimum.

To determine the appropriate number of bays for the Hex Foam and GREAT Systems, Table 11.7C presents the design data for a Hex Foam Sandwich, and Table 11.7D presents the design data for a GREAT system. The designer should contact the manufacturer for more information. The Traffic Engineering Division will approve the final design.

The design of a sand barrel system requires an iterative process based on the conservation of momentum principle. The manufacturer of the system should be consulted for the detailed design of a sand barrel system. The final design will be prepared by the Traffic Engineering Division for sand barrels.

11.7.5.3 Side Impacts

The impact attenuator design should allow for safe side impacts. All systems used by ODOT, except the sand barrels, are designed to redirect impacting vehicles on the side.

11.7.5.4 Placement

Several factors should be considered in the placement of an impact attenuator:

1. Level terrain. All impact attenuators have been designed and tested for level conditions. Vehicular impacts on devices placed on a non-level site could result in an impact at the improper height which could produce undesirable vehicular behavior. Therefore, the attenuator should be placed on a level surface or on a cross slope not to exceed 5%.

2. Curbs. No curbs should be present on new projects at proposed impact attenuator installations. On existing highways, all curbs should be removed at proposed installations if feasible, particularly those that are 4 inches or higher.

3. Surface. A paved, bituminous or concrete pad should be provided under the impact attenuator.

4. Elevated Structures. There is some concern that the unanchored sand barrels may walk or crack due to the vibration of an elevated structure. This could adversely affect its performance.

5. Orientation. The impact attenuator should be oriented to accommodate the probable impact angle of an encroaching vehicle. This will maximize the likelihood of a head-on impact, However, this is not as important for impact attenuators with redirective capability. The proper orientation angle will depend upon the design speed, roadway alignment and lateral offset distance to the attenuator. A maximum angle of approximately 10°, as measured between the highway and impact attenuator longitudinal centerlines,

Table 11.7C

DECELERATION DATA
(Hex Foam Sandwich)

No. of Bays	Impact Velocity (mph)						
	40	45	50	55	60	65	70
4 *10'-5½"	6.0	7.6	9.3	11.4	13.5	15.9	18.4
5 12'-8½"	5.0	6.3	7.7	9.4	11.1	13.1	15.2
6 14'-11½"	4.2	5.3	6.6	8.0	9.5	11.1	12.9
7 17'-2½"	3.7	4.6	5.7	6.9	8.2	9.7	11.2
8 19'-5½"	3.2	4.1	5.1	6.1	7.3	8.5	9.9
9 21'-8½"	2.9	3.7	4.5	5.5	6.5	7.7	8.9
10 23'-11½"	2.6	3.3	4.1	5.0	5.9	6.9	8.0
11 26'-2½"	2.4	3.0	3.8	4.5	5.4	6.3	7.3
12 28'-5½"		2.8	3.5	4.2	5.0	5.8	6.8

Source: (1)

* Total length of unit as measured from front face of backup to forward edge of front cartridge.

Notes:

1. Table presents average g's for deceleration forces. Shaded area denotes possible excessive decelerations.

2. These criteria are for preliminary design purposes only. The designer should contact the manufacturer for detailed design. The Traffic Engineering Division will have final approval for design.

Table 11.7D

DECELERATION DATA
(G.R.E.A.T.)

No. of Bays (length)	Impact Velocity (mph)											
	15	20	25	30	35	40	45	50	55	60	65	70
12 (38'-9")	-	-	-	-	-	-	-	-	-	-	-	5.0
11 (35'-9")	-	-	-	-	-	-	-	-	-	-	-	5.4
10 (32'-9")	-	-	-	-	-	-	-	-	-	-	5.1	5.9
9 (29'-9")	-	-	-	-	-	-	-	-	-	4.8	5.6	6.5
8 (26'-9")	-	-	-	-	-	-	-	-	4.4	5.3	6.2	7.5
7 (23'-9")	-	-	-	-	-	-	-	4.1	5.0	6.0	7.0	8.1
6 (20'-9")	-	-	-	-	-	-	3.8	4.7	5.7	6.8	8.0	9.8
5 (17'-9")	-	-	-	-	-	3.6	4.5	5.6	6.7	8.0	9.4	10.8
4 (14'-9")	-	-	-	-	3.3	4.3	5.4	6.7	8.1	9.6	11.3	-
3 (11'-9")	-	-	-	3.0	4.1	5.4	6.8	8.4	10.1	12.0	-	-
2 (8'-9")	-	-	2.8	4.0	5.5	7.2	9.1	11.2	-	-	-	-
1 (5'-9")	1.5	2.7	4.3	6.2	8.4	10.9	-	-	-	-	-	-

Source: (1)

Notes:

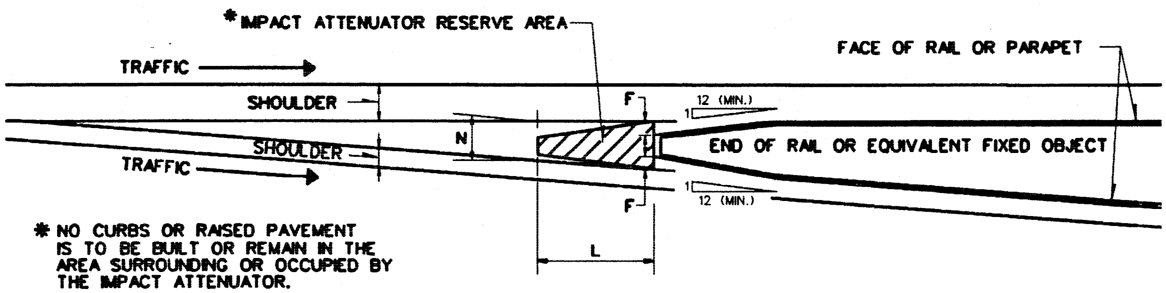
1. Table presents average g's for deceleration forces. Shaded area denotes possible excessive decelerations.

2. These criteria are for preliminary design purposes only. The designer should contact the manufacturer for detailed design. The Traffic Engineering Division will have final approval for design.

3. In most cases, the 8-bay design will be used.

is considered appropriate. See the *ODOT Standard Drawings* for more information.

6. Reserve Area. The designer should, as early as possible in the project design process, determine the need for and approximate dimensions of an impact attenuator. This will avoid late changes which could significantly affect the project design. Figure 11.7D provides recommended criteria for the impact attenuator reserve area.



Design Speed on Mainline (mph)	Dimensions for Crash Cushion Reserve Area (feet)								
	Minimum						Preferred		
	Restricted Conditions			Unrestricted Conditions					
	N	L	F	N	L	F	N	L	F
30	6	8	2	8	11	3	12	17	4
50	6	17	2	8	25	3	12	33	4
70	6	28	2	8	45	3	12	55	4
80	6	35	2	8	55	3	12	70	4

Source: (1)

Note: This figure is for preliminary design purposes only and must be used in combination with Tables 11.7C and 11.7D.

IMPACT ATTENUATOR RESERVE AREA

Figure 11.7D

11.8 REFERENCES

1. *Roadside Design Guide*, AASHTO, October 1988.

2. *Guide for Selecting, Locating, and Designing Traffic Barriers*, AASHTO, 1977.

3. *A Supplement to A Guide for Selecting, Locating, and Designing Traffic Barriers*, Texas Transportation Institute, March 1980.

4. *Safety Design and Operational Practices for Streets and Highways*, FHWA, March 1980.

5. FHWA-IP-83-4 *A Procedure for Determining Frequencies to Inspect and Repair Highway Safety Hardware*, December 1983.

6. "A Roadside Design Procedure," James Hatton, Federal Highway Administration, January 1974.

7. NCHRP 150 *Effect of Curb Geometry and Location on Vehicle Behavior*, Transportation Research Board, 1974.

8. NCHRP 158 *Selection of Safe Roadside Cross Sections*, Transportation Research Board, 1975.

9. NCHRP Synthesis 66 *Glare Screen Guidelines*, Transportation Research Board, December 1979.

10. NCHRP 230 *Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*, Transportation Research Board, March 1981.

11. *Crash Cushions -- Selection and Design Criteria*, FHWA, 1975.

12. "Crash Cushions, Safety Systems," Technical Notebook, Energy Absorption Systems, Inc.

13. *County Roads Design Guidelines Manual*, ODOT and Association of County Commissions of Oklahoma, 1991.

14. *Standard Specifications for Highway Construction*, ODOT, 1988 and Supplement.

15. *ODOT Standard Drawings*, current edition.

16. Policy Directive No. D-102-1 "Clear Zone," *ODOT Policy Manual*, ODOT, November 1987.

Chapter Twelve

GEOMETRIC DESIGN TABLES
(New Construction/Reconstruction)

Chapter Twelve

GEOMETRIC DESIGN TABLES

(New Construction/Reconstruction)

This chapter presents the Department's criteria for the geometric design of new construction and reconstruction projects. These are defined as follows:

1. New Construction. New construction is defined as horizontal and vertical alignment on new location. In addition, any intersection or interchange which falls within the project limits of a new highway mainline or is relocated to a new point of intersection is considered new construction.

2. Reconstruction. 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 facility cannot accommodate its current or future traffic demands or because the existing alignment is significantly deficient. The extent of needed pavement reconstruction should also be considered in identifying a project as reconstruction. In addition, any intersection which falls within the limits of a reconstruction project will be reconstructed as needed.

The designer should consider the following in the use of these tables:

1. Functional Classification. The selection of design values depends on the functional classification of the highway facility. This is discussed in Section 5.2.

2. Urban Design Classification. Within an urbanized or urban area, the selection of design values depends on the design classification of the facility. Separate tables are presented for "suburban" and "urban" classifications. See Section 5.2 for a description of the two classifications.

3. Rural "Suburban" Areas. Some segments of rural facilities may pass through areas which are built up or suburban in character. In these cases, the designer may use the suburban tables for application of geometric design, although the facility is categorized as rural.

4. Cross Section Elements. The designer should realize that some of the cross section elements included in a table (e.g., median width) are not automatically warranted in the project design. The values in the tables will only apply after the decision has been made to include the element in the highway cross section.

5. Manual Section References. These tables are intended to provide a concise listing of design values for easy use. They should be used by the designer already familiar with the full narrative as referenced in the tables.

6. Footnotes. The tables include many footnotes, which are identified by a number in parenthesis (e.g., (6)). The information in the footnotes is critical to the proper use of the design tables.

7. Controlling Design Criteria. The tables provide an asterisk to indicate controlling design criteria. Section 5.8 discusses this in more detail and presents the process for approving exceptions to controlling design criteria.

Table 12-1
GEOMETRIC DESIGN CRITERIA FOR FREEWAYS
(New Construction/Reconstruction)

Design Element			Manual Section	Rural	Urban			
Standard Designation			–	RF	UF			
Design Year (Geometrics)			5.3	20 years	20 years			
* Design Speed (mph) (1a)			5.2	70 (1b)	55-70			
Access Control			5.4	Full Control	Full Control			
Level of Service			5.3	Desirable: B Minimum: C	Desirable: C Minimum: D			
* Lane Width			8.1	12′	12′			
Shoulder	Type		8.1	Paved	Paved			
Shoulder	* Width (2c)		8.1	Right: 10′ (~~2a~~) Left: 4′ (~~2a~~ & 2b)	Right: 10′ (~~2a~~) Left: 4′ (~~2a~~ & 2b)			
Cross Slope	* Travel Lane		8.1	2% Typical	2% Typical			
Cross Slope	Shoulder		8.1	3% - 4% Typical	3% - 4% Typical			
Auxiliary Lanes	Lane Width		8.1	12′	12′			
Auxiliary Lanes	Shoulder Width		8.1	Desirable: Full Shoulder Width Minimum: 6′	Desirable: Full Shoulder Width Minimum: 6′			
Median Width (3)	With Barrier		8.2	N/A	4 lanes: 10′ min.; 14′ des. (3a); >4 lanes: 22′ min.			
Median Width (3)	Without Barrier		8.2	46′ Recommended; 64′ Desirable	46′ Recommended			
Right-of-Way Width			8.6	(4)	(4)			
Clear Zone			11.2	See Section 11.2	See Section 11.2			
Side Slopes	Cut (5)	Fore Slope	8.3	6:1	6:1			
Side Slopes	Cut (5)	Ditch Width	8.3	8′	8′			
Side Slopes	Cut (5)	Back Slope	8.3	4:1	4:1			
Side Slopes	Fill	0′-4′ Height	8.3	6:1	6:1			
Side Slopes	Fill	4′-10′ Height	8.3	6:1 to clear zone; 4:1 to toe	6:1 to clear zone; 4:1 to toe			
Side Slopes	Fill	>10′ Height	8.3	6:1 to clear zone; 3:1 to toe	6:1 to clear zone; 3:1 to toe			
				70 mph	55 mph	60 mph	65 mph	70 mph
* Desirable Stopping Sight Distance (6)			5.7	850′	550′	650′	725′	850′
Decision Sight Distance			5.7	See Section 5.7	See Section 5.7			
* Maximum Degree of Curvature	e_{max} = 0.06		6.1	2°45′	5°15′	4°15′	3°30′	2°45′
* Maximum Degree of Curvature	e_{max} = 0.08		6.1	3°00′	6°00′	4°45′	3°45′	3°00′
* Superelevation Rate			6.2	See Section 6.2	See Section 6.2			
* Vertical Curvature for Desirable SSD	Crest		7.2	K=540	K=220	K=310	K=400	K=540
* Vertical Curvature for Desirable SSD	Sag		7.2	K=220	K=130	K=160	K=180	K=220
* Maximum Grade (7)	Level		7.1	3%	3.5%	3%	3%	3%
* Maximum Grade (7)	Rolling		7.1	4%	4.5%	4%	4%	4%
* Maximum Grade (7)	Mountainous		7.1	5%	6%	6%	5.5%	5%
Minimum Grade			7.1	Desirable: 0.5% Minimum: 0%	Desirable: 0.5% Minimum: 0%			
* New/Reconstructed Bridges	Structural Capacity (8)		–	HS-20/OK Overload Truck	HS-20/OK Overload Truck			
* New/Reconstructed Bridges	Width		8.4	Full Approach Roadway Width	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-20 (Inventory Rating)	HS-20 (Inventory Rating)			
* Existing Bridges to Remain in Place	Width (9)		8.4	2 lanes: 28′ 3 lanes: 40′	2 lanes: 28′ 3 lanes: 40′			
* Vertical Clearance (10)	New/Replaced Bridges		7.4	16′-9″	16′-9″			
* Vertical Clearance (10)	Existing Bridges		7.4	14′-6″	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR FREEWAYS
(New Construction/Reconstruction)

Footnotes to Table 12-1

1. Design Speed. The following will apply:

   a. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.
   b. In rolling terrain, a design speed of 65 mph will be acceptable. This is allowed by the AASHTO *A Policy on Design Standards -- Interstate System*.

2. Shoulder Width. The following will apply:

   ~~a. Both Shoulders. When the volume of trucks exceeds 250 DDHV, both the right and left shoulders will be 12'.~~
   b. Left Shoulder. With three or more lanes in one direction, use a 10' shoulder.
   c. Barriers. All shoulder widths should desirably be increased by 2' when a barrier is present.

3. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions. In addition, the following will apply:

   a. Where light poles, glare screens, etc., will be mounted on the median barrier, the desirable median width is 22' - 26'.

   b. Additional median width may be necessary to meet horizontal sight distance criteria on horizontal curves. See Section 6.5.

   c. When the volume of trucks exceeds 250 DDHV, the minimum median width is 26'.

4. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

5. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts.

6. Desirable Stopping Sight Distance. Values in table are desirable SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

7. Maximum Grades. Grades 1% steeper may be used in restricted urban areas where development precludes the use of flatter grades. Grades 1% steeper may also be used for one-way downgrades at all sites except in mountainous terrain.

8. Structural Capacity (New/Reconstructed Bridges). The Alternate Military Loading will also apply to the Interstate system. The Oklahoma Overload Truck applies only to State highways.

9. Width (Existing Bridges to Remain in Place). For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

10. Minimum Vertical Clearance. The vertical clearances apply to the freeway passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17'-9" for a freeway passing beneath a new sign truss or pedestrian bridge and 17'-0" for a freeway passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a freeway. The typical clearance will be 23'-0". An allowance should also be made for future ballasting.

Table 12-2
GEOMETRIC DESIGN CRITERIA FOR RURAL PRINCIPAL ARTERIALS
(New Construction/Reconstruction)

Design Element			Manual Section	2-Lane DHV 100-200	2-Lane DHV > 200	Divided Multilane**
Standard Designation			–	RPA-1		RPA-2
Design Year (Geometrics)			5.3	20 years		20 years
* Design Speed (mph) (1)			5.2	Level: 60-70; Rolling: 60; Mountainous: 50		Level: 60-70; Rolling: 60; Mountainous: 50
Access Control			5.4	Control by Regulation		Control by Regulation/Partial Control
Level of Service			5.3	Desirable: B Minimum: C		Desirable: B Minimum: C
* Lane Width			8.1	12′		12′
Shoulder	Type		8.1	Paved		Paved
	* Width (2)		8.1	6′	8′	Right: Desirable: 10′, Min: 8′; Left: 4′(2a)
Cross Slope	* Travel Lane		8.1	2% Typical		2% Typical
	Shoulder		8.1	2%-4% Typical		2%-4% Typical
Auxiliary Lanes	Lane Width		8.1	12′		12′
	Shoulder Width		8.1	4′ min. (paved)		4′ min. (paved)
Median Width (3)			8.2	NA		46′ Recommended; 64′ Desirable
Right-of-Way Width			8.6	(4)		(4)
Clear Zone			11.2	See Section 11.2		See Section 11.2
Side Slopes	Cut (5)	Fore Slope	8.3	6:1		6:1
		Ditch Width	8.3	8′		8′
		Back Slope	8.3	4:1		4:1
	Fill	0′-4′ Height	8.3	6:1		6:1
		4′-10′ Height	8.3	6:1 to clear zone; 4:1 to toe		6:1 to clear zone; 4:1 to toe
		> 10′ Height	8.3	6:1 to clear zone; 3:1 to toe		6:1 to clear zone; 3:1 to toe

Design Element		Manual Section	50 mph	60 mph	70 mph
* Desirable Stopping Sight Distance (6)		5.7	475′	650′	850′
Intersection Sight Distance		9.2	See Section 9.2		
Decision Sight Distance		5.7	See Section 5.7		
Passing Sight Distance		5.7	1800′	2100′	2500′
* Maximum Degree of Curvature	$e_{max}=0.06$	6.1	6°45′	4°15′	2°45′
	$e_{max}=0.08$	6.1	7°30′	4°45′	3°00′
* Superelevation Rate		6.2	See Section 6.2		
* Vertical Curvature for Desirable SSD	Crest	7.2	K=160	K=310	K=540
	Sag	7.2	K=110	K=160	K=220
* Maximum Grade (7)	Level	7.1	4%	3%	3%
	Rolling	7.1	5%	4%	4%
	Mountainous	7.1	7%	6%	5%
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%		
* New/Reconstructed Bridges	Structural Capacity (8)	–	HS-20/OK Overload Truck		
	Width	8.4	Full Approach Roadway Width		
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)		
	Width (9)	8.4	Approach Travelway + 4′ (28′ min.)		
* Vertical Clearance (10)	New/Replaced Bridges	7.4	16′-9″		
	Existing Bridges	7.4	14′-6″		

* Controlling design criteria (See Section 5.8)

** See Table 12-6 "Suburban Principal Arterials" for multilane undivided design criteria for Rural Principal Arterials.

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

Footnotes to Table 12-2

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Shoulder Width. The following will apply:

   a. Where there are three or more lanes in one direction, the left shoulder should be 8'.
   b. All shoulder widths should desirably be increased by 2' when a barrier is present.

3. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions.

4. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

5. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts.

6. Desirable Stopping Sight Distance. Values in table are desirable SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

7. Maximum Grades. Grades 1% steeper may be used for one-way downgrades at all sites except in mountainous terrain.

8. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

9. Width (Existing Bridges to Remain in Place). For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

10. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17'-9" for an arterial passing beneath a new sign truss or pedestrian bridge and 17'-0" for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23'-0". An allowance should also be made for future ballasting.

Table 12-3
GEOMETRIC DESIGN CRITERIA FOR RURAL OTHER ARTERIALS
(New Construction/Reconstruction)

Design Element			Manual Section	2-Lane DHV<100	2-Lane DHV 100-200	2-Lane DHV>200	Divided Multilane**
Standard Designation			–	ROA-1			ROA-2
Design Year (Geometrics)			5.3	20 years			20 years
* Design Speed (mph) (1)			5.2	Level: 60-70; Rolling: 50-60; Mountainous: 40-50			Level: 60-70; Rolling: 50-60; Mountainous: 40-50
Access Control			5.4	Control by Regulation			Control by Regulation
Level of Service			5.3	Desirable: B Minimum: C			Desirable: B Minimum: C
* Lane Width (2)			8.1	12'			12'
Shoulder	Type		8.1	2' Paved/Remainder Sod		Paved	Paved
	* Width (3)		8.1	4'	6'	8'	Right: 8' Left: 3' min. (3a)
Cross Slope	* Travel Lane		8.1	2% Typical			2% Typical
	Shoulder		8.1	2% - 4% Typical			2% - 4% Typical
Auxiliary Lanes	Lane Width		8.1	Desirable: Same as Travel Lane; Minimum: 11'			12'
	Shoulder Width		8.1	2' Paved/2' Sod		4' Paved	Right: 4' Paved Left: 3' Paved
Median Width (4)			8.2	N/A			46' Recommended; 64' Desirable
Right-of-Way Width			8.6	(5)			(5)
Clear Zone			29.711.2	See Section 11.2			See Section 11.2
Side Slopes	Cut (6)	Fore Slope	8.3	V>50: 6:1 V≤50: 4:1			V>50: 6:1 V≤50: 4:1
		Ditch Width	8.3	8'			8'
		Back Slope	8.3	V>50: 4:1 V≤50: 3:1			V>50: 4:1 V≤50: 3:1
	Fill	0'-4' Height	8.3	Desirable: 6:1 Maximum: 4:1			Desirable: 6:1 Maximum: 4:1
		4'-10' Height	8.3	4:1			4:1
		>10' Height	8.3	4:1 to clear zone; 3:1 to toe			4:1 to clear zone; 3:1 to toe

Design Element		Manual Section	40 mph	50 mph	60 mph	70 mph
* Minimum Stopping Sight Distance (7)		5.7	275'	400'	525'	625'
Intersection Sight Distance		9.2	See Section 9.2			
Decision Sight Distance		5.7	See Section 5.7			
Passing Sight Distance		5.7	1500'	1800'	2100'	2500'
* Maximum Degree of Curvature	e_{max} = 0.06	6.1	11°15'	6°45'	4°15'	2°45'
	e_{max} = 0.08	6.1	12°15'	7°30'	4°45'	3°00'
* Superelevation Rate		6.2	See Section 6.2			
* Vertical Curvature for Minimum SSD	Crest	7.2	K=60	K=110	K=190	K=290
	Sag	7.2	K=60	K=90	K=120	K=150
* Maximum Grade (8)	Level	7.1	5%	4%	3%	3%
	Rolling	7.1	6%	5%	4%	4%
	Mountainous	7.1	8%	7%	6%	5%
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%			
* New/Reconstructed Bridges	Structural Capacity (9)	–	HS-20/OK Overload Truck			
	Width	8.4	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)			
	Width (10)	8.4	Approach Travelway + 4' (28' min.)			
* Vertical Clearance (11)	New/Replaced Bridges	7.4	16'-9"			
	Existing Bridges	7.4	14'-6"			

* Controlling design criteria (See Section 5.8)

** See Table 12-8 "Suburban Other Arterials" for multilane undivided design criteria for Rural Other Arterials.

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

Footnotes to Table 12-3

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Width. The following will apply:

   a. For a 50-mph design speed and for DHV<100, lane widths may be 11′ on 2-lane highways.

   b. Existing 11′ lanes on all reconstructed rural other arterials may be retained if alignment and safety record are satisfactory.

3. Shoulder Width. The following will apply:

   a. Where there are three or more lanes in one direction, the left shoulder should be 8′.
   b. All shoulder widths should desirably be increased by 2′ when a barrier is present.

4. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions.

5. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

6. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts.

7. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

8. Maximum Grades. Grades 1% steeper may be used for one-way downgrades at all sites except in mountainous terrain.

9. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

10. Width (Existing Bridges to Remain in Place). For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

11. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17′-9″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-4
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(New Construction/Reconstruction)

Design Element			Manual Section	State Highways AADT <400	State Highways DHV 100-200	State Highways DHV 200-400	State Highways DHV >400	Non-State Highways** AADT <400	Non-State Highways** DHV 100-200	Non-State Highways** DHV 200-400	Non-State Highways** DHV >400
Standard Designation			–	RCS				RCNS			
Design Year (Geometrics) (1)			5.3	20 years				20 years			
* Design Speed (mph) (2)	Level		5.2	45	50	60		40	50	60	
	Rolling		5.2	45	45	50		30	40	50	
	Mountainous		5.2	40	40	40		20	30	40	
Access Control			5.4	Control by Regulation				Control by Regulation			
Level of Service			5.3	Level/Rolling: C Mountainous: D				Level/Rolling: C Mountainous: D			
Lane Width	Type		8.1	Paved				Gravel/Paved			
	* Width		8.1	11′ (3a)		12′ (3c)	12′	10′	11′ (3b)	12′ (3c)	12′
Shoulder (4)	Type		8.1	2′ Pvd/ 2′ Sod	2′ Pvd/ 4′ Sod	4′ Paved/ 4′ Sod		Gravel/Sod			
	* Width		8.1	4′	6′	8′		2′ (4a)	6′ (4b)	8′	
Cross Slope	* Travel Lane		8.1	2% Typical				Paved: 2% Gravel: 2%-4%			
	Shoulder		8.1	2%-4% Typical				Gravel: 4%-6% Sod: 6%-8%			
Auxiliary Lanes	Lane Width		8.1	Des: 11′ Min: 10′		Des: 12′ Min: 11′		NA			
	Shoulder Width		8.1	Des: 4′ Min.: 2′				NA			
Right-of-Way Width			8.6	(5)				(5)			
Clear Zone			11.2	See Section 11.2				See Section 11.2			
Side Slopes	Cut (6)	Fore Slope	8.3	Desirable: 6:1 Maximum: 4:1				Desirable: 4:1 Maximum: 3:1			
		Ditch Width	8.3	Desirable: 8′ Minimum: 4′				Desirable: 4′ Minimum: 2′			
		Back Slope	8.3	3:1				Desirable: 3:1 Maximum: 2:1			
	Fill	0′-4′ Height	8.3	4:1				Desirable: 4:1 Maximum: 3:1			
		4′-10′ Height	8.3	Desirable: 4:1 Maximum: 3:1				3:1			
		> 10′ Height	8.3	3:1				3:1			

Design Element		Manual Section	20 mph	30 mph	40 mph	50 mph	60 mph
* Minimum Stopping Sight Distance (7)		5.7	125′	200′	275′	400′	525′
Intersection Sight Distance		9.2	See Section 9.2				
Decision Sight Distance		5.7	See Section 5.7				
Passing Sight Distance			NA	1100′	1500′	1800′	2100′
* Maximum Degree of Curvature	emax = 0.06	6.1	49°15′	21°00′	11°15′	6°45′	4°15′
	emax = 0.08	6.1	53°30′	22°45′	12°15′	7°30′	4°45′
* Superelevation Rate		6.2	See Section 6.2				
* Vertical Curvature for Minimum SSD	Crest	7.2	K=10	K=30	K=60	K=110	K=190
	Sag	7.2	K=20	K=40	K=60	K=90	K=120
* Maximum Grade (8)	Level	7.1	7%	7%	7%	6%	5%
	Rolling	7.1	10%	9%	8%	7%	6%
	Mountainous	7.1	12%	10%	10%	9%	8%
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%				

Design Element		Manual Section	State <400	State 100-200	State 200-400	State >400	Non-State <400	Non-State 100-200	Non-State 200-400	Non-State >400
* New/Reconstructed Bridges	Structural Capacity	–	HS-20/OK Overload Truck				HS-20			
	Width	8.4	TW + 8′		App. Roadway Width		TW + 4′	TW + 6′	TW + 8′	APR
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-15 (Inventory Rating)				HS-15			
	Width (9)	8.4	24′		26′	28′	22′		24′	28′
* Vertical Clearance (10)	New/Replaced Bridges	7.4	16′-9″				16′-9″			
	Existing Bridges	7.4	14′-6″				14′-0″			

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

Footnotes to Table 12-4

1. Design Year. If the DHV is less than 100 (based on a 20-year projection), then the current AADT may be used for design.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For a 26′ paved roadway, pavement striping will be for two 11′ lanes.
   b. For DHV 100-200, the lane width may be 10′ where the design speed is 30 mph.
   c. For DHV 200-400, the lane width may be 11′ where the design speed is 40 mph.

4. Shoulder Width. All shoulder widths should desirably be increased by 2′ when a barrier is present. The following will also apply:

   a. If guardrail is present, the minimum shoulder width is 4′.
   b. For current AADT>400 and (20-year) DHV<100, the shoulder width may be 4′ minimum.

5. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths plus the outside shoulder widths plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will also accommodate utility requirements.

6. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts.

7. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

8. Maximum Grades. For grades less than 500′ in length (PVT to PVC), the maximum grade may be 2% steeper than table value. For roads with less than 400 current AADT, the maximum grade may be 2% steeper than table value.

9. Width (Existing Bridges to Remain in Place). For bridges on the State highway system and where the width is wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

   For bridges not on the State highway system and where the bridge is more than 100′ in length, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

10. Minimum Vertical Clearance. The vertical clearances apply to the collector passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a collector. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-5
GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROADS**
(New Construction/Reconstruction)

Design Element			Manual Section	AADT<50	AADT 50-250	AADT 250-400	DHV 100-200	DHV 200-400	DHV>400
Standard Designation			–	RL					
Design Year (Geometrics)			5.3	20 years (1)					
* Design Speed (mph) (2)	Level		5.2	30	30	40	50		
	Rolling		5.2	20	30	30	40		
	Mountainous		5.2	20	20	20	30		
Access Control			5.4	Control by Regulation					
Level of Service			5.3	D					
* Travel Lanes	Type		8.1	Paved, Chip Seal, Gravel or Dirt					
	Width		8.1	V≤30: 9' V>30: 10'		10'	10' (3a)	11' (3b)	12'
Shoulder	Type		8.1	Gravel or Sod					
	* Width (4a)		8.1	2'			6' (4b)		8'
Cross Slope	* Travel Lane		8.1	Chip Seal: 2% Gravel: 2%-4% Dirt: 4%-6%					
	Shoulder		8.1	Gravel: 4%-6% Sod: 4%-8%					
Auxiliary Lanes			8.1	NA					
Right-of-Way Width			8.6	(5)					
* Clear Zone			11.2	See Section 11.2					
Side Slopes	Cut (6)	Fore Slope	8.3	3:1					
		Ditch Width	8.3	As Required for Drainage					
		Back Slope	8.3	Desirable: 3:1 Maximum: 2:1					
	Fill	0'-4' Height	8.3	Desirable: 4:1 Maximum: 3:1					
		4'-10' Height	8.3	3:1					
		> 10' Height	8.3	3:1					

Design Element		Manual Section	20 mph	30 mph	40 mph	50 mph
* Minimum Stopping Sight Distance (7)		5.7	125'	200'	275'	400'
Intersection Sight Distance		9.2	See Section 9.2			
Decision Sight Distance		5.7	NA	450'	600'	750'
Passing Sight Distance		5.7	NA	1100'	1500'	1800'
* Maximum Degree of Curvature	emax=0.06	6.1	49°15'	21°00'	11°15'	6°45'
	emax=0.08	6.1	53°30'	22°45'	12°15'	7°30'
* Superelevation Rate		6.2	See Section 6.2			
* Vertical Curvature for Minimum SSD	Crest	7.2	K=10	K=30	K=60	K=110
	Sag	7.2	K=20	K=40	K=60	K=90
* Maximum Grade	Level	7.1	–	7%	7%	6%
	Rolling	7.1	11%	10%	9%	8%
	Mountainous	7.1	16%	14%	12%	10%
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%			

Design Element		Manual Section	<50	50-250	250-400	100-200	200-400	>400
* New/Reconstructed Bridges	Structural Capacity	–	HS-20					
	Width (8)	8.4	Travelway + 4'			Travelway + 6'		App. Rd. Wid.
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-10	HS-15				
	Width (9)	8.4	20'		22'		24'	28'
* Vertical Clearance (10)	New/Replaced Bridges	7.4	16'-9"					
	Existing Bridges	7.4	14'-0"					

* Controlling design criteria (See Section 5.9)

** Table will only apply where State and/or Federal funds are used.

GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROADS (New Construction/Reconstruction)

Footnotes to Table 12-5

1. Design Year. If the DHV is less than 100 (based on a 20-year projection), then the current AADT may be used for design.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For DHV 100-200, use 11′ lanes where V ≥ 40 mph.
   b. For DHV 200-400, use 12′ lanes where V ≥ 50 mph.

4. Shoulder Width. The following will apply:

   a. All shoulder widths refer to the graded width, which is the distance between the edge of travel lane and the point of intersection of the shoulder slope and side slope.

   b. For AADT>400 but DHV<100, the shoulder width may be 4′.

5. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths plus the outside shoulder widths plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will also accommodate utility requirements.

6. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts.

7. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

8. Width (New and Reconstructed Bridges). Widths of bridges more than 100′ in length will be analyzed individually.

9. Width (Existing Bridges to Remain in Place). Minimum clear widths that are 2′ narrower may be used on roads with few trucks. However, the clear roadway width should be at least the same width as the approach travelway. For one-lane bridges, the width can be 18′. For bridges of more than 100′ in length, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

10. Minimum Vertical Clearance. The vertical clearances apply to the local road passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a local road. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-6
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN PRINCIPAL ARTERIALS
(New Construction/Reconstruction)

Design Element			Manual Section	2-Lane With Curb	2-Lane Without Curb	Multilane With Curb	Multilane Without Curb
Standard Designation			–	SPA-1	SPA-2	SPA-3	SPA-4
Design Year (Geometrics)			5.3	20 years		20 years	
* Design Speed (mph) (1)			5.2	45-60		45-60	
Access Control			5.4	Control by Regulation		Control by Regulation/Partial Control	
Level of Service			5.3	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width			8.1	12′		12′	
Shoulder/Curb Offset	Type		8.1	Paved		Paved	
	* Width		8.1	2′ Min. (2)	8′	Right: 2′ Left: 2′ (2)	Right: 8′ Left: 4′
Cross Slope	* Travel Lane		8.1	2% Typical		2% Typical (3)	
	Shoulder/Curb Offset		8.1	Same as adjacent T.L.	2%-4% Typical	Same as adjacent T.L.	2%-4% Typical
Auxiliary Lanes	Lane Width		8.1	12′		12′	
	Shoulder/Curb Offset		8.1	2′ Min. (4)	4′ Min.	2′ Min. (4)	4′ Min.
TWLT Lane Width			9.4	NA		14′ (5)	
Parking Lane Width (6)			17.1	Desirable: 12′ Minimum: 11′		Desirable: 12′ Minimum: 11′	
Median Width			8.2	NA		(7)	
Right-of-Way Width			8.6	(8)		(8)	
Clear Zone			11.2	1.5′ (9)	See Section 11.2	1.5′ (9)	See Section 11.2
Side Slopes	Cut (10)	Fore Slope	8.3	NA	4:1	NA	4:1
		Ditch Width	8.3	NA	8′	NA	8′
		Back Slope	8.3	3:1	3:1	3:1	3:1
	Fill (11)	0′-4′ Height	8.3	3:1	3:1	3:1	3:1
		4′-10′ Height	8.3	3:1	3:1	3:1	3:1
		> 10′ Height	8.3	3:1	3:1	3:1	3:1

Design Element		Manual Section	45 mph	50 mph	60 mph
* Desirable Stopping Sight Distance (12)		5.7	400′	475′	650′
Intersection Sight Distance		9.2	See Section 9.2		
Decision Sight Distance		5.7	See Section 5.7		
* Maximum Degree of Curvature	emax=0.04	6.1	8°30′	NA	NA
	emax=0.06	6.1	9°15′	6°45′	4°15′
* Superelevation Rate		6.2	See Section 6.2		
* Vertical Curvature for Desirable SSD	Crest	7.2	K=120	K=160	K=310
	Sag	7.2	K=90	K=110	K=160
* Maximum Grade	Level	7.1	6.5%	6%	5%
	Rolling	7.1	7.5%	7%	6%
	Mountainous	7.1	9.5%	9%	8%
Minimum Grade		7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%		
* New/Reconstructed Bridges	Structural Capacity (13)	–	HS-20/OK Overload Truck		
	Width	8.4	Full Approach Roadway Width		
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)		
	Width (14)	8.4	Uncurbed Sections: Travelway + 4′ Curbed Sections: Site Specific		
* Vertical Clearance (15)	New/Replaced Bridges	7.4	16′-9″		
	Existing Bridges	7.4	14′-6″		

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN PRINCIPAL ARTERIALS
(New Construction/Reconstruction)

Footnotes to Table 12-6

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

3. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

4. Auxiliary Lane Shoulder/Curb Offset. Widths adjacent to auxiliary lanes will typically be 4′ or equal to the width adjacent to the travel lane, whichever is less.

5. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

6. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane will be the same as that of the adjacent travel lane.

7. Median Width. The median width will depend upon many factors. These include the type of median (depressed, flush or raised), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions. In addition, the following will apply where a median barrier is warranted:

   a. Where light poles, glare screens, etc., will be mounted on a median barrier, the desirable median width is 22′ - 26′.

   b. Additional median width may be necessary to meet horizontal sight distance criteria on horizontal curves. See Section 6.5.

8. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

9. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

10. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

11. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

12. Desirable Stopping Sight Distance. Values in table are desirable SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

13. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

14. Width (Existing Bridges to Remain in Place). On State highways, the minimum width is 28′ for uncurbed sections. On all facilities, for bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

15. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

   a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17′-9″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

   b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-7
GEOMETRIC DESIGN CRITERIA FOR URBAN PRINCIPAL ARTERIALS
(New Construction/Reconstruction)

Design Element			Manual Section	2-Lane: With Curb	2-Lane: Without Curb	Multilane: With Curb	Multilane: Without Curb
Standard Designation			–	UPA-1	UPA-2	UPA-3	UPA-4
Design Year (Geometrics)			5.3	20 years		20 years	
* Design Speed (mph) (1)			5.2	40-50		40-60	
Access Control			5.4	Control by Regulation		Control by Regulation/Partial Control	
Level of Service			5.3	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width (2)			8.1	Desirable: 12′ Minimum: 11′		Desirable: 12′ Minimum: 11′	
Shoulder/Curb Offset	Type		8.1	Paved		Paved	
Shoulder/Curb Offset	* Width		8.1	2′ Min. (3a)	Des.: 8′ Min: 6′	Right: 2′ Left: 2′ (3a)	Right: 6′(3b) Left: 4′
Cross Slope	* Travel Lane		8.1	2% Typical		2% Typical (4)	
Cross Slope	Shoulder/Curb Offset		8.1	Same as adjacent T.L.	2%-4% Typical	Same as adjacent T.L.	2%-4% Typical
Auxiliary Lanes	Lane Width		8.1	Desirable: Same as Travel Lane; Minimum: 11′		Desirable: Same as Travel Lane; Minimum: 11′	
Auxiliary Lanes	Shoulder/Curb Offset		8.1	2′ Min. (5)	4′ Min.	2′ Min. (5)	4′ Min.
TWLT Lane Width				NA		14′ (6)	
Parking Lane Width (7)				Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
Median Width			8.2	NA		(8)	
Right-of-Way Width			8.6	(9)		(9)	
Clear Zone			11.2	1.5′ (10)	See Section 11.2	1.5′ (10)	See Section 11.2
Side Slopes	Cut (11)	Fore Slope	8.3	NA	4:1	NA	4:1
Side Slopes	Cut (11)	Ditch Width	8.3	NA	8′	NA	8′
Side Slopes	Cut (11)	Back Slope	8.3	3:1	3:1	3:1	3:1
Side Slopes	Fill (12)	0′-4′ Height	8.3	3:1	3:1	3:1	3:1
Side Slopes	Fill (12)	4′-10′ Height	8.3	3:1	3:1	3:1	3:1
Side Slopes	Fill (12)	> 10′ Height	8.3	3:1	3:1	3:1	3:1
* Desirable Stopping Sight Distance (13)			5.7	40 mph	45 mph	50 mph	60 mph
				325′	400′	475′	650′
Intersection Sight Distance			9.2	See Section 9.2			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		6.1	11°30′	8°30′	NA	NA
* Maximum Degree of Curvature	emax=0.06		6.1	12°45′	9°15′	6°45′	4°15′
* Superelevation Rate			6.2	See Section 6.2			
* Vertical Curvature for Desirable SSD	Crest		7.2	K=80	K=120	K=160	K=310
* Vertical Curvature for Desirable SSD	Sag		7.2	K=70	K=90	K=110	K=160
* Maximum Grade	Level		7.1	7%	6.5%	6%	5%
* Maximum Grade	Rolling		7.1	8%	7.5%	7%	6%
* Maximum Grade	Mountainous		7.1	10%	9.5%	9%	8%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%			
* New/Reconstructed Bridges	Structural Capacity (14)		–	HS-20/OK Overload Truck			
* New/Reconstructed Bridges	Width		8.4	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-20 (Inventory Rating)			
* Existing Bridges to Remain in Place	Width (15)		8.4	Uncurbed Section: Travelway + 4′ Curbed Section: Site Specific			
* Vertical Clearance (16)	New/Replaced Bridges		7.4	16′-9″			
* Vertical Clearance (16)	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR URBAN PRINCIPAL ARTERIALS
(New Construction/Reconstruction)

Footnotes to Table 12-7

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. Where truck volume ≥10%, lane widths should be 12′.

3. Shoulder/Curb Offset. The following will apply:

   a. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

   b. Where partial control of access is used, the right shoulder width will be 8′.

4. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

5. Auxiliary Lane Shoulder/Curb Offset. Widths adjacent to auxiliary lanes will typically be 4′ or equal to the width adjacent to the travel lane, whichever is less.

6. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

7. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane will be the same as that of the adjacent travel lane.

8. Median Width. The median width will depend upon many factors. These include the type of median (depressed, flush or raised), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions. In addition, the following will apply where a median barrier is warranted:

   a. Where light poles, glare screens, etc., will be mounted on a median barrier, the desirable median width is 22′ - 26′.

   b. Additional median width may be necessary to meet horizontal sight distance criteria on horizontal curves. See Section 6.5.

9. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

10. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

11. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

12. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

13. Desirable Stopping Sight Distance. Values in table are desirable SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

14. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

15. Width (Existing Bridges to Remain in Place). On State highways, the minimum width is 28′ for uncurbed sections. On all facilities, for bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

16. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17'-9" for an arterial passing beneath a new sign truss or pedestrian bridge and 17'-0" for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23'-0". An allowance should also be made for future ballasting.

Table 12-8
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN OTHER ARTERIALS
(New Construction/Reconstruction)

Design Element		Manual Section	2-Lane		Multilane	
			With Curb	Without Curb	With Curb	Without Curb
Standard Designation		–	SOA-1	SOA-2	SOA-3	SOA-4
Design Year (Geometrics)		5.3	20 years		20 years	
* Design Speed (mph) (1)		5.2	40-60		40-60	
Access Control		5.4	Control by Regulation		Control by Regulation	
Level of Service		5.3	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width (2)		8.1	Desirable: 12′ Minimum: 11′		Desirable: 12′ Minimum: 11′	
Shoulder/Curb Offset	Type	8.1	Paved		Paved	
	* Width	8.1	2′ Min. (3)	6′	Right: 2′ Left: 2′ (3)	Right: 6′(2) Left: 4′
Cross Slope	* Travel Lane	8.1	2% Typical		2% Typical (4)	
	Shoulder/Curb Offset	8.1	Same as adjacent T.L.	2%-4% Typical	Same as adjacent T.L.	2%-4% Typical
Auxiliary Lanes	Lane Width	8.1	Desirable: Same as Travel Lane; Minimum: 11′		Desirable: Same as Travel Lane; Minimum: 11′	
	Shoulder/Curb Offset	8.1	2′ Min. (5)	4′ Min.	2′ Min. (5)	4′ Min.
TWLT Lane Width		9.4	NA		14′ (6)	
Parking Lane Width (7)		17.1	Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
Median Width		8.2	NA		(8)	
Right-of-Way Width		8.6	(9)		(9)	
Clear Zone		11.2	1.5′ (10)	See Section 11.2	1.5′ (10)	See Section 11.2
Side Slopes	Cut (11) Fore Slope	8.3	NA	4:1	NA	4:1
	Cut (11) Ditch Width	8.3	NA	8′	NA	8′
	Cut (11) Back Slope	8.3	3:1	3:1	3:1	3:1
	Fill (12) 0′-4′ Height	8.3	3:1	3:1	3:1	3:1
	Fill (12) 4′-10′ Height	8.3	3:1	3:1	3:1	3:1
	Fill (12) > 10′ Height	8.3	3:1	3:1	3:1	3:1
* Minimum Stopping Sight Distance (13)			40 mph	45 mph	50 mph	60 mph
		5.7	275′	325′	400′	525′
Intersection Sight Distance		9.2	See Section 9.2			
Decision Sight Distance		5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04	6.1	11°30′	8°30′	NA	NA
	emax=0.06	6.1	12°45′	9°15′	6°45′	4°15′
* Superelevation Rate		6.2	See Section 6.2			
* Vertical Curvature for Minimum SSD	Crest	7.2	K=60	K=80	K=110	K=190
	Sag	7.2	K=60	K=70	K=90	K=120
* Maximum Grade	Level	7.1	7%	6.5%	6%	5%
	Rolling	7.1	8%	7.5%	7%	6%
	Mountainous	7.1	10%	9.5%	9%	8%
Minimum Grade		7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%			
* New/Reconstructed Bridges	Structural Capacity (14)	–	HS-20/OK Overload Truck			
	Width	8.4	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)			
	Width (15)	8.4	Uncurbed Section: Travelway + 4′ Curbed Section: Site Specific			
* Vertical Clearance (16)	New/Replaced Bridges	7.4	16′-9″			
	Existing Bridges	7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN OTHER ARTERIALS
(New Construction/Reconstruction)

Footnotes to Table 12-8

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. Where truck volume ≥10%, lane widths should be 12′.

3. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

4. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

5. Auxiliary Lane Shoulder/Curb Offset. Widths adjacent to auxiliary lanes will typically be 4′ or equal to the width adjacent to the travel lane, whichever is less. For left-turn lanes in flush medians, the shoulder width adjacent to the turn lane may be zero.

6. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

7. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane will be the same as that of the adjacent travel lane.

8. Median Width. The median width will depend upon many factors. These include the type of median (depressed, flush or raised), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions. In addition, the following will apply where a median barrier is warranted:

   a. Where light poles, glare screens, etc., will be mounted on a median barrier, the desirable median width is 22′ - 26′.

   b. Additional median width may be necessary to meet horizontal sight distance criteria on horizontal curves. See Section 6.5.

9. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

10. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

11. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

12. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

13. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

14. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

15. Width (Existing Bridges to Remain in Place). On State highways, the minimum width is 28′ for uncurbed sections. On all facilities, for bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

16. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17'-9" for an arterial passing beneath a new sign truss or pedestrian bridge and 17'-0" for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23'-0". An allowance should also be made for future ballasting.

Table 12-9
GEOMETRIC DESIGN CRITERIA FOR URBAN OTHER ARTERIALS
(New Construction/Reconstruction)

Design Element			Manual Section	2-Lane		Multilane	
				With Curb	Without Curb	With Curb	Without Curb
Standard Designation			--	UOA-1	UOA-2	UOA-3	UOA-4
Design Year (Geometrics)			5.3	20 years		20 years	
* Design Speed (mph) (1)			5.2	30-50		30-50	
Access Control			5.4	Control by Regulation		Control by Regulation	
Level of Service			5.3	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width (2)			8.1	Desirable: 12′ Minimum: 11′		Desirable: 12′ Minimum: 11′	
Shoulder/Curb Offset	Type		8.1	Paved		Paved	
	* Width		8.1	2′ Min. (3)	Des.: 6′ Min.: 4′	Right: 2′ Left: 2′ (3)	Right: 6′(3) Left: 3′
Cross Slope	* Travel Lane		8.1	2% Typical		2% Typical (4)	
	Shoulder/Curb Offset		8.1	Same as adjacent T.L.	2%-4% Typical	Same as adjacent T.L.	2%-4% Typical
Auxiliary Lanes	Lane Width		8.1	Desirable: Same as Travel Lane; Minimum: 11′		Desirable: Same as Travel Lane; Minimum: 11′	
	Shoulder/Curb Offset		8.1	2′ Min. (5)	4′ Min.	2′ Min. (5)	4′ Min.
TWLT Lane Width			9.4	NA		14′ (6)	
Parking Lane Width (7)			17.1	Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
Median Width			8.2	NA		(8)	
Right-of-Way Width			8.6	(9)		(9)	
Clear Zone			11.2	1.5′ (10)	See Section 11.2	1.5′ (10)	See Section 11.2
Side Slopes	Cut (11)	Fore Slope	8.3	NA	4:1	NA	4:1
		Ditch Width	8.3	NA	8′	NA	8′
		Back Slope	8.3	3:1	3:1	3:1	3:1
	Fill (12)	0′-4′ Height	8.3	3:1	3:1	3:1	3:1
		4′-10′ Height	8.3	3:1	3:1	3:1	3:1
		> 10′ Height	8.3	3:1	3:1	3:1	3:1
				30 mph	40 mph	45 mph	50 mph
* Minimum Stopping Sight Distance (13)			5.7	200′	275′	325′	400′
Intersection Sight Distance			9.2	See Section 9.2			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		6.1	24°45′	11°30′	8°30′	NA
	emax=0.06		6.1	26°45′	12°45′	9°15′	6°45′
* Superelevation Rate			6.2	See Section 6.2			
* Vertical Curvature for Minimum SSD	Crest		7.2	K=30	K=60	K=80	K=110
	Sag		7.2	K=40	K=60	K=70	K=90
* Maximum Grade	Level		7.1	8%	7%	6.5%	6%
	Rolling		7.1	9%	8%	7.5%	7%
	Mountainous		7.1	11%	10%	9.5%	9%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%			
* New/Reconstructed Bridges	Structural Capacity (14)		--	HS-20/Oklahoma Overload Truck			
	Width		8.4	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity		--	HS-20 (Inventory Rating)			
	Width (15)		8.4	Uncurbed Section: Travelway + 4′ Curbed Section: Site Specific			
* Vertical Clearance (16)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR URBAN OTHER ARTERIALS
(New Construction/Reconstruction)

Footnotes to Table 12-9

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. Where truck volume ≥10%, lane widths should be 12′.

3. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

4. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

5. Auxiliary Lane Shoulder/Curb Offset. Widths adjacent to auxiliary lanes will typically be 4′ or equal to the width adjacent to the travel lane, whichever is less. For left-turn lanes in flush medians, the shoulder width adjacent to the turn lane may be zero.

6. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

7. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane will be the same as that of the adjacent travel lane.

8. Median Width. The median width will depend upon many factors. These include the type of median (depressed, flush or raised), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes) and field conditions. In addition, the following will apply where a median barrier is warranted:

   a. Where light poles, glare screens, etc., will be mounted on a median barrier, the desirable median width is 22′ - 26′.

   b. Additional median width may be necessary to meet horizontal sight distance criteria on horizontal curves. See Section 6.5.

9. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the median width plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus utility requirements.

10. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

11. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

12. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

13. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

14. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

15. Width (Existing Bridges to Remain in Place). On State highways, the minimum width is 28′ for uncurbed sections. On all facilities, for bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or the approach roadway width is widened.

16. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 17′-9″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-10
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN COLLECTORS
(New Construction/Reconstruction)

Design Element			Manual Section	With Curb		Without Curb	
Standard Designation			–	SC-1		SC-2	
Design Year (Geometrics)			5.3	Desirable: 20 years Minimum: 10 years			
* Design Speed (mph) (1)			5.2	30-50			
Access Control			5.4	Control by Regulation			
Level of Service			5.3	D			
* Lane Width (2)			8.1	Desirable: 12′ Minimum: 11′			
Shoulder/Curb Offset	Type		8.1	Paved		Des.: 2′ Paved/Remainder Sod Min.: Sod	
	* Width		8.1	2′ Min. (3)		Desirable: 6′ Minimum: 4′	
Cross Slope	* Travel Lane		8.1	2% Typical			
	Shoulder/Curb Offset		8.1	Same as adjacent T.L.		Paved: 2%-4% Sod: 6%-8%	
Auxiliary Lanes	Lane Width (4)		8.1	Desirable: Same as Travel Lane; Minimum: 11′			
	Shoulder/Curb Offset		8.1	2′ Min. (5)		Desirable: 4′ Minimum: 2′	
Parking Lane Width (6)			17.1	Desirable: 10′ Minimum: 7′			
Right-of-Way Width			8.6	(7)			
Clear Zone			11.2	1.5′ (8)		See Section 11.2	
Side Slopes	Cut (9)	Fore Slope	8.3	NA		3:1	
		Ditch Width	8.3	NA		4′	
		Back Slope	8.3	2:1		3:1	
	Fill (10)	0′-4′ Height	8.3	3:1		3:1	
		4′-10′ Height	8.3	3:1		3:1	
		> 10′ Height	8.3	3:1		3:1	
				30 mph	40 mph	45 mph	50 mph
* Minimum Stopping Sight Distance (11)			5.7	200′	275′	325′	400′
Intersection Sight Distance			9.2	See Section 9.2			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		6.1	24°45′	11°30′	8°30′	NA
	emax=0.06		6.1	26°45′	12°45′	9°15′	6°45′
* Superelevation Rate			6.2	See Section 6.2			
* Vertical Curvature for Minimum SSD	Crest		7.2	K=30	K=60	K=80	K=110
	Sag		7.2	K=40	K=60	K=70	K=90
* Maximum Grade (12)	Level		7.1	9%	9%	8%	7%
	Rolling		7.1	11%	10%	9%	8%
	Mountainous		7.1	12%	12%	11%	10%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%			
				AADT < 400	DHV 100 - 200	DHV 200 - 400	DHV > 400
* New/Reconstructed Bridges	Structural Capacity (13)		–	HS-20/Oklahoma Overload Truck			
	Width		8.4	With Curbs: App. Roadway Width; Without Curbs: Travelway + 8′			App. Roadway Width
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-15 (Inventory Rating)			
	Width (State Highways) (14)		8.4	24′		26′	28′
	Width (Non-State Hwys) (14)		8.4	22′		24′	28′
* Vertical Clearance (15)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

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

Footnotes to Table 12-10

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. Where truck volume ≥10%, lane widths should be 12′. In industrial areas, lane widths should be 12′. In residential areas with right-of-way restrictions, 10′ lanes are acceptable.

3. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

4. Auxiliary Lane Width. In restricted locations, 10′ widths may be used.

5. Auxiliary Lane Shoulder/Curb Offset. Widths adjacent to auxiliary lanes will typically be 4′ or equal to the width adjacent to the travel lane, whichever is less.

6. Parking Lanes. In industrial or commercial areas, the minimum parking lane width is 9′. The cross slope of the parking lane will generally be the same as that of the adjacent travel lane.

7. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the necessary width for fill and cut slopes and clear zones.

8. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

9. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

10. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

11. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

12. Maximum Grades. For grades less than 500′ in length (PVT to PVC), the maximum grade may be 2% steeper than table value.

13. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

14. Width (Existing Bridges to Remain in Place). Widths in table are for uncurbed sections; curbed section bridges will be evaluated on a site-specific basis. For bridges of more than 100′ in length and not on the State highway system, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

15. Minimum Vertical Clearance. The vertical clearances apply to the collector passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a collector. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-11
GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS
(New Construction/Reconstruction)

Design Element			Manual Section	With Curb		Without Curb	
Standard Designation			–	UC-1		UC-2	
Design Year (Geometrics)			5.3	Desirable: 20 years Minimum: 10 years			
* Design Speed (mph) (1)			5.2	30-50			
Access Control			5.4	Control by Regulation			
Level of Service			5.3	D			
* Lane Width (2)			8.1	Desirable: 12′ Minimum: 11′			
Shoulder/Curb Offset	Type		8.1	Paved		Des.: 2′ Paved/Remainder Sod Min.: Sod	
	* Width		8.1	2′ Min. (3)		Desirable: 6′ Minimum: 4′	
Cross Slope	* Travel Lane		8.1	2% Typical			
	Shoulder/Curb Offset		8.1	Same as adjacent T.L.		Paved: 2%-4% Sod: 6%-8%	
Auxiliary Lanes	Lane Width		8.1	Desirable: Same as Travel Lane; Minimum: 10′			
	Shoulder/Curb Offset		8.1	2′ Min. (4)		Desirable: 4′ Minimum: 2′	
Parking Lane Width (5)			17.1	Desirable: 10′ Minimum: 7′			
Right-of-Way Width			8.6	(6)			
Clear Zone			11.2	1.5′ (7)		See Section 11.2	
Side Slopes	Cut (8)	Fore Slope	8.3	NA		3:1	
		Ditch Width	8.3	NA		4′	
		Back Slope	8.3	2:1		3:1	
	Fill (9)	0′-4′ Height	8.3	3:1		3:1	
		4′-10′ Height	8.3	3:1		3:1	
		> 10′ Height	8.3	3:1		3:1	
				30 mph	40 mph	45 mph	50 mph
* Minimum Stopping Sight Distance (10)			5.7	200′	275′	325′	400′
Intersection Sight Distance			9.2	See Section 9.2			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		6.1	24°45′	11°30′	8°30′	NA
	emax=0.06		6.1	26°45′	12°45′	9°15′	6°45′
* Superelevation Rate			6.2	See Section 6.2			
* Vertical Curvature for Minimum SSD	Crest		7.2	K=30	K=60	K=80	K=110
	Sag		7.2	K=40	K=60	K=70	K=90
* Maximum Grade (11)	Level		7.1	9%	9%	8%	7%
	Rolling		7.1	11%	10%	9%	8%
	Mountainous		7.1	12%	12%	11%	10%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%			
				AADT < 400	DHV 100 - 200	DHV 200 - 400	DHV > 400
* New/Reconstructed Bridges	Structural Capacity (12)		–	HS-20/Oklahoma Overload Truck			
	Width		8.4	With Curbs: App. Roadway Width; Without Curbs: Travelway + 8′			App. Roadway Width
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-15 (Inventory Rating)			
	Width (State Highways) (13)		8.4	24′		26′	28′
	Width (Non-State Hwys) (13)		8.4	22′		24′	28′
* Vertical Clearance (14)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

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

Footnotes to Table 12-11

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. Where truck volume ≥10%, lane widths should be 12′. In industrial areas, lane widths should be 12′. In residential areas with right-of-way restrictions, 10′ lanes are acceptable.

3. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

4. Auxiliary Lane Shoulder/Curb Offset. Widths adjacent to auxiliary lanes will typically be 4′ or equal to the width adjacent to the travel lane, whichever is less.

5. Parking Lanes. In industrial or commercial areas, the minimum parking lane width is 9′. The cross slope of the parking lane will generally be the same as that of the adjacent travel lane.

6. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the necessary width for fill and cut slopes and clear zones.

7. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

8. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

9. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

10. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

11. Maximum Grades. For grades less than 500′ in length (PVT to PVC), the maximum grade may be 2% steeper than table value.

12. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

13. Width (Existing Bridges to Remain in Place). Widths in table are for uncurbed sections; curbed section bridges will be evaluated on a site-specific basis. For bridges of more than 100′ in length and not on the State highway system, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

14. Minimum Vertical Clearance. The vertical clearances apply to the collector passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a collector. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-12
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN LOCAL STREETS**
(New Construction/Reconstruction)

Design Element			Manual Section	With Curb	Without Curb
Standard Designation			–	SL-1	SL-2
Design Year (Geometrics)			5.3	Desirable: 20 years Minimum: 10 years	
* Design Speed (mph) (1)			5.2	20-30	
Access Control			5.4	Control by Regulation	
Level of Service			5.3	D	
* Lane Width (2)			8.1	Desirable: 12′ Minimum: 11′	
Shoulder/Curb Offset	Type		8.1	Paved	Chip Seal, Gravel or Dirt
	* Width		8.1	2′ Min. (3)	Desirable: 6′ Minimum: 4′
Cross Slope	* Travel Lane		8.1	Paved/Chip Seal: 2% Gravel: 2%-4% Dirt: 4%-6%	
	Shoulder/Curb Offset		8.1	Same as adjacent T.L.	Gravel: 4%-6% Sod: 6%-8%
Parking Lane Width (4)			17.1	7′ Typical	
Right-of-Way Width			8.6	(5)	
Clear Zone			11.2	1.5′ (6)	See Section 11.2
Side Slopes	Cut (7)	Fore Slope	8.3	NA	3:1
		Ditch Width	8.3	NA	4′
		Back Slope	8.3	2:1	3:1
	Fill (8)	0′-4′ Height	8.3	3:1	3:1
		4′-10′ Height	8.3	3:1	3:1
		> 10′ Height	8.3	3:1	3:1
				20 mph	30 mph
* Minimum Stopping Sight Distance (9)			5.7	125′	200′
Intersection Sight Distance			9.2	See Section 9.2	
Decision Sight Distance			5.7	See Section 5.7	
* Maximum Degree of Curvature	emax=0.04		6.1	72°45′	24°45′
	emax=0.06		6.1	77°00′	26°45′
* Superelevation Rate			6.2	See Section 6.2	
* Vertical Curvature for Minimum SSD	Crest		7.2	K=10	K=30
	Sag		7.2	K=20	K=40
* Maximum Grade	Residential		7.1	15%	
	Commercial/Industrial		7.1	Desirable: 5% Maximum: 8%	
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%	

Design Element		Manual Section	AADT < 50	AADT 50-250	AADT 250-400	DHV 100-200	DHV 200-400	DHV > 400
* New/Reconstructed Bridges	Structural Capacity	–	HS-20					
	Width (10)	8.4	Travelway + 4′			Travelway + 6′		App Rd Wid
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-10	HS-15				
	Width (11)	8.4	20′		22′		24′	28′
* Vertical Clearance (12)	New/Replaced Bridges	7.4	16′-9″					
	Existing Bridges	7.4	14′-6″					

* Controlling design criteria (See Section 5.8)

** Table will only apply where State and/or Federal funds are used.

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN LOCAL STREETS
(New Construction/Reconstruction)

Footnotes to Table 12-12

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. In industrial areas, lane widths should be 12′. In residential areas with right-of-way restrictions, 10′ lanes are acceptable.

3. Shoulder/Curb Offset. The curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

4. Parking Lanes. In industrial or commercial areas, the minimum parking lane width is 9′. The cross slope of the parking lane will generally be the same as that of the adjacent travel lane.

5. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the necessary width for fill and cut slopes and clear zones.

6. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

7. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

8. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

9. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

10. Width (New and Reconstructed Bridges). Widths of bridges more than 100′ will be analyzed individually.

11. Width (Existing Bridges to Remain in Place). For bridges of more than 100′ in length, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

12. Minimum Vertical Clearance. The vertical clearances apply to the local street passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a local street. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 12-13
GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREETS**
(New Construction/Reconstruction)

Design Element			Manual Section	With Curb			Without Curb		
Standard Designation			–	UL-1			UL-2		
Design Year (Geometrics)			5.3	Desirable: 20 years Minimum: 10 years					
* Design Speed (mph) (1)			5.2	20-30					
Access Control			5.4	Control by Regulation					
Level of Service			5.3	D					
* Lane Width (2)			8.1	Desirable: 12' Minimum: 10'					
Shoulder/Curb Offset	Type		8.1	Paved			Chip Seal, Gravel or Dirt		
	* Width		8.1	2' Min. (3)			Desirable: 4' Minimum: 2'		
Cross Slope	* Travel Lane		8.1	Paved/Chip Seal: 2% Gravel: 2%-4% Dirt: 4%-6%					
	Shoulder/Curb Offset		8.1	Same as adjacent T.L.			Gravel: 4%-6% Sod: 6%-8%		
Parking Lane Width (4)			17.1	7' Typical					
Right-of-Way Width			8.6	(5)					
Clear Zone			11.2	1.5' (6)			See Section 11.2		
Side Slopes	Cut (7)	Fore Slope	8.3	NA			3:1		
		Ditch Width	8.3	NA			4'		
		Back Slope	8.3	2:1			3:1		
	Fill (8)	0'-4' Height	8.3	3:1			3:1		
		4'-10' Height	8.3	3:1			3:1		
		> 10' Height	8.3	3:1			3:1		
				20 mph			30 mph		
* Minimum Stopping Sight Distance (9)			5.7	125'			200'		
Intersection Sight Distance			9.2	See Section 9.2					
Decision Sight Distance			5.7	See Section 5.7					
* Maximum Degree of Curvature	emax=0.04		6.1	72°45'			24°45'		
	emax=0.06		6.1	77°00'			26°45'		
* Superelevation Rate			6.2	See Section 6.2					
* Vertical Curvature for Minimum SSD	Crest		7.2	K=10			K=30		
	Sag		7.2	K=20			K=40		
* Maximum Grade	Residential		7.1	15%					
	Commercial/Industrial		7.1	Desirable: 5% Maximum: 8%					
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.4%; Uncurbed: 0%					
				AADT < 50	AADT 50-250	AADT 250-400	DHV 100-200	DHV 200-400	DHV > 400
* New/Reconstructed Bridges	Structural Capacity		–	HS-20					
	Width (10)		8.4	Travelway + 4'			Travelway + 6'		App Rd Wid
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-10	HS-15				
	Width (11)		8.4	20'		22'		24'	28'
* Vertical Clearance (12)	New/Replaced Bridges		7.4	16'-9"					
	Existing Bridges		7.4	14'-6"					

* Controlling design criteria (See Section 5.8)

** Table will only apply where State and/or Federal funds are used.

GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREETS
(New Construction/Reconstruction)

Footnotes to Table 12-13

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Lane Widths. In industrial areas, lane widths should desirably be 12′. Where right-of-way is restricted, lane widths may be 11′ in industrial areas and commercial areas and may be 9′ in residential areas.

3. Shoulder/Curb Offset. The curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

4. Parking Lanes. In industrial or commercial areas, the minimum parking lane width is 9′. The cross slope of the parking lane will generally be the same as that of the adjacent travel lane.

5. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane width plus the outside shoulder widths plus the necessary width for fill and cut slopes and clear zones.

6. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

7. Cut Slopes. Typical values in table apply to earth cuts. See Section 8.3 for rock cuts. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the toe of the back slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

8. Fill Slopes. On facilities with curbs, it is desirable to provide a 6′ sodded "shelf" between the curb and the break in the fill slope or an 8′ shelf where a sidewalk is present. See Section 8.3.

9. Minimum Stopping Sight Distance. Values in table are minimum SSD criteria for passenger cars on level grades. See Section 6.5 for the application of the SSD to horizontal curves.

10. Width (New and Reconstructed Bridges). Widths of bridges more than 100′ will be analyzed individually.

11. Width (Existing Bridges to Remain in Place). For bridges of more than 100′ in length, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

12. Minimum Vertical Clearance. The vertical clearances apply to the local street passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a local street. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Chapter Thirteen

Geometric Design of Existing Highways

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Chapter Thirteen

GEOMETRIC DESIGN OF EXISTING HIGHWAYS

13.1 INTRODUCTION

Section 5.6 of the *ODOT Roadway Design Manual* identifies five project scopes of work:

1. new construction,
2. reconstruction (non-freeways),
3. 3R projects (non-freeways),
4. 3R projects (freeways), and
5. spot improvements.

Chapter Twelve presents tables of ODOT's geometric design criteria which apply to new construction/reconstruction projects. In addition, Chapters Five through Eleven present many design concepts and criteria which are directly applicable to new construction/ reconstruction. For these projects, the designer typically has the liberty of designing the highway to meet the most desirable and stringent criteria practical.

The geometric design of projects on existing highways must be viewed from a different perspective. These projects are often initiated for reasons other than geometric design deficiencies (e.g., pavement deterioration), and they often must 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, ODOT must use the opportunity to make cost-effective, practical improvements to the geometric design of existing highways and streets.

For these reasons, ODOT 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. The revised criteria for existing highways are based on a sound, engineering assessment of the underlying principles behind geometric design and on how the criteria for new construction can be legitimately modified to apply to existing highways.

This chapter presents ODOT's criteria for 3R non-freeway projects, 3R freeway projects and spot improvements. These criteria are intended to find the balance among many competing and often conflicting objectives. The objectives include improving Oklahoma's existing highways; minimizing the adverse impacts of highway construction on existing highways; and improving the greatest number of miles within the available funds for capital improvements.

3R = RESURFACING, RESTORATION, REHABILITATION

13.2 3R NON-FREEWAY PROJECTS

13.2.1 Background

13.2.1.1 Federal 3R Regulations

On June 10, 1982, the 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 non-freeway 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 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 its 3R program consistent with the conditions which prevail 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.

13.2.1.2 Special Report 214

In 1987, the Transportation Research Board published Special Report 214 *Designing Safer Roads; Practices for Resurfacing, Restoration and Rehabilitation*. 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. 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 Oklahoma, into ODOT's criteria and procedures for 3R projects. The designer should reference SR214 for more discussion on 3R projects.

13.2.1.3 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 12 controlling design criteria on 3R projects; and
4. presents specific criteria for the design of 3R projects.

The information from the Technical Advisory has been incorporated, where considered appropriate for Oklahoma, into ODOT's criteria and procedures for 3R projects.

13.2.2 Objectives

From an overall perspective, the 3R program is intended to improve the greatest number of highway miles within the available funds for highway construction 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. ODOT's responsibility is to realize the greatest overall benefit from the available funds for construction. 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 criteria) and what is practical for the specific conditions of each highway project.

Therefore, considering the above discussion, ODOT has developed its own criteria for the geometric design of 3R non-freeway projects. The overall objective of ODOT'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 incorporate highway safety enhancements, where judged to be cost effective.

3. 3R projects are intended to incorporate cost-effective, practical improvements to the geometric design of the existing facility.

13.2.3 Approach

ODOT's approach to the geometric design of 3R non-freeway projects is to adopt, where justifiable, a revised set of numerical criteria. The design criteria throughout the other chapters in the *Roadway Design Manual* provide the frame of reference for the 3R criteria. The following summarizes the approach which has been used:

1. Design Speed. As discussed in Section 13.2.6, the design speed will typically be based on the existing posted or regulatory speed limit. The selected 3R design speed will then be used to evaluate all geometric design features of the existing highway which are based on speed (e.g., horizontal and vertical curvature).

2. Cross Section Widths. The criteria in Chapter Twelve for new construction/ reconstruction have been evaluated relative to the typical constraints of 3R projects. Where justifiable, the cross section width criteria have been reduced. Where a range of values is provided in the Chapter Twelve tables, the upper values have been incorporated into the 3R criteria to provide a desirable objective. This provides an expanded range of acceptable values for application on 3R projects. See Section 13.2.9 for additional discussion on cross section widths.

3. Other Design Criteria. The *ODOT Roadway Design Manual* contains many other details on proper geometric design techniques. These criteria are obviously applicable to new construction/ 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.*

4. Evaluation. The designer should evaluate available data (e.g., accident experience) when determining the geometric design of 3R projects. The necessary evaluations presented for 3R projects are based on the FHWA Technical Advisory T5040.28 "Geometric Design Criteria for Non-Freeway RRR Projects." Section 13.2.4 discusses 3R project evaluation in more detail.

13.2.4 3R Project Evaluation

Sections 13.2.5 to 13.2.12 present the specific geometric design and roadside safety criteria which will be used to determine the design of 3R projects. In addition, several other factors must be considered in a 3R project, and the designer should conduct applicable evaluations as may be deemed necessary. These evaluations are discussed below:

1. Accident Experience. The historical accident data within the project limits of the 3R project will be evaluated. This is typically the most critical element of 3R project evaluation to determine the appropriate level of geometric and safety improvement. Accident data is available from the ODOT Traffic Engineering 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 large number of head-on and/or sideswipe accidents may indicate inadequate roadway width. A large 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 designer should check to see if any locations on ODOT's Suggested Surveillance Study Sites, as identified by ODOT's accident data system, fall within the proposed project limits.

d. Collision Diagrams. The Traffic Engineering Division provides, upon request by the designer, collision diagrams at intersections. To receive these, the designer must check the applicable box on the ODOT Accident Report Form. The designer's evaluation of collision diagrams may indicate the need for intersection improvements within the limits of the 3R project. See Chapter Fourteen for more information.

2. Existing Geometrics. The designer will normally review the as-built plans and combine this with the field review and field survey to determine the existing geometrics within the project limits. This includes lane and shoulder widths, horizontal and vertical alignment, intersection geometrics and the roadside safety design.

3. Speed Studies. The designer will make an initial determination on a case-by-case basis that a speed study may be needed for project design. Upon request, the Traffic Engineering Division will review any recent speed studies in the vicinity of the project and, if necessary, conduct a field study to determine the design speed of the 3R project. In addition, it may be desirable to conduct spot speed studies in specific locations (e.g., in advance of a specific horizontal or vertical curve) to assist in the determination of geometric design improvements. The speed study should be conducted before the field review.

4. 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).

5. Field Review. The designer will normally conduct a thorough field review of the proposed 3R project. Other personnel should attend the field review as appropriate, including personnel from traffic, maintenance, construction, FHWA, etc. The objective of the field review should be to identify potential safety hazards and potential safety improvements to the facility.

6. Pavement Condition. 3R projects are usually programmed because of a significant deterioration of the existing pavement structure (including subbase, base and surface course). The extent of deterioration will determine the necessary level of pavement improvements. This decision will also influence the extent of practical geometric improvements.

7. Geometric Design of Adjacent Highway Sections. The designer should examine the geometric features and operating speeds of highway sections adjacent to the 3R project. This will include 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.

8. Early Coordination for Right-of-Way Acquisition/Utilities. Significant R/W acquisitions are typically outside the scope of 3R projects. However, the field review and accident or speed studies may indicate the need for selective safety improvements which will require R/W purchases. Therefore, the designer should, as early as feasible, determine improvements which will be incorporated into the project design and initiate the R/W acquisition process.

   Utility relocation and accommodation is often a problem on 3R projects because of the typically restrictive project constraints. The need for utility accommodation may lead to the need for R/W acquisition and/or easements. If this assessment is not made early in project development, utility accommodation may result in project delays.

9. Maintenance and Protection of Traffic. A significant portion of 3R work will occur on existing highways. Therefore, maintenance and protection of traffic during construction will be an important consideration in 3R project development. The protection of the highway construction workers is also an important factor. The designer should reference Chapter Fourteen for ODOT criteria on the design of work zones for traffic accommodation. This will require significant coordination with the Traffic Engineering Division.

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. However, the designer should work with Traffic to identify possible geometric and safety deficiencies which will remain in place (i.e., no improvement will be made). These may include:

    a. narrow bridges,

    b. horizontal and vertical curves which do not meet the 3R criteria, and

    c. roadside hazards within the clear zone.

    The Traffic Engineering Division will then determine if additional signing, traffic control devices or delineation treatments are warranted.

11. Document the Design Process. The designer should prepare a safety and design report. This report should describe the following:

    a. existing geometric and roadside features, traffic volumes and speeds, and accident history;

    b. applicable minimum design criteria;

    c. specific safety problems or concerns raised by a review of accident data, by a field inspection, or by the public;

    d. design options for correcting safety problems and the cost, safety and other relevant impacts of these options;

e. proposed exceptions to applicable design criteria and the rationale to support the exceptions; and

f. the recommended design proposal.

The designer must also prepare the Design Exception Checklist for the 3R project (see Section 5.8).

13.2.5 Tables of 3R Geometric Design Values

Tables 13.2A through 13.2L present ODOT's criteria for the design of 3R projects for both rural and urban areas. The designer should consider the following in the use of the 3R design tables:

1. Functional/Design Classification. The selection of design values for 3R projects depends on the functional and design classification of the highway facility. This is discussed in Section 5.1. The designer should use the highway classification for the selected design year (e.g., desirably 10 years beyond the construction completion date).

2. Cross Section Elements. The designer should realize that some of the cross section elements included in the table (e.g., median width) are not automatically warranted in the project design. The values in the tables will only apply after the decision has been made to include the element in the highway cross section.

   General ODOT policy is that a 3R project will not be designed with a narrower roadway width than the existing facility. See Section 13.2.9.

3. Manual Section References. These tables are intended to provide a concise listing of design values for easy use. However, the designer should review the appropriate section references for greater insight into the design elements.

4. Footnotes. The tables include many footnotes, which are identified by a number in parenthesis (e.g., (6)). The information in the footnotes is critical to the proper use of the design tables.

5. Controlling Design Criteria. The 3R tables of geometric design criteria provide an asterisk to indicate controlling design criteria. The discussion in Section 5.8 on design exceptions applies equally to the geometric design of 3R projects. However, the designer will evaluate the proposed design against the criteria presented in Chapter Thirteen.

Table 13.2A
GEOMETRIC DESIGN CRITERIA FOR RURAL PRINCIPAL ARTERIALS
(3R Projects)

Design Element			Manual Section	2-Lane: AADT 751-2000	2-Lane: AADT > 2000	Divided Multilane**
Standard Designation			--	RPA-1 (3R)		RPA-2 (3R)
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years		Desirable: 10 years
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit		Posted Speed Limit
Access Control			5.4	Control by Regulation		Control by Regulation/Partial Control
Level of Service			5.3	Desirable: B Minimum: D		Desirable: B Minimum: D
* Lane Width (3)	Design Speed < 50 mph		13.2.9	11′	12′	12′
	Design Speed ≥ 50 mph		13.2.9	12′	12′	
Shoulder	Type		13.2.9	Des.: Full Paved; Min.: Sod		Paved
	* Width (4)		13.2.9	3′ (4a)	6′	RT: Des.: 10′, Min: 6′; LT: Des.: 4′, Min.: 2′ (4b)
Cross Slope	* Travel Lane		13.2.9	1.5% - 2%		1.5% - 2%
	Shoulder (5)		13.2.9	2% - 6%		2% - 6%
Auxiliary Lanes (6)	Lane Width		13.2.9	Desirable: 12′ Minimum: 11′		Desirable: 12′ Minimum: 11′
	Shoulder Width		13.2.9	Desirable: 4′ (paved); Minimum: 4′ (sod)		Desirable: 4′ (paved); Minimum: 4′ (sod)
Median Width			13.2.9	NA		Existing (7)
Right-of-Way Width			13.2.9	(8)		(8)
Clear Zone			13.2.11	See Section 13.2.11		See Section 13.2.11
Side Slopes (9)	Cut	Fore Slope	13.2.9	Existing		Existing
		Ditch Width	13.2.9	Existing		Existing
		Back Slope	13.2.9	Existing		Existing
	Fill	0′-4′ Height	13.2.9	Existing		Existing
		4′-10′ Height	13.2.9	Existing		Existing
		> 10′ Height	13.2.9	Existing		Existing

Design Element		Manual Section	50 mph	60 mph	70 mph
* Minimum Stopping Sight Distance (10)		5.7	400′	525′	625′
Intersection Sight Distance		13.2.10	See Section 13.2.10		
Decision Sight Distance		5.7	See Section 5.7		
Passing Sight Distance		5.7	Existing	Existing	Existing
* Maximum Degree of Curvature	$e_{max}=0.06$	13.2.7	See Section 13.2.7		
	$e_{max}=0.08$	13.2.7	See Section 13.2.7		
* Superelevation Rate		13.2.7	See Section 13.2.7		
* Vertical Curvature for Desirable SSD	Crest	13.2.8	See Section 13.2.8		
	Sag	13.2.8	See Section 13.2.8		
Maximum Grade	Level	13.2.8	6%	5%	5%
	Rolling	13.2.8	7%	6%	6%
	Mountainous	13.2.8	9%	8%	7%
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%		
* New/Reconstructed Bridges	Structural Capacity (11)	--	HS-20/OK Overload Truck		
	Width	13.2.9	Full Approach Roadway Width		
* Existing Bridges to Remain in Place	Structural Capacity	--	HS-20 (Inventory Rating)		
	Width (12)	13.2.9	Approach Travelway +4′ (28′ min)		
* Vertical Clearance (13)	New/Replaced Bridges	7.4	16′-9″		
	Existing Bridges	7.4	14′-6″		

* Controlling design criteria (See Section 5.8)

** See Table 13.2E "Suburban Principal Arterials" for multilane undivided design criteria for Rural Principal Arterials.

GEOMETRIC DESIGN CRITERIA FOR RURAL PRINCIPAL ARTERIALS (3R Projects)

Footnotes to Table 13.2A

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. See the Planning Division for National Network system.
   b. If there are less than 10% trucks, lane widths may be 1′ less from table values.
   c. If there are 10% or more trucks, lane widths should desirably be 12′.
   d. Existing 11′ lanes may be retained if alignment and safety records are satisfactory, except for routes on the National Network or on reasonable access routes.

4. Shoulder Width. The following will apply:

   a. If the design speed is less than 50 mph, the shoulder width may be 2′.
   b. Where there are three or more lanes in one direction, the left shoulder should desirably be 8′.
   c. Where a barrier is present, all shoulders should be paved and their widths should desirably be increased by 2′.
   d. If guardrail is present, the minimum shoulder width is 4′ and should be paved.
   e. Shoulder widths may be 1′ less than table values in mountainous terrain.

5. Shoulder Cross Slope. For paved shoulders less than 4′ wide, the shoulder cross slope will typically be the same as the travel lane.

6. Auxiliary Lanes. The following will apply:

   a. The minimum climbing lane width will be 11′.
   b. The minimum shoulder width adjacent to the climbing lane will be 4′ and paved.

7. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way and field conditions. See Section 13.2.9.

8. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often 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.

9. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

10. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which will remain.

11. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

12. Existing Bridges to Remain in Place. For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

13. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

   a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.
   b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2B
GEOMETRIC DESIGN CRITERIA FOR RURAL OTHER ARTERIALS
(3R Projects)

Design Element			Manual Section	2-Lane AADT≤750	2-Lane AADT 751-2000	2-Lane AADT >2000	Divided Multilane**	
Standard Designation			–	ROA-1 (3R)			ROA-2 (3R)	
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years			Desirable: 10 years	
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit			Posted Speed Limit	
Access Control			5.4	Control by Regulation			Control by Regulation	
Level of Service			5.3	Desirable: B Minimum: D			Desirable: B Minimum: D	
* Lane Width (3)	Design Speed < 50 mph		13.2.9	10′	11′	12′	Desirable: 12′ Minimum: 11′	
	Design Speed ≥ 50 mph			10′	12′	12′		
Shoulder	Type		13.2.9	Des.: Full Paved; Min.: Sod			Paved	
	* Width (4)		13.2.9	2′	3′ (4a)	6′	RT: Des.: 8′ Min.: 6′; LT: Des. 4′, Min.: 2 (4b)	
Cross Slope	* Travel Lane		13.2.9	1.5% - 2%			1.5% - 2%	
	Shoulder (5)		13.2.9	2% - 6%			2% - 6%	
Auxiliary Lanes (6)	Lane Width		13.2.9	Desirable: Same as Travel Lane; Minimum: 10′			Desirable: 12′ Minimum: 11′	
	Shoulder Width		13.2.9	Desirable: 4′ Paved; Minimum: 4′ Sod			Desirable: 4′ Paved; Minimum: 4′ Sod	
Median Width			13.2.9	N/A			Existing (7)	
Right-of-Way Width			13.2.9	(8)			(8)	
Clear Zone			13.2.11	See Section 13.2.11			See Section 13.2.11	
Side Slopes (9)	Cut	Fore Slope	13.2.9	Existing			Existing	
		Ditch Width	13.2.9	Existing			Existing	
		Back Slope	13.2.9	Existing			Existing	
	Fill	0′-4′ Height	13.2.9	Existing			Existing	
		4′-10′ Height	13.2.9	Existing			Existing	
		>10′ Height	13.2.9	Existing			Existing	
				40 mph	50 mph		60 mph	70 mph
* Minimum Stopping Sight Distance (10)			5.7	275′	400′		525′	625′
Intersection Sight Distance			13.2.10	See Section 13.2.10				
Decision Sight Distance			5.7	See Section 5.7				
Passing Sight Distance			5.7	Existing	Existing		Existing	Existing
* Maximum Degree of Curvature	e_{max} = 0.06		13.2.7	See Section 13.2.7				
	e_{max} = 0.08		13.2.7	See Section 13.2.7				
* Superelevation Rate			13.2.7	See Section 13.2.7				
* Vertical Curvature for Minimum SSD	Crest		13.2.8	See Section 13.2.8				
	Sag		13.2.8	See Section 13.2.8				
Maximum Grade	Level		13.2.8	7%	6%		5%	5%
	Rolling		13.2.8	8%	7%		6%	6%
	Mountainous		13.2.8	10%	9%		8%	7%
Minimum Grade			7.1	Desirable: 0.5% Minimum: 0%				
* New/Reconstructed Bridges	Structural Capacity (11)		–	HS-20/OK Overload Truck				
	Width		13.2.9	Full Approach Roadway Width				
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-20 (Inventory Rating)				
	Width (12)		13.2.9	Approach Travelway +4′ (28′ minimum)				
* Vertical Clearance (13)	New/Replaced Bridges		7.4	16′-9″				
	Existing Bridges		7.4	14′-6″				

* Controlling design criteria (See Section 5.8)

** See Table 13.2.G "Suburban Other Arterials" for multilane undivided design criteria for Rural Other Arterials.

GEOMETRIC DESIGN CRITERIA FOR RURAL OTHER ARTERIALS (3R Projects)

Footnotes to Table 13.2B

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. See the Planning Division for National Network system.
   b. If there are less than 10% trucks, lane widths may be 1′ less from table values.
   c. If there are 10% or more trucks, lane widths should desirably be 12′.
   d. Existing 11′ lanes may be retained if alignment and safety records are satisfactory, except for routes on the National Network or on reasonable access routes.

4. Shoulder Width. The following will apply:

   a. If the design speed is less than 50 mph, the shoulder width may be 2′.
   b. Where there are three or more lanes in one direction, the left shoulder should desirably be 8′.
   c. Where a barrier is present, all shoulders should be paved and their widths should desirably be increased by 2′.
   d. If guardrail is present, the minimum shoulder width is 4′ paved.
   e. Shoulder widths may be 1′ less than table values in mountainous terrain.

5. Shoulder Cross Slope. For paved shoulders less than 4′ wide, the shoulder cross slope will typically be the same as the travel lane.

6. Auxiliary Lanes. The following will apply:

   a. The minimum climbing lane width will be 11′.
   b. The minimum shoulder width adjacent to the climbing lane will be 4′ and should be paved.

7. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way and field conditions. See Section 13.2.9.

8. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often 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.

9. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

10. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which will remain.

11. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

12. Existing Bridges to Remain in Place. For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

13. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

   a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.
   b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2C
GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (3R Projects)

Design Element			Manual Section	State Highways AADT ≤750	State Highways AADT 751-2000	State Highways AADT >2000	Non-State Highways** AADT ≤750	Non-State Highways** AADT 751-2000	Non-State Highways** AADT >2000
Standard Designation			–	RCS (3R)			RCNS (3R)		
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years			Desirable: 10 years		
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit			Posted Speed Limit		
Access Control			5.4	Control by Regulation			Control by Regulation		
Level of Service			5.3	Level/Rolling: C Mountainous: D			Level/Rolling: C Mountainous: D		
Travel Lane	Type		13.2.9	Paved			Gravel/Paved		
	*Width (3)	Design Speed <50 mph	13.2.9	10′	11′	12′	10′	11′	12′
		Design Speed ≥ 50 mph	13.2.9	10′	12′	12′	10′	12′	12′
Shoulder	Type		13.2.9	Des.: Paved Min.: Sod			Gravel/Sod		
	* Width (4)		13.2.9	2′	3′ (4a)	6′	2′	3′ (4a)	6′
Cross Slope	* Travel Lane		13.2.9	1.5%-2%			Paved: 1.5%-2% Gravel: 2%-4%		
	Shoulder (5)		13.2.9	2%-6%			Gravel: 4%-6% Sod: 6%-8%		
Auxiliary Lanes (6)	Lane Width		13.2.9	Des: 11′ Min: 10′	Des: 12′ Min: 10′		NA		
	Shoulder Width		13.2.9	Des: 4′ Min.: 2′			NA		
Right-of-Way Width			13.2.9	(7)			(7)		
Clear Zone			13.2.11	See Section 13.2.11			See Section 13.2.11		
Side Slopes (8)	Cut	Fore Slope	13.2.9	Existing			Existing		
		Ditch Width	13.2.9	Existing			Existing		
		Back Slope	13.2.9	Existing			Existing		
	Fill	0′-4′ Height	13.2.9	Existing			Existing		
		4′-10′ Height	13.2.9	Existing			Existing		
		> 10′ Height	13.2.9	Existing			Existing		

Design Element		Manual Section	20 mph	30 mph	40 mph	50 mph	60 mph
* Minimum Stopping Sight Distance (9)		5.7	125′	200′	275′	400′	525′
Intersection Sight Distance		13.2.10	See Section 13.2.10				
Decision Sight Distance		5.7	See Section 5.7				
Passing Sight Distance			NA	Existing	Existing	Existing	Existing
* Maximum Degree of Curvature	emax = 0.06	13.2.7	See Section 13.2.7				
	emax = 0.08	13.2.7	See Section 13.2.7				
* Superelevation Rate		13.2.7	See Section 13.2.7				
* Vertical Curvature for Minimum SSD	Crest	13.2.8	See Section 13.2.8				
	Sag	13.2.8	See Section 13.2.8				
Maximum Grade	Level	13.2.8	9%	9%	9%	8%	7%
	Rolling	13.2.8	11%	11%	10%	9%	8%
	Mountainous	13.2.8	14%	12%	12%	11%	10%
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%				

Design Element		Manual Section	<200 DHV	200-400 DHV	>400 DHV	≤750	751-2000	>2000
* New/Reconstructed Bridges	Structural Capacity	–	HS-20/OK Overload Truck			HS-20		
	Width	13.2.9	TW + 8′	App. Roadway Width		TW + 4′	TW + 6′	APR
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-15 (Inventory Rating)			HS-15		
	Width	13.2.9	24′	26′	28′	(10)		
* Vertical Clearance (11)	New/Replaced Bridges	7.4	16′-9″			16′-9″		
	Existing Bridges	7.4	14′-6″			14′-0″		

* Controlling design criteria (See Section 5.8) ** These criteria will only apply where State and/or Federal funds are used. TW = Travelway; APR = Approach Roadway Width

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS (3R Projects)

Footnotes to Table 13.2C

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. See the Planning Division for National Network system.
   b. If there are less than 10% trucks, lane widths may be 1′ less from table values.
   c. Existing 11′ lanes may be retained if alignment and safety records are satisfactory, except for routes on the National Network or on reasonable access routes.

4. Shoulder Width. The following will apply:

   a. If the design speed is less than 50 mph, the shoulder width may be 2′.
   b. Where a barrier is present, all shoulders should be paved and their widths should desirably be increased by 2′.
   c. Shoulder widths may be 1′ less than table values in mountainous terrain.

5. Shoulder Cross Slope. For paved shoulders less than 4′ wide, the shoulder cross slope will typically be the same as the travel lane.

6. Auxiliary Lanes. The following will apply:

   a. The minimum climbing lane width will be 11′.
   b. The minimum shoulder width adjacent to the climbing lane will be 4′ and should be paved.

7. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often 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. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

9. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which will remain.

10. Existing Bridges to Remain in Place. For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

    On non-State highways, the bridge should be evaluated for widening to full approach roadway width if the bridge is less than 100′ long and the usable width does not meet the following:

Design Year Volume (AADT)	*Usable Bridge Width (ft)*[a]
0-750	Width of approach travelway
751-2000	Width of approach travelway plus 2 ft
Over 2000	Width of approach travelway plus 4 ft

[a] If lane widening is planned as part of the 3R project, the usable bridge width should be compared with the planned width of the approaches after they are widened.

11. Minimum Vertical Clearance. The vertical clearances apply to the collector passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a collector. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2D
GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROADS**
(3R Projects)

<table>
<tr><th colspan="3">Design Element</th><th>Manual Section</th><th colspan="4">AADT ≤ 750</th><th colspan="4">AADT 751-2000</th><th colspan="4">AADT > 2000</th></tr>
<tr><td colspan="3">Standard Designation</td><td>–</td><td colspan="12">RL (3R)</td></tr>
<tr><td colspan="3">Design Year (Geometrics) (1)</td><td>13.2.6</td><td colspan="12">Desirable: 10 years</td></tr>
<tr><td colspan="3">* Design Speed (mph) (2)</td><td>13.2.6</td><td colspan="12">Posted Speed Limit</td></tr>
<tr><td colspan="3">Access Control</td><td>5.4</td><td colspan="12">Control by Regulation</td></tr>
<tr><td colspan="3">Level of Service</td><td>5.3</td><td colspan="12">D</td></tr>
<tr><td rowspan="3">* Travel Lanes</td><td colspan="2">Type</td><td>13.2.9</td><td colspan="12">Chip Seal, Gravel or Dirt</td></tr>
<tr><td rowspan="2">Width (3)</td><td>Design Speed <50 mph</td><td rowspan="2">13.2.9</td><td colspan="4">9′</td><td colspan="4">10′</td><td colspan="4">11′</td></tr>
<tr><td>Design Speed ≥50 mph</td><td colspan="4">10′</td><td colspan="4">11′</td><td colspan="4">11′</td></tr>
<tr><td rowspan="2">Shoulder</td><td colspan="2">Type</td><td>13.2.9</td><td colspan="12">Gravel or Sod</td></tr>
<tr><td colspan="2">* Width (4a)</td><td>13.2.9</td><td colspan="4">2′</td><td colspan="4">3′ (4b)</td><td colspan="4">6′</td></tr>
<tr><td rowspan="2">Cross Slope</td><td colspan="2">* Travel Lane</td><td>13.2.9</td><td colspan="12">Chip Seal: 2%-3% Gravel: 2%-4% Dirt: 4%-6%</td></tr>
<tr><td colspan="2">Shoulder</td><td>13.2.9</td><td colspan="12">Gravel: 4%-6% Sod: 4%-8%</td></tr>
<tr><td colspan="3">Auxiliary Lanes</td><td>13.2.9</td><td colspan="12">NA</td></tr>
<tr><td colspan="3">Right-of-Way Width</td><td>13.2.9</td><td colspan="12">(5)</td></tr>
<tr><td colspan="3">* Clear Zone</td><td>13.2.11</td><td colspan="12">See Section 13.2.11</td></tr>
<tr><td rowspan="6">Side Slopes (6)</td><td rowspan="3">Cut</td><td>Fore Slope</td><td>13.2.9</td><td colspan="12">Existing</td></tr>
<tr><td>Ditch Width</td><td>13.2.9</td><td colspan="12">Existing</td></tr>
<tr><td>Back Slope</td><td>13.2.9</td><td colspan="12">Existing</td></tr>
<tr><td rowspan="3">Fill</td><td>0′-4′ Height</td><td>13.2.9</td><td colspan="12">Existing</td></tr>
<tr><td>4′-10′ Height</td><td>13.2.9</td><td colspan="12">Existing</td></tr>
<tr><td>> 10′ Height</td><td>13.2.9</td><td colspan="12">Existing</td></tr>
<tr><td colspan="3" rowspan="2">* Minimum Stopping Sight Distance (7)</td><td rowspan="2">5.7</td><td colspan="3">20 mph</td><td colspan="3">30 mph</td><td colspan="3">40 mph</td><td colspan="3">50 mph</td></tr>
<tr><td colspan="3">125′</td><td colspan="3">200′</td><td colspan="3">275′</td><td colspan="3">400′</td></tr>
<tr><td colspan="3">Intersection Sight Distance</td><td>13.2.10</td><td colspan="12">See Section 13.2.10</td></tr>
<tr><td colspan="3">Decision Sight Distance</td><td>5.7</td><td colspan="12">See Section 5.7</td></tr>
<tr><td colspan="3">Passing Sight Distance</td><td>5.7</td><td colspan="4">Existing</td><td colspan="4">Existing</td><td colspan="4">Existing</td></tr>
<tr><td rowspan="2">* Maximum Degree of Curvature</td><td colspan="2">e_{max}=0.06</td><td>13.2.7</td><td colspan="12">See Section 13.2.7</td></tr>
<tr><td colspan="2">e_{max}=0.08</td><td>13.2.7</td><td colspan="12">See Section 13.2.7</td></tr>
<tr><td colspan="3">* Superelevation Rate</td><td>13.2.7</td><td colspan="12">See Section 13.2.7</td></tr>
<tr><td rowspan="2">* Vertical Curvature for Minimum SSD</td><td colspan="2">Crest</td><td>13.2.8</td><td colspan="12">See Section 13.2.8</td></tr>
<tr><td colspan="2">Sag</td><td>13.2.8</td><td colspan="12">See Section 13.2.8</td></tr>
<tr><td rowspan="3">Maximum Grade</td><td colspan="2">Level</td><td>13.2.8</td><td colspan="3">9%</td><td colspan="3">9%</td><td colspan="3">9%</td><td colspan="3">8%</td></tr>
<tr><td colspan="2">Rolling</td><td>13.2.8</td><td colspan="3">13%</td><td colspan="3">12%</td><td colspan="3">11%</td><td colspan="3">10%</td></tr>
<tr><td colspan="2">Mountainous</td><td>13.2.8</td><td colspan="3">16%</td><td colspan="3">15%</td><td colspan="3">14%</td><td colspan="3">12%</td></tr>
<tr><td colspan="3">Minimum Grade</td><td>7.1</td><td colspan="12">Desirable: 0.5% Minimum: 0%</td></tr>
<tr><td rowspan="3">* New/Reconstructed Bridges</td><td colspan="2" rowspan="2">Structural Capacity</td><td rowspan="2">–</td><td colspan="4">AADT ≤ 751</td><td colspan="4">AADT 751-2000</td><td colspan="4">AADT > 2000</td></tr>
<tr><td colspan="12">HS-20</td></tr>
<tr><td colspan="2">Width (8)</td><td>13.2.9</td><td colspan="4">Travelway + 4′</td><td colspan="4">Travelway + 6′</td><td colspan="4">App. Rd. Wid.</td></tr>
<tr><td rowspan="2">* Existing Bridges to Remain in Place</td><td colspan="2">Structural Capacity</td><td>–</td><td colspan="4">HS-10</td><td colspan="8">HS-15</td></tr>
<tr><td colspan="2">Width</td><td>13.2.9</td><td colspan="12">(9)</td></tr>
<tr><td rowspan="2">* Vertical Clearance (10)</td><td colspan="2">New/Replaced Bridges</td><td>7.4</td><td colspan="12">16′-9″</td></tr>
<tr><td colspan="2">Existing Bridges</td><td>7.4</td><td colspan="12">14′-0″</td></tr>
</table>

* Controlling design criteria (See Section 5.9)

** Table will only apply where State and/or Federal funds are used.

GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROADS (3R Projects)

Footnotes to Table 13.2D

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. If there are 10% or more trucks, lane widths should be 1′ more than table values.
   b. Existing 10′ lanes may be retained if alignment and safety records are satisfactory.

4. Shoulder Width. The following will apply:

   a. All shoulder widths refer to the graded width, which is the distance between the edge of travel lane and the point of intersection of the shoulder slope and side slope.

   b. If the design speed is less than 50 mph, the shoulder width may be 2′.

5. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often 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.

6. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

7. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

8. New and Reconstructed Bridges (Width). Width of bridges more than 100′ in length will be analyzed individually.

9. Existing Bridges to Remain in Place. The bridge should be evaluated for widening to full approach roadway width if the bridge is less than 100′ long and the usable width does not meet the following:

Design Year Volume (AADT)	*Usable Bridge Width (ft)*[a]
0-750	Width of approach travelway
751-2000	Width of approach travelway plus 2 ft
Over 2000	Width of approach travelway plus 4 ft

[a] If lane widening is planned as part of the 3R project, the usable bridge width should be compared with the planned width of the approaches after they are widened.

10. Minimum Vertical Clearance. The vertical clearances apply to the local road passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a local road. The typical clearance will be 23′0″. An allowance should also be made for future ballasting.

Table 13.2E
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN PRINCIPAL ARTERIALS
(3R Projects)

Design Element			Manual Section	2-Lane		Multilane	
				With Curb	Without Curb	With Curb	Without Curb
Standard Designation			–	SPA-1 (3R)	SPA-2 (3R)	SPA-3 (3R)	SPA-4 (3R)
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years		Desirable: 10 years	
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit		Posted Speed Limit	
Access Control			5.4	Control by Regulation		Control by Regulation/Partial Control	
Level of Service			5.3	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width			13.2.9	Des.: 12′ Min.: 11′ (3)		Des.: 12′ Min.: 11′ (3)	
Shoulder/Curb Offset	Type		13.2.9	Paved	Des.: Paved; Min.: Sod	Paved	Des.: Paved; Min.: Sod
	* Width		13.2.9	2′ Min. (4a)	Des.: 8′ Min.: 4′	Rt: 2′ Left: 2′ (4a)	Rt: 6′ Left: 2′(4b)
Cross Slope	* Travel Lane		13.2.9	1.5%-2%		1.5%-2% (5)	
	Shoulder/Curb Offset (6)		13.2.9	Same as adjacent T.L.	2%-6%	Same as adjacent T.L.	2%-6% Typical
Auxiliary Lanes	Lane Width		13.2.9	Des.: Same as Travel Lane	Min.: 11′	Des.: Same as Travel Lane	Min.: 11′
	Shoulder/Curb Offset (7)		13.2.9	2′ Min.	3′ Min.	2′ Min.	3′ Min.
TWLT Lane Width			9.4	NA		Desirable: 14′ Minimum: 12′(8)	
Parking Lane Width (9)			17.1	Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
Median Width			13.2.9	NA		Existing (10)	
Right-of-Way Width			13.2.9	(11)		(11)	
Clear Zone			13.2.11	1.5′ (12)	See Section 13.2.11	1.5′ (12)	See Section 13.2.11
Side Slopes (13)	Cut	Fore Slope	13.2.9	NA	Existing	NA	Existing
		Ditch Width	13.2.9	NA	Existing	NA	Existing
		Back Slope	13.2.9	Existing	Existing	Existing	Existing
	Fill	0′-4′ Height	13.2.9	Existing	Existing	Existing	Existing
		4′-10′ Height	13.2.9	Existing	Existing	Existing	Existing
		> 10′ Height	13.2.9	Existing	Existing	Existing	Existing
				40 mph	45 mph	50 mph	60 mph
* Minimum Stopping Sight Distance (14)			5.7	275′	325′	400′	525′
Intersection Sight Distance			13.2.10	See Section 13.2.10			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		13.2.7	See Section 13.2.7		NA	NA
	emax=0.06		13.2.7	See Section 13.2.7			
* Superelevation Rate			13.2.7	See Section 13.2.7			
* Vertical Curvature for Desirable SSD	Crest		13.2.8	See Section 13.2.8			
	Sag		13.2.8	See Section 13.2.8			
Maximum Grade	Level		13.2.8	9%	8.5%	8%	7%
	Rolling		13.2.8	10%	9.5%	9%	8%
	Mountainous		13.2.8	12%	11.5%	11%	10%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%			
* New/Reconstructed Bridges	Structural Capacity (15)		–	HS-20/OK Overload Truck			
	Width		13.2.9	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-20 (Inventory Rating)			
	Width (State Highways) (16)		13.2.9	Uncurbed Sections: Travelway +4′		Curbed Sections: Site Specific	
* Vertical Clearance (17)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN PRINCIPAL ARTERIALS (3R Projects)

Footnotes to Table 13.2E

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. See the Planning Division for National Network system.

   b. If there are less than 10% trucks, lane widths may be 1′ less from table values.

   c. If there are 10% or more trucks, lane widths should desirably be 12′.

   d. Existing 11′ lanes may be retained if alignment and safety records are satisfactory, except for routes on the National Network or on reasonable access routes.

4. Shoulder/Curb Offset. The following will apply:

   a. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both the left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

   b. Where partial control of access is used, desirably the right shoulder width should be 8′ and the left shoulder width 4′.

5. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

6. Shoulder Cross Slope. For paved shoulders less than 4′, the shoulder cross slope will typically be the same as the travel lane.

7. Auxiliary Lanes (Shoulder/Curb Offset). The following will apply:

   a. Desirably the width adjacent to auxiliary lanes will be equal to the width adjacent to the travel lane.

   b. On facilities with curbs and where the design speed is 45 mph or less on the mainline, the curb offset (for both left and right) may be 1′ minimum for barrier curb and may be zero for mountable curb.

8. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

9. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should desirably be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane should be the same as that of the adjacent travel lane.

10. Median Widths. In most cases, the existing median width will be retained. However, in some situations, the best available alternative may be to reduce the existing median to increase the width of other cross section elements (e.g., lane and shoulders). Section 13.2.9 provides additional information on median widths.

11. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

12. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

13. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN PRINCIPAL ARTERIALS (3R Projects)

Footnotes to Table 13.2E (Continued)

14. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

15. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

16. Existing Bridges to Remain in Place. For all bridges where the bridge width is wider than those in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

    For bridges not on the State highway system, the bridge should be evaluated for widening if the bridge is uncurbed and is less than 100′ long and the usable width does not meet the following:

Design Year Volume (AADT)	*Usable Bridge Width (ft)*[a]
0-750	Width of approach travelway
751-2000	Width of approach travelway plus 2 ft
Over 2000	Width of approach travelway plus 4 ft

[a] If lane widening is planned as part of the 3R project, the usable bridge width should be compared with the planned width of the approaches after they are widened.

    Curbed section bridges will be evaluated on a site specific basis.

17. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2F
GEOMETRIC DESIGN CRITERIA FOR URBAN PRINCIPAL ARTERIALS (3R Projects)

Design Element			Manual Section	2-Lane With Curb	2-Lane Without Curb	Multilane With Curb	Multilane Without Curb
Standard Designation			–	UPA-1 (3R)	UPA-2 (3R)	UPA-3 (3R)	UPA-4 (3R)
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years		Desirable: 10 years	
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit		Posted Speed Limit	
Access Control			5.5	Control by Regulation		Control by Regulation/Partial Control	
Level of Service			5.4	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width			13.2.9	Desirable: 12′ Minimum: 10′ (3)		Desirable: 12′ Minimum: 10′ (3)	
Shoulder/Curb Offset	Type		13.2.9	Paved		Paved	
	* Width		13.2.9	2′ Min. (4a)	Des.: 8′ Min: 2′	Right: 2′ Left: 2′ (4a)	Right: 6′ Left: 2′ (4b)
Cross Slope	* Travel Lane		13.2.9	1.5% - 2%		1.5% - 2% (5)	
	Shoulder/Curb Offset (6)		13.2.9	Same as adjacent T.L.	2% - 4%	Same as adjacent T.L.	2% - 4%
Auxiliary Lanes	Lane Width		13.2.9	Desirable: Same as Travel Lane; Minimum: 10′		Desirable: Same as Travel Lane; Minimum: 10′	
	Shoulder/Curb Offset (7)		13.2.9	(7b)	2′ Min.	(7b)	2′ Min.
TWLT Lane Width			9.4	NA		Desirable: 14′ Minimum: 12′ (8)	
Parking Lane Width (9)			17.1	Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
Median Width			13.2.9	NA		Existing (10)	
Right-of-Way Width			13.2.9	(11)		(11)	
Clear Zone			13.2.11	1.5′ (12)	See Section 13.2.11	1.5′ (12)	See Section 13.2.11
Side Slopes (13)	Cut	Fore Slope	13.2.9	NA	Existing	NA	Existing
		Ditch Width	13.2.9	NA	Existing	NA	Existing
		Back Slope	13.2.9	Existing	Existing	Existing	Existing
	Fill	0′-4′ Height	13.2.9	Existing	Existing	Existing	Existing
		4′-10′ Height	13.2.9	Existing	Existing	Existing	Existing
		> 10′ Height	13.2.9	Existing	Existing	Existing	Existing

Design Element		Manual Section	30 mph	40 mph	45 mph	50 mph	60 mph
* Minimum Stopping Sight Distance (14)		5.7	200′	275′	325′	400′	525′
Intersection Sight Distance		13.2.10	See Section 13.2.10				
Decision Sight Distance		5.7	See Section 5.7				
* Maximum Degree of Curvature	emax=0.04	13.2.7	See Section 13.2.7				
	emax=0.06	13.2.7	See Section 13.2.7				
* Superelevation Rate		13.2.7	See Section 13.2.7				
* Vertical Curvature for Desirable SSD	Crest	13.2.8	See Section 13.2.8				
	Sag	13.2.8	See Section 13.2.8				
Maximum Grade	Level	13.2.8	10%	9%	8.5%	8%	7%
	Rolling	13.2.8	11%	10%	9.5%	9%	8%
	Mountainous	13.2.8	12%	12%	11.5%	11%	10%
Minimum Grade		7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%				
* New/Reconstructed Bridges	Structural Capacity (15)	–	HS-20/OK Overload Truck				
	Width	13.2.9	Full Approach Roadway Width				
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)				
	Width (State Highways) (16)	13.2.9	Uncurbed Sections: Travelway +4′ Curbed Sections: Site Specific				
* Vertical Clearance (17)	New/Replaced Bridges	7.4	16′-9″				
	Existing Bridges	7.4	14′-6″				

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR URBAN PRINCIPAL ARTERIALS (3R Projects)

Footnotes to Table 13.2F

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. See the Planning Division for National Network system.

   b. If there are less than 10% trucks, lane widths may be 1′ less from table values.

   c. If there are 10% or more trucks, lane widths should desirably be 12′.

   d. Existing 11′ lanes may be retained if alignment and safety records are satisfactory, except for routes on the National Network or on reasonable access routes.

4. Shoulder/Curb Offset. The following will apply:

   a. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both the left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

   b. Where partial control of access is used, desirably the right shoulder width should be 8′ and the left shoulder width 4′.

5. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

6. Shoulder Cross Slope. For paved shoulders less than 4′, the shoulder cross slope will typically be the same as the travel lane.

7. Auxiliary Lanes (Shoulder/Curb Offset). The following will apply:

   a. Desirably the width adjacent to auxiliary lanes will be equal to the width adjacent to the travel lane.

   b. On facilities with curbs and where the design speed is 45 mph or less on the mainline, the curb offset (for both left and right) may be 1′ minimum for barrier curb and may be zero for mountable curb.

8. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

9. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should desirably be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane should be the same as that of the adjacent travel lane.

10. Median Widths. In most cases, the existing median width will be retained. However, in some situations, the best available alternative may be to reduce the existing median to increase the width of other cross section elements (e.g., lane and shoulders). Section 13.2.9 provides additional information on median widths.

11. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

12. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

13. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

GEOMETRIC DESIGN CRITERIA FOR URBAN PRINCIPAL ARTERIALS (3R Projects)

Footnotes to Table 13.2F (Continued)

14. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

15. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

16. Existing Bridges to Remain in Place. For all bridges where the bridge width is wider than those in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

    For bridges not on the State highway system, the bridge should be evaluated for widening if the bridge is uncurbed and is less than 100′ long and the usable width does not meet the following:

Design Year Volume (AADT)	*Usable Bridge Width (ft)[a]*
0-750	Width of approach travelway
751-2000	Width of approach travelway plus 2 ft
Over 2000	Width of approach travelway plus 4 ft

[a] If lane widening is planned as part of the 3R project, the usable bridge width should be compared with the planned width of the approaches after they are widened.

Curbed section bridges will be evaluated on a site specific basis.

17. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2G
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN OTHER ARTERIALS
(3R Projects)

Design Element		Manual Section	2-Lane		Multilane	
			With Curb	Without Curb	With Curb	Without Curb
Standard Designation		–	SOA-1 (3R)	SOA-2 (3R)	SOA-3 (3R)	SOA-4 (3R)
Design Year (Geometrics) (1)		13.2.6	Desirable: 10 years		Desirable: 10 years	
* Design Speed (mph) (2)		13.2.6	Posted Speed Limit		Posted Speed Limit	
Access Control		5.5	Control by Regulation		Control by Regulation	
Level of Service		5.4	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width (3)		13.2.9	Desirable: 12′ Minimum: 11′		Desirable: 12′ Minimum: 11′	
Shoulder/Curb Offset	Type	13.2.9	Paved	Des: Paved; Min: Sod	Paved	Des: Paved; Min: Sod
	* Width	13.2.9	2′ Min. (4a)	4′	Right: 2′ Left: 2′ (4a)	Right: 6′(4b) Left: 4′
Cross Slope	* Travel Lane	13.2.9	1.5%-2%		1.5%-2% (5)	
	Shoulder/Curb Offset (6)	13.2.9	Same as adjacent T.L.	2%-6% Typical	Same as adjacent T.L.	2%-6% Typical
Auxiliary Lanes	Lane Width	13.2.9	Desirable: Same as Travel Lane; Minimum: 10′		Desirable: Same as Travel Lane; Minimum: 10′	
	Shoulder/Curb Offset (7)	13.2.9	2′ Min.	2′ Min.	2′ Min.	2′ Min.
TWLT Lane Width		9.4	NA		Desirable: 14′ Minimum: 12′ (8)	
Parking Lane Width (9)		17.1	Desirable: 12′ Minimum: 9′		Desirable: 12′ Minimum: 9′	
Median Width		13.2.9	NA		Existing (10)	
Right-of-Way Width		13.2.9	(11)		(11)	
Clear Zone		13.2.11	1.5′ (12)	See Section 13.2.11	1.5′ (12)	See Section 13.2.11
Side Slopes (13)	Cut: Fore Slope	13.2.9	NA	Existing	NA	Existing
	Cut: Ditch Width	13.2.9	NA	Existing	NA	Existing
	Cut: Back Slope	13.2.9	Existing	Existing	Existing	Existing
	Fill: 0′-4′ Height	13.2.9	Existing	Existing	Existing	Existing
	Fill: 4′-10′ Height	13.2.9	Existing	Existing	Existing	Existing
	Fill: > 10′ Height	13.2.9	Existing	Existing	Existing	Existing
* Minimum Stopping Sight Distance (14)			40 mph	45 mph	50 mph	60 mph
		5.7	275′	325′	400′	525′
Intersection Sight Distance		13.2.10	See Section 13.2.10			
Decision Sight Distance		5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04	13.2.7	See Section 13.2.7		NA	NA
	emax=0.06	13.2.7	See Section 13.2.7			
* Superelevation Rate		13.2.7	See Section 13.2.7			
* Vertical Curvature for Minimum SSD	Crest	13.2.8	See Section 13.2.8			
	Sag	13.2.8	See Section 13.2.8			
Maximum Grade	Level	13.2.8	9%	8.5%	8%	7%
	Rolling	13.2.8	10%	9.5%	9%	8%
	Mountainous	13.2.8	12%	11.5%	11%	10%
Minimum Grade		7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%			
* New/Reconstructed Bridges	Structural Capacity (15)	–	HS-20/OK Overload Truck			
	Width	13.2.9	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)			
	Width (State Highways) (16)	13.2.9	Uncurbed Sections: Travelway +4′ Curbed Sections: Site Specific			
* Vertical Clearance (17)	New/Replaced Bridges	7.4	16′-9″			
	Existing Bridges	7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN OTHER ARTERIALS (3R Projects)

Footnotes to Table 13.2G

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12'. See the Planning Division for National Network system.

   b. If there are less than 10% trucks, lane widths may be 1' less from table values.

   c. If there are 10% or more trucks, lane widths should desirably be 12'.

4. Shoulder/Curb Offset. The following will apply:

   a. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both the left and right) may be 1' minimum for barrier curbs and may be zero for mountable curbs.

   b. Where partial control of access is used, desirably the right shoulder width should be 8'.

5. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

6. Shoulder Cross Slope. For paved shoulders less than 4', the shoulder cross slope will typically be the same as the travel lane.

7. Auxiliary Lanes (Shoulder/Curb Offset). The following will apply:

   a. Desirably the width adjacent to auxiliary lanes will be equal to the width adjacent to the travel lane.

   b. On facilities with curbs and where the design speed is 45 mph or less on the mainline, the curb offset (for both left and right) may be 1' minimum for barrier curb and may be zero for mountable curb.

8. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16'.

9. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should desirably be 12' plus a 1' offset to the curb (if present). The cross slope of the parking lane should be the same as that of the adjacent travel lane.

10. Median Widths. In most cases, the existing median width will be retained. However, in some situations, the best available alternative may be to reduce the existing median to increase the width of other cross section elements (e.g., lane and shoulders). Section 13.2.9 provides additional information on median widths.

11. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

12. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10' from the edge of travel lane, if this yields a greater clear distance than the 1.5' from the gutter line. The 1.5' minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

13. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN OTHER ARTERIALS (3R Projects)

Footnotes to Table 13.2G (Continued)

14. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

15. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

16. Existing Bridges to Remain in Place. For all bridges where the bridge width is wider than those in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

    For bridges not on the State highway system, the bridge should be evaluated for widening if the bridge is uncurbed and is less than 100′ long and the usable width does not meet the following:

Design Year Volume (AADT)	*Usable Bridge Width (ft)*[a]
0-750	Width of approach travelway
751-2000	Width of approach travelway plus 2 ft
Over 2000	Width of approach travelway plus 4 ft

a If lane widening is planned as part of the 3R project, the usable bridge width should be compared with the planned width of the approaches after they are widened.

    Curbed section bridges will be evaluated on a site specific basis.

17. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2H
GEOMETRIC DESIGN CRITERIA FOR URBAN OTHER ARTERIALS
(3R Projects)

Design Element			Manual Section	2-Lane		Multilane	
				With Curb	Without Curb	With Curb	Without Curb
Standard Designation			–	UOA-1 (3R)	UOA-2 (3R)	UOA-3 (3R)	UOA-4 (3R)
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years		Desirable: 10 years	
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit		Posted Speed Limit	
Access Control			5.5	Control by Regulation		Control by Regulation	
Level of Service			5.4	Desirable: C Minimum: D		Desirable: C Minimum: D	
* Lane Width (3)			13.2.9	Desirable: 12′ Minimum: 10′		Desirable: 12′ Minimum: 10′	
Shoulder/Curb Offset	Type		13.2.9	Paved		Paved	
	* Width		13.2.9	2′ Min. (4a)	Des.: 6′ Min.: 2′	Right: 2′ Left: 2′ (4a)	Right: 6′ Left: 2′ (4b)
Cross Slope	* Travel Lane		13.2.9	1.5%-2%		1.5%-2% (5)	
	Shoulder/Curb Offset (6)		13.2.9	Same as adjacent T.L.	2%-4%	Same as adjacent T.L.	2%-4%
Auxiliary Lanes	Lane Width		13.2.9	Desirable: Same as Travel Lane; Minimum: 10′		Desirable: Same as Travel Lane; Minimum: 10′	
	Shoulder/Curb Offset (7)		13.2.9	2′ Min.	0′	2′ Min.	0′
TWLT Lane Width			9.4	NA		Desirable: 14′ Minimum: 12′ (8)	
Parking Lane Width (9)			17.1	Desirable: 12′ Minimum: 9′		Desirable: 12′ Minimum: 9′	
Median Width			13.2.9	NA		Existing (10)	
Right-of-Way Width			13.2.9	(11)		(11)	
Clear Zone			13.2.11	1.5′ (12)	See Section 13.2.11	1.5′ (12)	See Section 13.2.11
Side Slopes (13)	Cut	Fore Slope	13.2.9	NA	Existing	NA	Existing
		Ditch Width	13.2.9	NA	Existing	NA	Existing
		Back Slope	13.2.9	Existing	Existing	Existing	Existing
	Fill	0′-4′ Height	13.2.9	Existing	Existing	Existing	Existing
		4′-10′ Height	13.2.9	Existing	Existing	Existing	Existing
		> 10′ Height	13.2.9	Existing	Existing	Existing	Existing
* Minimum Stopping Sight Distance (14)			5.7	30 mph	40 mph	45 mph	50 mph
				200′	275′	325′	400′
Intersection Sight Distance			13.2.10	See Section 13.2.10			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		13.2.7	See Section 13.2.7			NA
	emax=0.06		13.2.7	See Section 13.2.7			
* Superelevation Rate			13.2.7	See Section 13.2.7			
* Vertical Curvature for Minimum SSD	Crest		13.2.8	See Section 13.2.7			
	Sag		13.2.8	See Section 13.2.8			
Maximum Grade	Level		13.2.8	10%	9%	8.5%	8%
	Rolling		13.2.8	11%	10%	9.5%	9%
	Mountainous		13.2.8	13%	12%	11.5%	11%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%			
* New/Reconstructed Bridges	Structural Capacity (15)		–	HS-20/OK Overload Truck			
	Width		13.2.9	Full Approach Roadway Width			
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-20 (Inventory Rating)			
	Width (State Highways) (16)		13.2.9	Uncurbed Sections: Travelway +4′ Curbed Sections: Site Specific			
* Vertical Clearance (17)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR URBAN OTHER ARTERIALS (3R Projects)

Footnotes to Table 13.2H

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. The following will apply:

   a. For highways on the National Network or on reasonable access routes, the minimum lane width is 12′. See the Planning Division for National Network system.

   b. If there are less than 10% trucks, lane widths may be 1′ less from table values.

   c. If there are 10% or more trucks, lane widths should desirably be 12′.

4. Shoulder/Curb Offset. The following will apply:

   a. Where the design speed is 45 mph or less on facilities with curbs, the curb offset (for both the left and right) may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

   b. Where partial control of access is used, desirably the right shoulder width should be 8′ and the left shoulder width should be 4′.

5. Travel Lane Cross Slope (Multilane). On curbed multilane facilities, the typical cross slope is 3% for any travel lanes adjacent to the curb.

6. Shoulder Cross Slope. For paved shoulders less than 4′, the shoulder cross slope will typically be the same as the travel lane.

7. Auxiliary Lanes (Shoulder/Curb Offset). The following will apply:

   a. Desirably the width adjacent to auxiliary lanes will be equal to the width adjacent to the travel lane.

   b. On facilities with curbs and where the design speed is 45 mph or less on the mainline, the curb offset (for both left and right) may be 1′ minimum for barrier curb and may be zero for mountable curb.

8. TWLT Lane Width. In industrial areas with large trucks turning frequently, the desirable TWLT lane width is 16′.

9. Parking Lanes. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should desirably be 12′ plus a 1′ offset to the curb (if present). The cross slope of the parking lane should be the same as that of the adjacent travel lane.

10. Median Widths. In most cases, the existing median width will be retained. However, in some situations, the best available alternative may be to reduce the existing median to increase the width of other cross section elements (e.g., lane and shoulders). Section 13.2.9 provides additional information on median widths.

11. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

12. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

13. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

GEOMETRIC DESIGN CRITERIA FOR URBAN OTHER ARTERIALS (3R Projects)

Footnotes to Table 13.2H
(Continued)

14. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

15. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

16. Existing Bridges to Remain in Place. For all bridges where the bridge width is wider than those in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

    For bridges not on the State highway system, the bridge should be evaluated for widening if the bridge is uncurbed and is less than 100′ long and the usable width does not meet the following:

Design Year Volume (AADT)	*Usable Bridge Width (ft)[a]*
0-750	Width of approach travelway
751-2000	Width of approach travelway plus 2 ft
Over 2000	Width of approach travelway plus 4 ft

[a] If lane widening is planned as part of the 3R project, the usable bridge width should be compared with the planned width of the approaches after they are widened.

    Curbed section bridges will be evaluated on a site specific basis.

17. Minimum Vertical Clearance. The vertical clearances apply to the arterial passing under a bridge. The following also apply:

    a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for an arterial passing beneath a new sign truss or pedestrian bridge and 17′-0″ for an arterial passing beneath an existing pedestrian bridge.

    b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath an arterial. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2I
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN COLLECTORS
(3R Projects)

Design Element			Manual Section	With Curb		Without Curb	
Standard Designation			--	SC-1 (3R)		SC-2 (3R)	
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years			
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit			
Access Control			5.5	Control by Regulation			
Level of Service			5.4	D			
* Lane Width (3)			13.2.9	Desirable: 12′ Minimum: 11′			
Shoulder/Curb Offset	Type		13.2.9	Paved		Desirable: Paved Minimum: Sod	
	* Width		13.2.9	2′ (4)		Desirable: 6′ Minimum: 2′	
Cross Slope	* Travel Lane		13.2.9	1.5%-2%			
	Shoulder/Curb Offset		13.2.9	Same as adjacent T.L.		Paved: 2%-4% (5) Sod: 6%-8%	
Auxiliary Lanes	Lane Width		13.2.9	Desirable: Same as Travel Lane; Minimum: 10′			
	Shoulder/Curb Offset		13.2.9	2′ (6a)		Desirable: 4′ Minimum: 2′ (6b)	
Parking Lane Width (7)			17.1	Desirable: 10′ Minimum: 7′			
Right-of-Way Width			13.2.9	(8)			
Clear Zone			13.2.11	1.5′ (9)		See Section 13.2.11	
Side Slopes (10)	Cut	Fore Slope	13.2.9	NA		Existing	
		Ditch Width	13.2.9	NA		Existing	
		Back Slope	13.2.9	Existing		Existing	
	Fill	0′-4′ Height	13.2.9	Existing		Existing	
		4′-10′ Height	13.2.9	Existing		Existing	
		> 10′ Height	13.2.9	Existing		Existing	
				30 mph	40 mph	45 mph	50 mph
* Minimum Stopping Sight Distance (11)			5.7	200′	275′	325′	400′
Intersection Sight Distance			13.2.10	See Section 13.2.10			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		13.2.7	See Section 13.2.7			NA
	emax=0.06		13.2.7	See Section 13.2.7			
* Superelevation Rate			13.2.7	See Section 13.2.7			
* Vertical Curvature for Minimum SSD	Crest		13.2.8	See Section 13.2.8			
	Sag		13.2.8	See Section 13.2.8			
Maximum Grade	Level		13.2.8	11%	11%	10%	9%
	Rolling		13.2.8	13%	12%	11%	10%
	Mountainous		13.2.8	14%	14%	13%	12%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%			
				DHV < 200		DHV 200-400	DHV > 400
* New/Reconstructed Bridges	Structural Capacity (12)		--	HS-20/OK Overload Truck			
	Width		13.2.9	With Curbs: App. Roadway Width; Without Curbs: Travelway + 6′			App. Roadway Width
* Existing Bridges to Remain in Place	Structural Capacity		--	HS-15 (Inventory Rating)			
	Width (State Highways) (13)		--	24′		26′	28′
	Width (Non-State Hwys)(13)		13.2.9	22′		24′	28′
* Vertical Clearance (14)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN COLLECTORS (3R Projects)

Footnotes to Table 13.2I

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. Lane widths must be 12′ if facility is on the National Network or on a reasonable access route. See the Planning Division for National Network system. If there are 10% or more trucks, lane widths should desirably be 12′. In residential areas and where right-of-way is restricted, 10′ lanes are acceptable.

4. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

5. Shoulder Cross Slope. For paved shoulders less than 4′, the shoulder cross slope will be the same as the travel lane.

6. Auxiliary Lanes (Shoulder/Curb Offset). The following will apply:

   a. On facilities with curbs and where the design speed is 45 mph or less on the mainline, the curb offset may be 1′ minimum for barrier curb and may be zero for mountable curb.

   b. Desirably the width adjacent to auxiliary lanes will be equal to the width adjacent to the travel lane.

7. Parking Lanes. In industrial or commercial areas, the recommended minimum parking lane width is 9′. The cross slope of the parking lane should be the same as that of the adjacent travel lane.

8. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

9. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

10. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

11. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

12. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

13. Width (Existing Bridges to Remain in Place). Widths in table are for uncurbed sections; curbed-section bridges will be evaluated on a site-specific basis. For bridges of more than 100′ in length and not on the State highway system, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

14. Minimum Vertical Clearance. The vertical clearances apply to the collector passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a collector. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2J
GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS
(3R Projects)

Design Element			Manual Section	With Curb		Without Curb	
Standard Designation			–	UC-1 (3R)		UC-2 (3R)	
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years			
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit			
Access Control			5.5	Control by Regulation			
Level of Service			5.4	D			
* Lane Width (3)			13.2.9	Desirable: 12′ Minimum: 10′			
Shoulder/Curb Offset	Type		13.2.9	Paved		Desirable: Paved Minimum: Sod	
	* Width		13.2.9	2′ (4)		Desirable: 6′ Minimum: 1′	
Cross Slope	* Travel Lane		13.2.9	1.5%-2%			
	Shoulder/Curb Offset		13.2.9	Same as adjacent T.L.		Paved: 2%-4% (5) Sod: 6%-8%	
Auxiliary Lanes	Lane Width		13.2.9	Desirable: Same as Travel Lane; Minimum: 10′			
	Shoulder/Curb Offset		13.2.9	2′ (6a)		Desirable: 4′ Minimum: 0′ (6b)	
Parking Lane Width (7)			17.1	Desirable: 10′ Minimum: 7′			
Right-of-Way Width			13.2.9	(8)			
Clear Zone			13.2.11	1.5′ (9)		See Section 13.2.11	
Side Slopes (10)	Cut	Fore Slope	13.2.9	NA		Existing	
		Ditch Width	13.2.9	NA		Existing	
		Back Slope	13.2.9	Existing		Existing	
	Fill	0′-4′ Height	13.2.9	Existing		Existing	
		4′-10′ Height	13.2.9	Existing		Existing	
		> 10′ Height	13.2.9	Existing		Existing	
				30 mph	40 mph	45 mph	50 mph
* Minimum Stopping Sight Distance (11)			5.7	200′	275′	325′	400′
Intersection Sight Distance			13.2.10	See Section 13.2.10			
Decision Sight Distance			5.7	See Section 5.7			
* Maximum Degree of Curvature	emax=0.04		13.2.7	See Section 13.2.7			NA
	emax=0.06		13.2.7	See Section 13.2.7			
* Superelevation Rate			13.2.7	See Section 13.2.7			
* Vertical Curvature for Minimum SSD	Crest		13.2.8	See Section 13.2.8			
	Sag		13.2.8	See Section 13.2.8			
Maximum Grade	Level		13.2.8	11%	11%	10%	9%
	Rolling		13.2.8	13%	12%	11%	10%
	Mountainous		13.2.8	14%	14%	13%	12%
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%			
				DHV < 200		DHV 200-400	DHV > 400
* New/Reconstructed Bridges	Structural Capacity (12)		–	HS-20/OK Overload Truck			
	Width		13.2.9	With Curbs: App. Roadway Width; Without Curbs: Travelway + 4′			App. Roadway Width
* Existing Bridges to Remain in Place	Structural Capacity		–	HS-15 (Inventory Rating)			
	Width (State Highways) (13)		–	24′		26′	28′
	Width (Non-State Hwys) (13)		13.2.9	20′		24′	28′
* Vertical Clearance (14)	New/Replaced Bridges		7.4	16′-9″			
	Existing Bridges		7.4	14′-6″			

* Controlling design criteria (See Section 5.8)

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS (3R Projects)

Footnotes to Table 13.2J

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. Lane widths must be 12′ if facility is on the National Network or on a reasonable access route. See the Planning Division for National Network system. If there are 10% or more trucks, lane widths should desirably be 12′. In residential areas and where right-of-way is restricted, 10′ lanes are acceptable.

4. Shoulder/Curb Offset. Where the design speed is 45 mph or less on facilities with curbs, the curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

5. Shoulder Cross Slope. For paved shoulders less than 4′, the shoulder cross slope will be the same as the travel lane.

6. Auxiliary Lanes (Shoulder/Curb Offset). The following will apply:

    a. On facilities with curbs and where the design speed is 45 mph or less on the mainline, the curb offset may be 1′ minimum for barrier curb and may be zero for mountable curb.

    b. Desirably the width adjacent to auxiliary lanes will be equal to the width adjacent to the travel lane.

7. Parking Lanes. In industrial or commercial areas, the recommended minimum parking lane width is 9′. The cross slope of the parking lane should be the same as that of the adjacent travel lane.

8. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

9. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

10. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

11. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

12. Structural Capacity (New/Reconstructed Bridges). The Oklahoma Overload Truck applies only to State highways.

13. Width (Existing Bridges to Remain in Place). Widths in table are for uncurbed sections; curbed-section bridges will be evaluated on a site-specific basis. For bridges of more than 100′ in length and not on the State highway system, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

14. Minimum Vertical Clearance. The vertical clearances apply to the collector passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a collector. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2K
GEOMETRIC DESIGN CRITERIA FOR SUBURBAN LOCAL STREETS**
(3R Projects)

Design Element			Manual Section	With Curb		Without Curb
Standard Designation			--	SL-1 (3R)		SL-2 (3R)
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years		
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit		
Access Control			5.5	Control by Regulation		
Level of Service			5.4	D		
* Lane Width (3)			13.2.9	Desirable: 11′ Minimum: 10′		
Shoulder/Curb Offset	Type		13.2.9	Paved		Chip Seal, Gravel or Dirt
	* Width		13.2.9	2′ (4)		Desirable: 6′ Minimum: 2′
Cross Slope	* Travel Lane		13.2.9	Paved/Chip Seal: 1.5%-2% Gravel: 2%-4% Dirt: 4%-6%		
	Shoulder/Curb Offset		13.2.9	Same as adjacent T.L.		Gravel: 4%-6% Sod: 6%-8%
Parking Lane Width (5)			17.1	7′ Typical		
Right-of-Way Width			13.2.9	(6)		
Clear Zone			13.2.11	1.5′ (7)		See Section 13.2.11
Side Slopes (8)	Cut	Fore Slope	13.2.9	NA		Existing
		Ditch Width	13.2.9	NA		Existing
		Back Slope	13.2.9	Existing		Existing
	Fill	0′-4′ Height	13.2.9	Existing		Existing
		4′-10′ Height	13.2.9	Existing		Existing
		> 10′ Height	13.2.9	Existing		Existing
				20 mph		30 mph
* Minimum Stopping Sight Distance (9)			5.7	125′		200′
Intersection Sight Distance			13.2.10	See Section 13.2.10		
Decision Sight Distance			5.7	See Section 5.7		
* Maximum Degree of Curvature	emax=0.04		13.2.7	See Section 13.2.7		
	emax=0.06		13.2.7	See Section 13.2.7		
* Superelevation Rate			13.2.7	See Section 13.2.7		
* Vertical Curvature for Minimum SSD	Crest		13.2.8	See Section 13.2.8		
	Sag		13.2.8	See Section 13.2.8		
Maximum Grade	Residential		13.2.8	15%		
	Commercial/Industrial		13.2.8	Desirable: 7% Maximum: 10%		
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%		
				AADT ≤ 750	AADT 751-2000	AADT > 2000
* New/Reconstructed Bridges	Structural Capacity		--	HS-20		
	Width (10)		13.2.9	Travelway + 4′	Travelway + 6′	Approach Roadway Width
* Existing Bridges to Remain in Place	Structural Capacity (11)		--	HS-15		
	Width (12)		13.2.9	20′	22′	24′
* Vertical Clearance (13)	New/Replaced Bridges		7.4	16′-9″		
	Existing Bridges		7.4	14′-6″		

* Controlling design criteria (See Section 5.8)
** Table will only apply where State and/or Federal funds are used.

GEOMETRIC DESIGN CRITERIA FOR SUBURBAN LOCAL STREETS (3R Projects)

Footnotes to Table 13.2K

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. In industrial areas, lane widths should be 12′ (desirable) and 11′ (minimum). In residential areas and where right-of-way is restricted, existing 9′ lanes are acceptable.

4. Shoulder/Curb Offset. The curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

5. Parking Lanes. In industrial or commercial areas, the recommended parking lane width is 9′. The cross slope of the parking lane should be the same as that of the adjacent travel lane.

6. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

7. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

8. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

9. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

10. Width (New and Reconstructed Bridges). Widths of bridges more than 100′ will be analyzed individually.

11. Structural Capacity (Existing Bridges to Remain in Place). Where the AADT ≤ 50, the minimum structural capacity is HS-10.

12. Width (Existing Bridges to Remain in Place). Where the AADT ≤ 750, the minimum width is 20′ or the approach travelway width, whichever is less. For bridges of more than 100′ in length, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

13. Minimum Vertical Clearance. The vertical clearances apply to the local street passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a local street. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

Table 13.2L
GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREETS**
(3R Projects)

Design Element			Manual Section	With Curb	Without Curb
Standard Designation			–	UL-1 (3R)	UL-2 (3R)
Design Year (Geometrics) (1)			13.2.6	Desirable: 10 years	
* Design Speed (mph) (2)			13.2.6	Posted Speed Limit	
Access Control			5.5	Control by Regulation	
Level of Service			5.4	D	
* Lane Width (3)			13.2.9	Desirable: 11′ Minimum: 10′	
Shoulder/Curb Offset	Type		13.2.9	Paved	Chip Seal, Gravel or Dirt
	* Width		13.2.9	2′ (3)	Desirable: 4′ Minimum: 0′
Cross Slope	* Travel Lane		13.2.9	Paved/Chip Seal: 1.5%-2%	Gravel: 2%-4% Dirt: 4%-6%
	Shoulder/Curb Offset		13.2.9	Same as adjacent T.L.	Gravel: 4%-6% Sod: 6%-8%
Parking Lane Width (4)			17.1	7′ Typical	
Right-of-Way Width			13.2.9	(5)	
Clear Zone			13.2.11	1.5′ (6)	See Section 13.2.11
Side Slopes (7)	Cut	Fore Slope	13.2.9	NA	Existing
		Ditch Width	13.2.9	NA	Existing
		Back Slope	13.2.9	Existing	Existing
	Fill	0′-4′ Height	13.2.9	Existing	Existing
		4′-10′ Height	13.2.9	Existing	Existing
		> 10′ Height	13.2.9	Existing	Existing
				20 mph	30 mph
* Minimum Stopping Sight Distance (8)			5.7	125′	200′
Intersection Sight Distance			13.2.10	See Section 13.2.10	
Decision Sight Distance			5.7	See Section 5.7	
* Maximum Degree of Curvature	emax=0.04		13.2.7	See Section 13.2.7	
	emax=0.06		13.2.7	See Section 13.2.7	
* Superelevation Rate			13.2.7	See Section 13.2.7	
* Vertical Curvature for Minimum SSD	Crest		13.2.8	See Section 13.2.8	
	Sag		13.2.8	See Section 13.2.8	
Maximum Grade	Residential		13.2.8	15%	
	Commercial/Industrial		13.2.8	Desirable: 7% Maximum: 10%	
Minimum Grade			7.1	Desirable: 0.5% Minimum: Curbed: 0.3%; Uncurbed: 0%	

Design Element		Manual Section	AADT ≤ 750	AADT 751-2000	AADT > 2000
* New/Reconstructed Bridges	Structural Capacity	–	HS-20		
	Width (10)	8.4	Travelway + 4′	Travelway + 6′	App Rd Wid
* Existing Bridges to Remain in Place	Structural Capacity (11)	–	HS-15		
	Width (12)	8.4	20′	22′	24′
* Vertical Clearance (13)	New/Replaced Bridges	7.4	16′-9″		
	Existing Bridges	7.4	14′-6″		

* Controlling design criteria (See Section 5.8)

** Table will only apply where State and/or Federal funds are used.

GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREETS (3R Projects)

Footnotes to Table 13.2L

1. Design Year AADT. Desirably, the design year AADT 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. See Section 13.2.6.

2. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

3. Lane Width. In industrial areas, lane widths should be 12′ (desirable) and 11′ (minimum). In residential areas and where right-of-way is restricted, existing 9′ lanes are acceptable.

4. Shoulder/Curb Offset. The curb offset may be 1′ minimum for barrier curbs and may be zero for mountable curbs.

5. Parking Lanes. In industrial or commercial areas, the recommended parking lane width is 9′. The cross slope of the parking lane should be the same as that of the adjacent travel lane.

6. Right-of-Way. The acquisition of significant amounts of right-of-way is often outside the scope of a 3R project. Therefore, the existing right-of-way will often be unchanged by the 3R project. However, the designer should, wherever possible, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements.

7. Clear Zone (Curbed Facilities). Desirably, the clear zone will be 10′ from the edge of travel lane, if this yields a greater clear distance than the 1.5′ from the gutter line. The 1.5′ minimum is measured from the gutter line and applies regardless of the shoulder or curb offset width.

8. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. Section 13.2.9 provides additional information for side slope criteria on projects with roadway widening (i.e., lane and/or shoulder widening).

9. Minimum Stopping Sight Distance. Values in the table are minimum SSD criteria when horizontal and/or vertical curves are reconstructed. See Sections 13.2.7 and 13.2.8 for additional criteria on existing horizontal and vertical curves which are to remain.

10. Width (New and Reconstructed Bridges). Widths of bridges more than 100′ will be analyzed individually.

11. Structural Capacity (Existing Bridges to Remain in Place). Where the AADT ≤ 50, the minimum structural capacity is HS-10.

12. Width (Existing Bridges to Remain in Place). Where the AADT ≤ 750, the minimum width is 20′ or the approach travelway width, whichever is less. For bridges of more than 100′ in length, the values in the table do not apply. The acceptability of these bridges will be assessed individually.

13. Minimum Vertical Clearance. The vertical clearances apply to the local street passing under a bridge. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a local street. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

13.2.6 Design Controls

13.2.6.1 Design Speed

In most cases, the existing posted or regulatory speed limit can be selected as the design speed on 3R projects. However, if the existing posted speed limit is likely to change after project completion, the designer should consult with the Traffic Engineering Division to determine an appropriate design speed for the project. On a case-by-case basis, the designer may request and Traffic may determine that a speed study within the project limits is necessary to establish a 3R design speed.

The design speed must be equal to or greater than the posted or regulatory speed limit after project completion, or a design exception will be required. See Sections 5.2 and 5.8 for additional information.

In summary, the selection of a 3R project design speed will be one of the following:

1. The existing posted or regulatory speed may be selected.
2. The 85th percentile speed based on a speed study may be selected. This may be especially applicable to intersection modification studies (e.g., intersection sight distance, length of auxiliary turn lanes).
3. The designer may select a design speed which is higher than the anticipated posted or regulatory speed limit, where deemed to be appropriate.
4. On projects of less than one-half mile length on facilities with unposted speed limits (typically intersections or bridge replacement projects), the selection of design speed should be made considering the operating speed, the overall characteristics of the corridor and the regulatory limit. If the selected design speed is less than the unposted regulatory speed limit, the project must provide for either a posted speed limit consistent with the design speed, or warning signs in combination with appropriate advisory speed signing. Such projects will not require a design exception.

13.2.6.2 Traffic Volume Analysis

The following traffic volume controls will apply to 3R projects:

1. Level of Service (LOS). Tables 13.2.A through 13.2L provide the desirable and minimum LOS criteria for 3R projects.
2. Design Year Traffic Volumes. Desirably, all elements of the facility will meet the LOS criteria for the DHV determined for 10 years beyond the expected construction completion date. At a minimum, the highway facility should meet the LOS criteria for the DHV based on current traffic volumes as determined at the construction completion date.
3. Traffic Data. The designer should obtain from the ODOT 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.
4. Capacity Analysis. The analytical techniques in the *Highway Capacity Manual* will be used to conduct the capacity analysis. See Reference 5.

13.2.6.3 Sight Distance

The criteria presented in Section 5.7 on sight distance applies equally to 3R projects. However, the application of the sight distance criteria to individual highway elements (e.g., vertical curves) on a 3R project will differ from that on a new construction/reconstruction project. These are discussed at the applicable locations elsewhere in this Section.

13.2.6.4 Adherence to Design Criteria

The discussion in Section 5.8 on design exceptions applies equally to the geometric design of 3R projects. However, the designer will evaluate the proposed design against the criteria presented in Chapter Thirteen.

13.2.7 Horizontal Alignment

Engineering judgment and/or a cost-effectiveness evaluation will ultimately reveal the need for improvements to the horizontal alignment within the 3R project. In general, improvements to the horizontal alignment should be considered if a specific problem is identified. Examples include:

1. a disproportionate run-off-the-road accident rate at curve sites,
2. a disproportionate number of multi-vehicle accidents at curve sites, or
3. the presence of an intersection within a horizontal curve.

The evaluation of potential improvements will include a consideration of traffic volumes, truck volumes, right-of-way and utility impacts, environmental impacts, driver expectancy, construction costs, etc.

13.2.7.1 Degree of Curvature/Superelevation

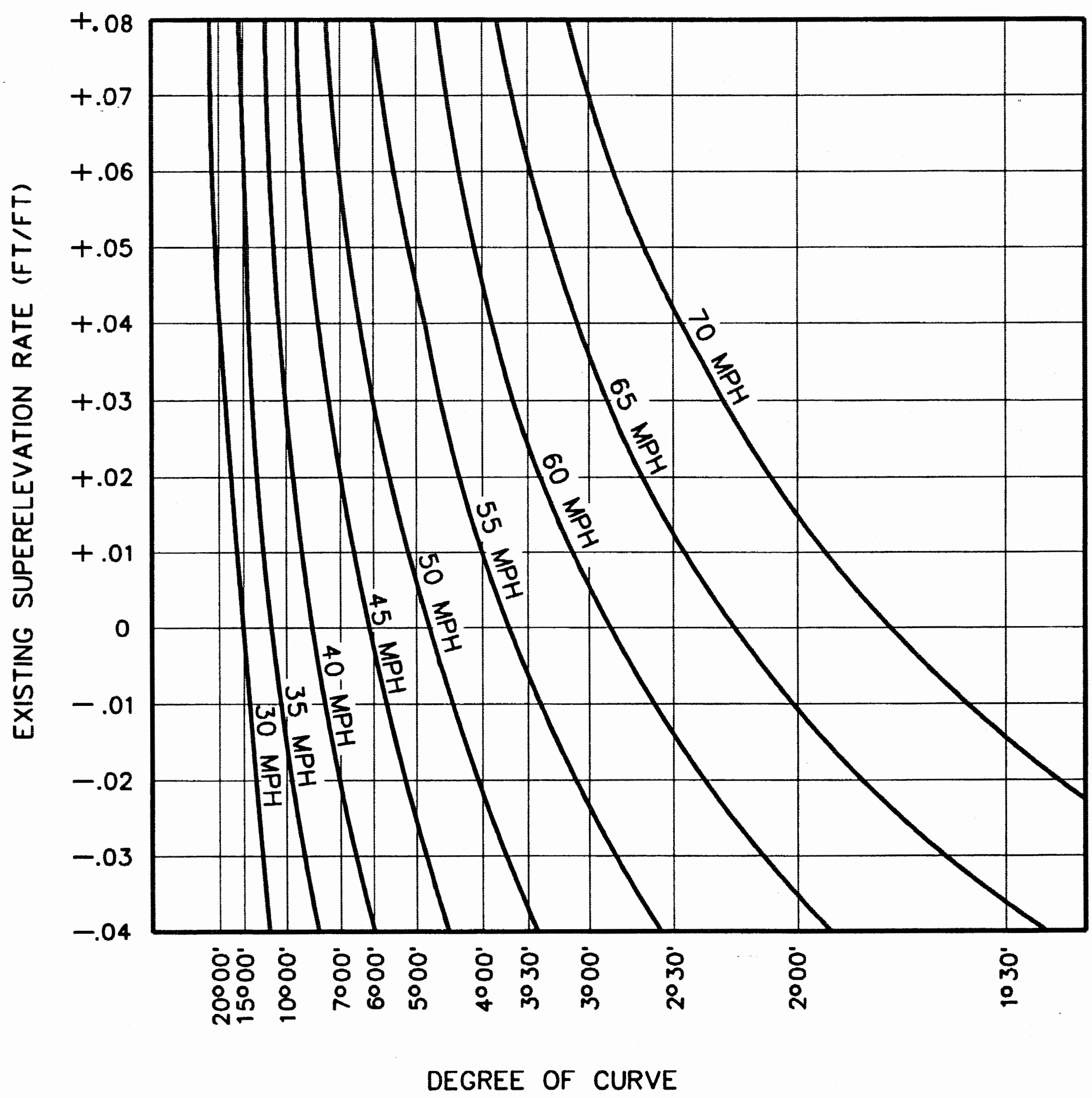
It is often impractical and unnecessary to correct curves on 3R projects to meet ODOT's new construction/reconstruction criteria for maximum degree of curvature. Consequently, existing horizontal curves should be considered for reconstruction if they meet one or both of the following conditions:

1. Where the accident data indicates a problem at the curve site.
2. Where the design speed of the existing curve is more than 10 mph below the design speed (assuming improved superelevation cannot reduce this difference) of the 3R project and the AADT is greater than 750 vehicles per day. Figure 13.2A should be used to determine the design speed of the existing curve. Figure 13.2A is based on AASHTO Method 2 for the distribution of superelevation and side friction (see Section 6.2).

Curves which do not meet these criteria (i.e., those that are not evaluated) are acceptable without improvement.

If it is determined to reconstruct the curve to meet the degree of curvature criteria, the curve should be reconstructed to meet all horizontal alignment details for new construction/reconstruction (e.g., superelevation rate, superelevation transition lengths, distribution of superelevation between tangent and curve), as discussed in Chapter Six.

If it is determined to retain the existing curve based on the above criteria, the designer may still be able to cost effectively improve other details of the horizontal curve. The frictional characteristics of the roadway may be improved with a new surface. The designer may specify a tapered overlay to increase the



Source: (1) Revised

NOTE: Figure applies to all rural highways at all design speeds and to urban facilities where the design speed is greater than 45 mph. For low-speed urban streets ($V \leq 45$ mph), the criteria in Section 6.2 will be used to determine the design speed of existing horizontal curves.

HORIZONTAL CURVATURE FOR 3R PROJECTS

Figure 13.2A

design speed of the curve by increasing the superelevation rate or by improving the superelevation development (transition length) at the curve to meet the new-construction criteria.

13.2.7.2 Reverse Curves

It will be generally acceptable to leave reverse curves in place on 3R projects (i.e., the PT & PC may be coincident). However, to determine if improvements are warranted, existing reverse curves should be evaluated in combination using the criteria in Section 13.2.7.1. The designer should also consider the impact one curve has on the other. An evaluation of the accident history will be especially important at existing reverse curves (e.g., multi-vehicle accidents).

13.2.7.3 Broken-Back Curves

Broken-back curves are closely spaced horizontal curves which provide deflection angles in the same direction. As an approximation, consecutive curves will be considered broken-back if the distance between the PT of the first curve and the PC of the second curve is less than 2 seconds of travel time at the design speed. These curve arrangements can interfere with smooth vehicular operation. For existing broken-back curves within the limits of a 3R project, the designer should, if practical, eliminate the curves and combine them into a single, continuous horizontal curve, especially where an evaluation of the accident history indicates a problem.

13.2.7.4 Travelway Widening

The use of travelway widening on a horizontal curve may be justified on a 3R project even where no other changes will be made to the curve. Section 6.4 presents warrants and applicable criteria for travelway widening.

13.2.7.5 Horizontal Sight Distance

Section 6.5 presents criteria for determining if the applicable sight distance is available at a horizontal curve. These criteria will apply to horizontal curves within the limits of a 3R project with the following modifications:

1. <u>Height of Object</u>. The height of eye and height of object will determine the line of sight at the curve. For 3R projects, the height of object will be 18 inches. Combined with a height of eye of 3.5 ft, the line-of-sight intercept with the obstruction will be 2.5 ft above the center of the inside travel lane.

2. <u>Longitudinal Barriers</u>. If an existing longitudinal barrier is the only obstacle which interferes with the line of sight at a horizontal curve, it generally will be acceptable to leave the barrier in its existing location, and it will <u>not</u> be necessary to seek a design exception.

13.2.7.6 Traffic Control Devices

For existing horizontal curves within a 3R project, a variety of traffic control devices may be considered to improve driver safety and comfort. These include:

1. signing (e.g., advance warning, chevron);

2. raised and/or standard pavement markers; and

3. reflective marker posts or delineators.

Chapter Fourteen and the MUTCD discuss the selection and installation of traffic control devices in more detail.

13.2.8 Vertical Alignment

13.2.8.1 Grades

Tables 13.2A through 13.2L present ODOT's criteria for maximum and minimum grades on 3R projects. The maximum grades are generally 2% steeper than those for new construction/reconstruction. Improvements to existing grades should be considered if a specific problem is identified. Examples include:

1. rear-end collisions,

2. head-on accidents due to improper passing maneuvers, or

3. evidence of pushed/rolled pavement distress at the bottom of steep downgrades.

13.2.8.2 Crest Vertical Curves

In most cases, existing crest vertical curves will be incorporated into the 3R project. However, the designer should consider reconstruction of a crest vertical curve if:

1. there is a history of accidents related to the vertical curve (e.g., rear-end accidents); or

2. the geometrics meet the following conditions:

   a. the crest hides from view major hazards such as intersections, sharp horizontal curves or narrow bridges; <u>and</u>

   b. the design speed of the existing crest (based on minimum stopping sight distance for passenger cars on level grade) is more than 20 mph below the 3R design speed using a 6-inch object height; <u>and</u>

   c. the design year AADT is greater than 1500 vehicles per day.

If it is determined to retain the existing curve based on the above criteria, the designer will not normally consider improving other geometric details of the crest vertical curve (e.g., minimum length, drainage maximum). If the decision is made to flatten the crest vertical curve on a 3R project, the following will apply:

3. Desirably, the vertical curve will be designed to meet the criteria for new construction/reconstruction using a 6-inch height of object (see Section 7.2).

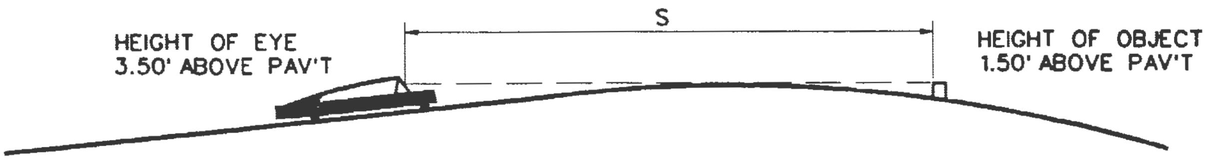
4. Under restricted conditions, an 18-inch height of object may be used for design purposes. Figure 13.2B presents the K-values for an 18-inch object height and a passenger car as the design vehicle.

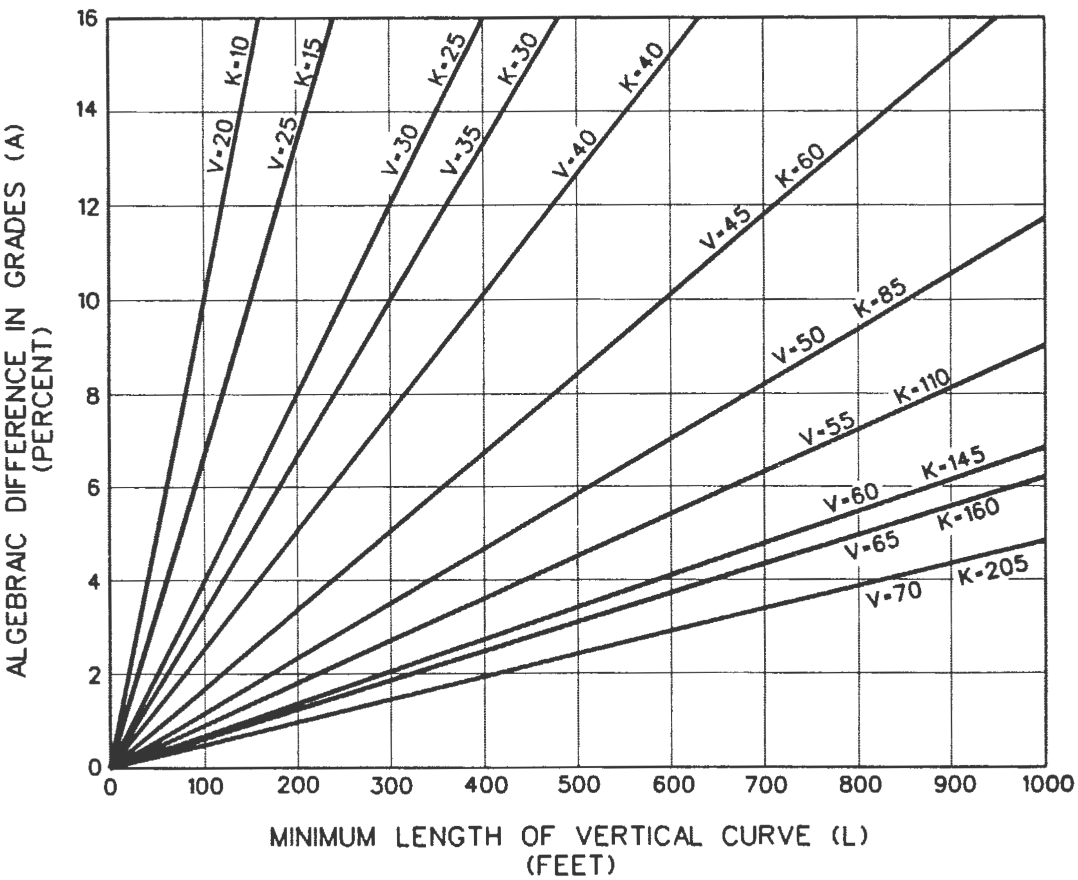
5. See Section 7.2 for additional criteria on the design of crest vertical curves (e.g., minimum length of curve, drainage considerations).

13.2.8.3 Sag Vertical Curves

Section 7.2 presents ODOT's criteria for the design of sag vertical curves for new construction/reconstruction. These criteria are based on designing the sag to allow the vehicular headlights to illuminate the pavement for a distance equal to the stopping sight distance for passenger cars. For 3R projects, the following will apply:

1. <u>Analysis</u>. The comfort criteria represent the minimum criteria for the retention of existing sag vertical curves. Figure 13.2C presents the comfort criteria. If an existing sag does not meet the criteria in Figure 13.2C, then the designer may consider flattening the sag vertical curve.





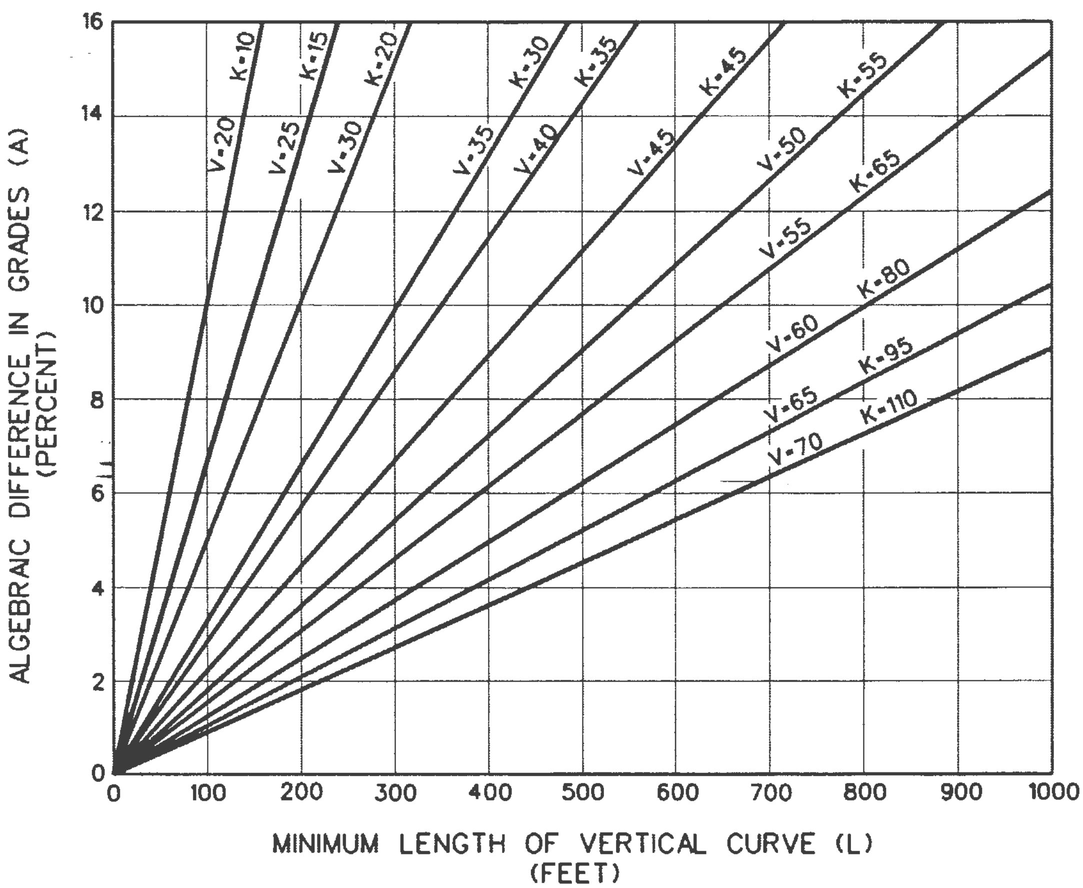
Source: (1) Revised

Note: Figure is based on the following:

1. passenger car,
2. level grade,
3. minimum stopping sight distance, and
4. 18″ height of object.

LENGTH OF CREST VERTICAL CURVE
(Minimum Stopping Sight Distance - 3R Projects)

Figure 13.2B



Source: (1) Revised

$$L = \frac{AV^2}{46.5} = KA$$

Where:

L = Length of vertical curve, ft
A = Algebraic difference between grades, %
K = Horizontal difference required to effect a 1% change in gradient

LENGTH OF SAG VERTICAL CURVES
(Comfort Criteria — 3R Projects)

Figure 13.2C

If it is determined to retain the existing curve based on the above criteria, the designer will not normally consider improving other geometric details of the sag vertical curve (e.g., minimum length, drainage maximum).

2. Corrective Action. If the decision is made to flatten the sag, the design desirably will meet the criteria for new construction/ reconstruction using headlight sight distance for passenger cars in Section 7.2. It is acceptable for the re-designed sag to meet the comfort criteria in Figure 13.2C.

13.2.8.4 Angle Points

It is acceptable to retain an existing "angle" point (i.e., no vertical curve) of 0.6% for crest vertical curves and 0.8% for sag vertical curves on a 3R project.

13.2.9 Cross Sections

Chapters Eight and Twelve present ODOT's criteria for cross section elements for new construction/reconstruction projects. The tables in Section 13.2.5 present the cross section criteria for 3R projects. In general, the criteria were established as follows:

1. Upper Limit. The upper limit (or "desirable") of the range was established as equal to the upper level for new construction criteria. On 3R projects, these still provide a desirable objective for the design of the cross section elements.

2. Lower Limit. The lower limit (or "minimum") of the range was established by considering the minimum acceptable width for the element from an operational and safety perspective; by considering what will be available for a practical improvement on a "typical" 3R project; and by 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 ODOT's 3R program.

The widths and/or steepness of the existing cross section should be evaluated against the criteria in the 3R tables. ODOT's policy on the application of the cross section widths and/or steepness in the tables is as follows:

3. Existing Width and/or Steepness Does Not Meet the Minimum 3R Criteria. The designer should consider widening and/or flattening the element. If the decision is made to widen and/or flatten the cross section element, the designer should provide a design which at least meets the minimum 3R criteria. This will be sufficient for the majority of 3R projects. However, if practical, it may be appropriate to widen or flatten the highway elements to meet the desirable 3R criteria.

4. Existing Width and/or Steepness Does Meet the Minimum 3R Width. In most cases, it will not be cost effective to widen or flatten the cross section element. However, if practical, it may be appropriate to widen or flatten the highway elements to meet the desirable 3R criteria.

5. Travel Lane Edge Dropoff. Even if the roadway width meets the minimum 3R criteria, the designer may consider roadway widening or the addition of a paved shoulder to remove any long extents of travel lane edge dropoff. See Chapter Sixteen for ODOT practices on pavement design.

6. The designer should check maintenance and safety records, and consult with personnel from the Field Division to determine which cross-section design

elements warrant documentation and evaluation for possible improvements.

The following sections summarize ODOT's 3R criteria for cross section elements.

13.2.9.1 Lane Width

3R projects should include practical improvements to the existing lane widths, if needed. Tables 13.2A through 13.2L present ODOT's lane width criteria for 3R projects.

13.2.9.2 Usable Shoulder Width

3R projects should include widening of the existing shoulders, if needed. Tables 13.2A through 13.2L present ODOT's usable shoulder width criteria for 3R projects.

13.2.9.3 Roadway/Paved Width

The following will apply to 3R projects:

1. It is general ODOT policy that a 3R project will not have a narrower roadway width than the existing facility.

2. It is general ODOT policy that an existing paved shoulder will not be converted to an unpaved shoulder.

13.2.9.4 Lane and Shoulder Cross Slope

On tangent sections, the lane and shoulder cross slopes on 3R projects should meet the criteria in Tables 13.2A through 13.2L.

Restoring or improving the pavement cross-slope is often considered cost-effective, resulting in improved ride, safety, drainage and maintainability of roadway pavements.

13.2.9.5 Auxiliary Lanes

Tables 13.2A through 13.2L present ODOT's criteria for lane and shoulder width for auxiliary lanes on 3R projects. These should be provided, if practical.

13.2.9.6 Climbing Lanes

The cross section criteria in Section 7.5 will apply to existing or proposed climbing lanes within the limits of 3R projects; however, for non-freeway projects, the following criteria are acceptable:

1. Lane Width. The minimum width of the climbing lane may be 11 ft.

2. Shoulder Width. The minimum width of the shoulder adjacent to the climbing lane may be 4 ft.

Note that shoulders adjacent to climbing lanes should be paved.

13.2.9.7 Curbs

On 3R projects, the following will apply to the installation or retention of curbs:

1. Types. Where a project will disturb existing curbs, the curb is generally replaced in-kind.

2. Height. 3R projects may include pavement work which will not affect the lateral location of existing curbs but will affect their finished height. The designer will consider adjusting the curb height (or the pavement design) if:

    a. an analysis of the storm water flow in the gutter indicates overtopping the curb for the design parameters (e.g., design-year frequency, ponding on roadway); and/or

b. the curb height after construction will be less than 3 inches.

3. Safety Considerations. On high-speed facilities ($V \geq 50$ mph), existing curbs may be removed for safety reasons, if they are not needed for drainage.

13.2.9.8 Sidewalks

Where a 3R project will disturb existing sidewalks, the sidewalk is generally replaced in-kind. Existing sidewalks may also be resurfaced when necessary. Where sidewalks do not currently exist, the need for sidewalks will be determined on a case-by-case basis as discussed in Section 8.1.

13.2.9.9 Median Width

The following will apply to medians on 3R projects:

1. Existing Medians. An existing multilane, divided highway (non-freeway) may be improved as a 3R project. If so, the existing median width will normally be retained.

2. New Medians. 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 meet ODOT criteria for new construction (see Section 8.2).

3. Raised Medians. In restricted locations, the minimum width of the raised island portion of a raised median may be 2 ft. This applies to both existing and proposed raised medians within 3R projects.

4. Brick Medians. Any existing brick medians should be removed if they are structurally unsound or if the bricks will be subjected to traffic loads. Otherwise, existing brick medians may remain.

13.2.9.10 Fill/Cut Slopes

The following will apply to fill and cut slopes within the limits of a 3R project:

1. No Roadway Widening. The existing fill and cut slopes will typically be retained.

2. Roadway Widening. If the lane and/or shoulders are widened as part of the 3R project, this will produce a steeper fill slope or ditch fore slope (assuming the toe of fill slope or toe of back slope remains in the same location). Desirably, the designer will modify the roadside design to provide a configuration which is the same as or flatter than the roadside cross section before the 3R project. However, it will be acceptable to provide a minimum reshaping of the roadside section to accommodate the widened roadway without adjusting the location of toe of fill slope or ditch fore slope.

   Where an existing fill slope will be steepened, the designer should consider appropriate benching or other stabilizing treatments for the revised slope.

3. Roadside Safety. Upgrading the roadside safety of the highway is often a major objective of the 3R project. On all 3R projects, the designer should consider the safety benefits of flattening fill and cut slopes to eliminate guardrail and to meet the criteria presented in Section 8.3 for new construction/reconstruction. An evaluation of run-off-the-road accidents will assist in the assessment.

13.2.9.11 Right-of-Way

As indicated in the basic definition of a 3R project, right-of-way acquisition will usually not be required or will be limited to small takings (e.g., easements for culvert extensions). Occasionally, more extensive right-of-way involvement may be appropriate if, for example, a horizontal curve is flattened. Wherever practical, additional right-of-way should be secured to allow cost-effective geometric and roadside safety improvements.

13.2.9.12 Existing Bridges

Tables 13.2A through 13.2L provide ODOT's criteria for structural capacity and widths of new and reconstructed bridges and for existing bridges to remain in place within the limits of a 3R project. If an existing bridge is structurally sound and if it meets ODOT's design loading for structural capacity, it is unlikely to be economical to improve the geometrics of the bridge. However, any geometric deficiencies and the accident experience at the bridge should be thoroughly analyzed. The following will apply to existing bridges within the limits of a 3R project:

1. Width. The width of the existing bridge should be evaluated against the criteria in Tables 13.2A through 13.2L. If the existing bridge does not meet these criteria, it should be evaluated for widening, including a review of the accident experience at the bridge.

2. Narrow Bridges. All bridges which are narrower than the approach roadway width (and will not be widened) 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 Bridge Division and Traffic Engineering Division, should evaluate the value of these additional treatments at the bridge site.

3. Bridge Rails. All existing bridge rails and their approaches will be evaluated to determine if they meet ODOT's current criteria. The Bridge Division will be responsible for the assessment of existing bridge rails and for making any recommendations for improvement. The designer is responsible for evaluating the adequacy of the approaching guardrail transitions.

Many bridge improvements are performed as spot improvements. Section 13.4 discusses ODOT's criteria for spot improvements.

13.2.10 Intersections At-Grade

Chapter Nine provides criteria for the detailed design of intersections at-grade for new construction/reconstruction. Wherever practical, these criteria apply to 3R projects and should be implemented. The following sections indicate areas where modifications to the intersection design criteria may be made for 3R projects.

13.2.10.1 General Design Controls

The criteria presented in Section 9.1 for intersection alignment, profile, design vehicle selection, etc., also apply to 3R projects, except where noted in the following:

1. Intersection Alignment. Preferably the angle of intersection should be within 20° of perpendicular. On 3R projects, existing intersections may be retained with angles up to 30°.

2. Y Intersections. On 3R projects, all existing Y intersections should desirably be converted to T intersections.

3. Design Vehicle Selection. Existing intersections should be checked to determine if the suggested design vehicle criteria in Table 9.1A can be accommodated using the criteria in Section 13.2.10.4 for turning radii. Intersections which are unable to accommodate the minimum design vehicle should be considered for reconstruction.

13.2.10.2 Intersection Sight Distance (ISD)

Often, it is difficult to provide ISD at existing intersections; for example, flattening a crest vertical curve to achieve the ISD criteria may be impractical. However, less expensive options may be practical to implement (e.g., relocating signs, utility poles and fences, removing trees, cutting back slopes). For 3R projects, the following will apply to ISD at stop-controlled intersections:

1. Desirable. If practical, the ISD criteria for new construction/reconstruction in Section 9.2 should be met. In particular, the designer should attempt to meet the minimum ISD criteria for new construction/reconstruction -- to provide 8 seconds of travel time for passenger cars and 12 seconds of travel time where trucks become the governing design vehicle.

2. Minimum. At a minimum, it is acceptable if the driver on the main road (i.e., free flowing) has stopping sight distance for passenger cars on level grades available to the intersection area, assuming a 6-inch height of object.

13.2.10.3 Auxiliary Lanes

Section 9.3 presents warrants for right- and left-turn lanes and design details for auxiliary turn lanes. These should be met if practical on 3R projects. However, the criteria for new construction/reconstruction may be impractical because of restricted site conditions. In general, the designer will provide the best design practical for the field conditions. Following are several specific examples of acceptable design criteria for auxiliary turn lanes on 3R projects:

1. Shoulders. Existing paved shoulders of sufficient width and pavement strength may be striped to indicate a separate right-turn lane at an intersection. If so, it may be necessary to rebuild and/or re-design the curb return to accommodate the selected design vehicle.

2. Reduced Travel Lane Width. It is acceptable to reduce the width(s) of the approaching travel lane(s) to provide a reasonable width for turn lanes. However, travel lanes should be at least 10-ft wide at the intersection and may warrant wider lanes if truck traffic turns must be accommodated.

3. Turn-Lane Widths. As indicated in Tables 13.2A through 13.2L, turn-lane widths on 3R projects may be narrower than those for new construction/reconstruction projects.

4. Length. The lengths for turn lanes should desirably include the components for taper, deceleration and storage as presented in Section 9.3. These criteria may be especially impractical on 3R projects, particularly the length for the vehicular deceleration component. However, the minimum turn-lane lengths in Section 9.3 apply to 3R projects.

5. Tapers (Right-Turn Lanes). For 3R projects on uncurbed facilities, it is acceptable to provide a squared-off design for a right-turn lane with a 2:1 painted stripe.

13.2.10.4 Turning Radii

Unless alerted by field personnel or where there is physical evidence of problems at an intersection (e.g., tire tracks over curbs, broken curbs, scraped utility poles), it will probably not be warranted to reconstruct the intersection to improve the turning radii design as part of the 3R project. However, once it has been determined to upgrade the intersection, the designer should desirably meet the criteria presented in Section 9.5. In urban areas, however, 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. Inside Clearance. The minimum inside clearance of the selected design vehicle may be zero; i.e., the inside tire track may "touch" the curb line or pavement edge.

2. Encroachment. Once a decision has been made to improve the intersection, the selected design vehicle should meet the encroachment criteria as discussed in Section 9.5.

3. Swept Width. The designer should review the existing or redesigned intersection with the turning templates to ensure that there are not obstacles in the swept path of the turning design vehicle.

4. Minor Intersections. At intersections with at least one leg considered a minor road, school buses, garbage trucks and fire trucks should be able to physically make the turn onto the minor street.

As a general summary of acceptable existing turning radii on 3R projects, the following will apply:

5. Passenger Cars. Simple radii of 15-25 ft are adequate for passenger vehicles. These radii may be retained on 3R projects on existing 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 it is restricted a sufficient distance from the intersection, and is used as a parking lane throughout the day.

6. SU Vehicles. Existing simple radii of 30 ft or simple radii with tapers (for the SU design vehicle) may be retained at all intersections and at all minor intersections.

7. Semi-Trailers. At intersections where semitrailer combinations and buses turn frequently, an existing simple radius of 40 ft or more may be retained. Preferably, the designer will use a radius with taper offsets for the selected design vehicle.

8. Pedestrians. Radii dimensions should be coordinated with crosswalks for pedestrians and handicapped individuals.

13.2.10.5 Driveways

For driveways on 3R projects, the designer should meet the criteria presented in the ODOT *Policy on Driveway Regulations for*

Oklahoma Highways and the *ODOT Standard Drawings*.

13.2.11 Roadside Safety

ODOT's objective on 3R projects is to implement practical and economical roadside safety improvements. The designer should review the roadside accident history and identify possible locations for roadside safety improvements. The following discussion offers roadside safety criteria which apply specifically to 3R projects.

13.2.11.1 Clear Zones

Attempting to achieve a roadside clear zone on a 3R project can cause significant problems, especially related to right-of-way. The roadside environment 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 judgement when determining the appropriate clear zone on the 3R project. The designer should consider the following:

1. Accident Records. 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 (e.g., on the outside of horizontal curves).

2. Location Relative to Clear Zone Distance. The closer an obstacle is to the traveled way, the greater the potential benefits of treatment. It is less likely to be cost effective to treat a hazard near the outer edge of the clear zone boundary.

3. Location Relative to Other Hazards. If a hazard is one of many at about the same distance from the traveled way, this decreases the benefits of treatment. As an example, it may have little benefit to remove an obstacle 12 ft from the travel lane if a line of other obstacles (e.g., trees) are located at 15 ft from the travel lane. However, it may be beneficial to treat an isolated hazard along the roadside which is within the clear zone distance.

4. Treatment Costs. A hazard may be removed, relocated, shielded or made breakaway. The costs of these treatments will be a significant factor in the decision-making process.

5. Nature of Hazard. The type of hazard and the available treatments will be a significant factor in the decision-making process. For example, a non-breakaway sign post, which is owned and maintained by ODOT, can be made breakaway without any impact on the surrounding environment. However, removing natural features (e.g., trees) may impact the environment and may meet with strong public opposition.

6. Utilities. Utility poles are a common roadside obstacle on 3R projects. Relocation is mandatory when the utility poles physically interfere with construction or when their placement is inconsistent with ODOT's *Utility Recommendations Policy*. Relocation for safety benefits must be evaluated on a case-by-case basis.

7. Barrier Protection. The designer should realize that the barrier warrants in Chapter Eleven are based on the relative severity between hazard and guardrail; they do not address the question of

whether or not a barrier installation is cost effective. On 3R projects, the designer must judge whether or not a barrier should be installed when a hazard is within the clear zone and will be left in place.

Section 11.2 presents the specific numerical criteria for roadside clear zones on new construction/reconstruction projects. In addition to the preceding general discussion, the clear zone criteria for 3R projects will be as follows:

8. Desirable. The most desirable objective will be to provide a clear zone equal to the criteria in Section 11.2.

9. Acceptable. Table 13.2M presents acceptable target values for clear zones on 3R projects adjusted for the project design speed, slope condition and traffic volumes. These values are judged to be reasonable considering the scope of a 3R project. Note that the procedures in Section 11.2 for clear zones on non-recoverable fill slopes and across ditch sections apply to 3R projects, except that Table 13.2M will be used as the basis for the analysis.

10. Curbed Sections. For urban streets with curbs, the minimum clear zone distance is 1.5 ft from the face of curb; this is a clear dimension to the obstacle, not to its center. See Section 11.2 for additional details on the application of the clear zone criteria on curbed sections.

11. Horizontal Curves. Although desirable, the horizontal curve correction in Section 11.2 will normally not be applicable on 3R projects.

12. Right-of-Way. If the clear zone as determined in the above comments exceed the limits of the existing right-of-way, the minimum clear zone distance will be the existing right-of-way line. If permanent right-of-way is purchased for other reasons, sufficient right-of-way should be purchased to provide the clear zone as determined in the above comments.

Note: The clear zones for 3R freeway projects are presented in Section 11.2 and Section 13.3.

13.2.11.2 Safety Appurtenances

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 and bridge parapets. Normally, substandard safety appurtenances will be upgraded to meet the most recent criteria. Chapter Eleven presents ODOT's criteria for the layout and design of safety appurtenances.

Roadside barrier warrants on 3R projects can be especially difficult to resolve. The designer should evaluate the roadside environment against the criteria in Chapter Eleven. Basically, the process will be:

1. Determine if barrier is warranted. However, also see Comment #7 "Barrier Protection" in Section 13.2.11.1.

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 its severity (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 barrier. All new installations of guardrail will meet the criteria set forth in Chapter Eleven and in the *ODOT Standard Drawings.*

Table 13.2M

CLEAR ZONE DISTANCE (FT)
(3R Non-Freeway Projects Only)

Design Speed	Design ADT	Fill Slopes			Cut Slopes		
		6:1 or flatter	5:1 to 4:1	3:1	3:1	4:1 to 5:1	6:1 or flatter
40 mph or less	Under 750	6	6	See Procedure in Section 11.2.2	6	6	6
	750-1500	6	6-7		6	6	6
	1500-6000	6-7	7-8		6-7	6-7	6-7
	Over 6000	7-8	8-9		7-8	7-8	7-8
45-50 mph	Under 750	6	6-7		6	6	6
	750-1500	7-8	8-10		6	6-7	7-8
	1500-6000	8-9	10-13		6-7	7-8	8-9
	Over 6000	10-11	12-14		7-8	9-10	10-11
55 mph	Under 750	6-7	7-9		6	6	6
	750-1500	8-9	10-12		6	7-8	8-9
	1500-6000	10-11	12-15		7-8	8-9	10-11
	Over 6000	11-12	13-16		8-9	10-11	11-12
60 mph	Under 750	8-9	10-12		6	6-7	7-8
	750-1500	10-12	13-16		6-7	8-9	10-11
	1500-6000	13-15	16-20		7-9	9-11	12-13
	Over 6000	15-16	18-22		10-11	12-13	13-14
65-70 mph	Under 750	9-10	10-13		6	7-8	7-8
	750-1500	12-13	14-18		6-8	9-10	10-11
	1500-6000	14-16	17-21		8-10	11-12	13-14
	Over 6000	15-17	19-23		11-12	13-15	14-15

Notes:

1. All distances are measured from the edge of the travel lane.
2. See discussion in Section 11.2.2 (Comment #2) for application of clear zone criteria on non-recoverable fill slopes.
3. See discussion in Section 11.2.3 for application of clear zone criteria across ditch sections.
4. For clear zones, the "Design ADT" will be the total ADT on two-way roadways and directional ADT on one-way roadways (e.g., ramps and one roadway of a divided highway).

13.2.11.3 Existing Barriers

For existing runs of barrier which will remain, ensure that they meet, as practical, the applicable performance and design criteria presented in Chapter Eleven, including:

1. operational acceptability (hardware, height, etc.);
2. dynamic deflection criteria;
3. length of need;
4. lateral placement;
5. placement on slopes and behind curbs; and
6. terminal treatments.

For runs of existing barrier on 3R projects, the designer especially should consider the following:

7. Barrier Height. A common problem on 3R projects will be the height of existing installations because of a pavement overlay. Each existing run which will remain must be considered individually. As a general rule, the designer should consider raising the barrier when its height, after construction, will be more than 3 inches below the standard height.
8. Slopes in Front of Barrier. It will be acceptable to retain existing installations on 6:1 or flatter slopes where the installation is otherwise acceptable. Where a flared section crosses a ditch section, it may be necessary to place a pipe and regrade through the ditch to meet this criteria.

   Existing barrier installations may have terminal sections which flare away and terminate on a slope steeper than 10:1, but flatter than or equal to 6:1. If no other barrier deficiencies exist, and the accident history does not indicate a problem, it will be acceptable to leave the existing installation in place.
9. Length of Need. If the required length of need as determined from Section 11.5 is within 10% of that of the existing barrier to remain in place, then it will rarely be warranted to increase the length of barrier to meet the criteria in Section 11.5, provided that there is not a run-off-the-road accident problem related to the length of need.
10. Lateral Placement. Where guardrail is placed to shield a fill slope, there will desirably be 2 ft between the back of the barrier post and the slope break (PI) in the fill slope. On 3R projects, this may not be practical. Therefore, a longer guardrail post should be used so that the barrier can be placed at the break in the slope.

13.2.12 Traffic Engineering Elements

A 3R project may include upgrading of signs, pavement markings, signals and/or lighting. New installations or the replacement of these design traffic elements should meet the criteria presented in Chapter Fourteen and the MUTCD. The designer should consider upgrading or replacing any existing substandard traffic elements when warranted.

The designer must coordinate with the Traffic Engineering Division to determine the appropriate signing and/or markings for existing geometric elements which do not meet the 3R criteria and will remain.

13.3 3R FREEWAY PROJECTS

13.3.1 Background

ODOT began construction of its freeway system in the 1950's and, today, the Oklahoma system has been completed. 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 higher design speeds. In the past few decades, this system has carried freeway 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/or rehabilitation) which applies to any project on an existing freeway. As with non-freeway 3R projects, it is impractical to fully apply new construction criteria to 3R freeway projects. Therefore, the geometric design of 3R freeway projects requires special design considerations which are discussed in the following sections.

Note that 3R freeway projects do not include reconstruction. When the scope of the project work is judged to be reconstruction, the project will be designed according to the applicable criteria for new construction.

13.3.2 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, including:

1. structural adequacy,
2. drainage,
3. level of service for the traffic flow,
4. geometric design,
5. roadside safety, and
6. traffic control.

13.3.3 Approach

3R freeway projects are most often initiated to make a specific improvement to the freeway. Therefore, ODOT's approach to the geometric design of 3R freeway projects is to selectively evaluate and improve the existing geometrics. The 3R approach is summarized as follows:

1. Nature of Improvements. Identify the specific improvements intended for the 3R project. For example, project improvements might include:

    a. pavement improvements, including full-depth pavement reconstruction;

    b. widening existing travel lanes or shoulders;

    c. improving the superelevation of existing horizontal curves;

    d. adding auxiliary lanes;

    e. improving roadway delineation;

    f. upgrading roadside safety;

    g. increasing the length of acceleration or deceleration lanes at an interchange;

    h. widening an existing bridge as part of a bridge rehabilitation project;

    i. eliminating a weaving area at an interchange; and/or

    j. improving the roadside drainage.

3R freeway work does not include, for example, the addition of travel lanes or significant improvements to the existing horizontal and vertical alignment. This level of work is typically considered reconstruction.

2. Numerical Criteria. Table 13.3A presents ODOT's numerical criteria for the design of 3R freeway projects. In particular, the footnotes to the table provide essential information on 3R projects. In addition, the information in the remainder of Section 13.3 provides numerical criteria on other geometric design elements on 3R projects.

3. Secondary Impacts. Identify and evaluate any secondary impacts which may be precipitated by the freeway improvement. For example:

   a. The installation of a CMB may restrict horizontal sight distance.

   b. The addition of through lanes on the outside may reduce the available roadside clear zone to a level below ODOT criteria.

   c. A pavement overlay may require the adjustment of barrier height.

4. Other Improvements. Identify any geometric design deficiencies within the project limits which can be practically corrected without exceeding the intended project scope of work. A review of the accident history is important in conducting this evaluation.

5. Exceptions. The discussion in Section 5.8 on design exceptions applies equally to the geometric design of 3R freeway projects using the criteria in this section.

13.3.4 Geometric Design of 3R Freeway Projects

Table 13.3A presents ODOT's criteria for the geometric design of 3R freeway projects. However, the designer must still make certain decisions, and there is some flexibility that can be applied. These are discussed in the following sections.

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 Interstate standards that were in effect at the time of original construction or inclusion into the Interstate system.

13.3.4.1 Design Speed

Chapter Twelve presents ODOT's criteria for selecting the design speed for new freeway construction. Desirably, these will also apply to 3R projects. The existing posted speed limit may be acceptable as the minimum design speed. However, the designer should check with the Traffic Engineering Division to determine if the existing posted speed limit is likely to change after project completion or if the original construction design speed was higher than the posted speed limit.

The design speed must be equal to or greater than the posted or regulatory speed limit after project completion, or a design exception will be required. See Sections 5.2 and 5.8 for additional information.

In summary, the selection of a 3R project design speed will be one of the following:

1. The existing posted or regulatory speed may be selected.

2. The 85th percentile speed based on a speed study may be selected.

Table 13.3A
GEOMETRIC DESIGN CRITERIA FOR FREEWAYS
(3R Projects)

Design Element			Manual Section**	Rural	Urban
Standard Designation			–	RF (3R)	UF (3R)
Design Year (Geometrics)			5.3	Desirable: 20 years Minimum: 10 years	Desirable: 20 years Minimum: 10 years
* Design Speed (mph) (1)			5.2	Minimum: Posted Speed Limit	Minimum: Posted Speed Limit
Access Control			5.4	Full Control	Full Control
Level of Service			5.3	Desirable: B Minimum: C	Desirable: C Minimum: D
* Lane Width			8.1	12′	12′
Shoulder	Type		8.1	Paved	Paved
	* Width		8.1	Right: 10′ (2a) Left: 4′ (2a & 2b)	Right: 10′ (2a) Left: 4′ (2a & 2b)
Cross Slope	* Travel Lane		8.1	2% Typical	2% Typical
	Shoulder		8.1	3%-4%	3%-4%
Auxiliary Lanes	Lane Width		8.1	12′	12′
	Shoulder Width		8.1	Desirable: Full Shoulder Width Minimum: 6′	Desirable: Full Shoulder Width Minimum: 6′
Median Width	With Barrier		8.2	N/A	(3)
	Without Barrier		8.2	Existing	Existing
Right-of-Way Width			8.6	(4)	(4)
Clear Zone			11.2	See Section 11.2	See Section 11.2
Side Slopes (5)	Cut	Fore Slope	8.3	Existing	Existing
		Ditch Width	8.3	Existing	Existing
		Back Slope	8.3	Existing	Existing
	Fill	0′-4′ Height	8.3	Existing	Existing
		4′-10′ Height	8.3	Existing	Existing
		>10′ Height	8.3	Existing	Existing

Design Element	Manual Section**	50 mph	55 mph	60 mph	65 mph	70 mph
* Minimum Stopping Sight Distance	5.7	400′	450′	525′	550′	625′

Design Element		Manual Section**	Rural	Urban
Decision Sight Distance		5.7	See Section 5.7	See Section 5.7
* Maximum Degree of Curvature	e_{max} = 0.06	6.1	Existing	Existing
	e_{max} = 0.08	6.1	Existing	Existing
* Superelevation Rate (6)		6.2	Existing	Existing
* Vertical Curvature for Desirable SSD	Crest	7.2	Existing	Existing
	Sag	7.2	Existing	Existing
* Maximum Grade	Level	7.1	Existing	Existing
	Rolling	7.1	Existing	Existing
	Mountainous	7.1	Existing	Existing
Minimum Grade		7.1	Desirable: 0.5% Minimum: 0%	Desirable: 0.5% Minimum: 0%
* New/Reconstructed Bridges	Structural Capacity (7)	–	HS-20/OK Overload Truck	HS-20/OK Overload Truck
	Width	8.4	Full Approach Roadway Width	Full Approach Roadway Width
* Existing Bridges to Remain in Place	Structural Capacity	–	HS-20 (Inventory Rating)	HS-20 (Inventory Rating)
	Width (8)	8.4	2 lanes: 28′ 3 lanes: 40′	2 lanes: 28′ 3 lanes: 40′
* Vertical Clearance (9a & 9b)	New/Replaced Bridges	7.4	16′-9″	16′-9″
	Existing Bridges	7.4	14′-6″ (9c)	14′-6″ (9c)

* Controlling design criteria (See Section 5.8)

** See also Section 13.3 for specific discussion on 3R projects.

GEOMETRIC DESIGN CRITERIA FOR FREEWAYS (3R Projects)

Footnotes to Table 13.3A

1. Design Speed. The design speed should equal or exceed the anticipated posted or regulatory speed limit after construction.

2. Shoulder Width. The following will apply:

   a. Both Shoulders. When the volume of trucks exceeds 250 DDHV, both the right and left shoulders will desirably be 12′.

   b. Left Shoulder. With three or more lanes in one direction, a 10′ shoulder should be used.

   All shoulder widths should desirably be increased by 2′ when a barrier is present.

3. Median Width. Expansion of the median width is typically outside the scope of a 3R project. However, when widening or adding additional lanes, the following should be considered:

   a. A 10′ minimum median width should be provided on a freeway with 4 lanes.
   b. A 22′ minimum median width should be provided on a freeway with more than 4 lanes.
   c. Where light poles, glare screens, etc., will be mounted on the median barrier, the desirable median width is 22′-26″.
   d. When the volume of trucks exceeds 250 DDHV, the recommended minimum median width is 26′.

4. Right-of-Way Width. The acquisition of significant amounts of right-of-way may be outside the scope of a 3R project. The designer should, wherever practical, secure additional right-of-way to allow cost-effective geometric and roadside safety improvements, especially where outside auxiliary lanes are added.

5. Side Slopes. In most cases, retention of the existing side slope shape according to the original design will be acceptable. However, existing fill slopes steeper than 3:1 should be considered for flattening.

6. Superelevation Rate. Normally, the existing superelevation rate will be adequate. Where the pavement will be reconstructed, it is typically practical to improve the superelevation rate, if needed. However, the designer should refer to Section 6.2 to determine if any improvements should be considered.

7. Structural Capacity (New/Reconstructed Bridges). The Alternate Military Loading will also apply to the Interstate system. The Oklahoma Overload Truck only applies to State highways.

8. Width (Existing Bridges to Remain in Place). For bridge widths wider than the widths in the table, the bridge should be evaluated for widening to full approach roadway width only if one of the condition codes from the NBIS inspection report is less than 5 (deck, superstructure or substructure) or if the approach roadway width is widened.

9. Minimum Vertical Clearance. The vertical clearances apply to the freeway passing under a bridge. The following also apply:

   a. Sign Truss/Pedestrian Bridge. The minimum clearance is 18′-0″ for a freeway passing beneath a new sign truss or pedestrian bridge and 17′-0″ for a freeway passing beneath an existing pedestrian bridge.

   b. Railroads. The Planning Division (Rail Planning Branch) will determine the vertical clearance for railroads passing beneath a freeway. The typical clearance will be 23′-0″. An allowance should also be made for future ballasting.

   c. Urban Areas. A minimum clearance of 14′-6″ may be retained in urban areas if there is an alternative freeway facility with the minimum 16′-0″ clearance.

3. The designer may select a design speed which is higher than the anticipated posted or regulatory speed limit, where deemed to be appropriate.

13.3.4.2 Design Hourly Volume (DHV)

Some design elements on 3R freeway projects will require the selection of a DHV. Desirably, the geometric elements of the freeway will be designed to meet the level-of-service criteria for a DHV determined for 20 years beyond the expected completion date. At a minimum, the DHV will be determined for 10 years beyond the completion date.

13.3.4.3 Horizontal/Vertical Alignment

Unless the specific objective of the 3R freeway project is to improve one or more horizontal/vertical features, the existing alignment will normally be acceptable, if a review of the accident history does not indicate a problem. Once the decision has been made to reconstruct a horizontal/vertical curve, the designer will apply the criteria in Chapters Six and Seven.

13.3.4.4 Vertical Clearances

The minimum vertical clearance should desirably be 16′-0″. In highly developed urban areas, where attaining the 16-ft clearance would result in unreasonable costs, the existing clearance as low as 14′-6″, may be maintained, if there is an alternative freeway facility with the minimum 16′-0″ clearance.

13.3.4.5 Bridges

The following discusses ODOT's design criteria for bridges within the limits of a 3R freeway project:

1. <u>Bridges to Remain in Place</u>. A 3R project may be primarily intended, for example, to improve the pavement condition over several miles. A bridge or several bridges may be within the limits of the 3R project. Desirably, the bridge width will equal the approach roadway width, including shoulders. At a minimum, the existing roadway width on the bridge will equal the approaching travelway width plus 3 ft on each side. However, this may not be the case. If the existing bridge is structurally sound and if it meets ODOT's design loading structural capacity, it is unlikely to be cost effective to improve the geometrics of the bridge. However, geometric deficiencies may exist, and/or there may be an adverse accident experience at the bridge. In this case, it may be warranted to widen the bridge as part of the 3R project.

   Existing bridge rails and the approach transitions should be evaluated to determine if they meet ODOT's current criteria. The Bridge Division will be responsible for assessing bridge rails and for making any recommendations for improvement. The designer is responsible for evaluating the adequacy of the approach guardrail transitions.

2. <u>Bridge Replacement/Rehabilitation</u>. 3R freeway projects will often include bridge replacement or bridge rehabilitation and, in some cases, this will be the entire project scope of work. The following will apply to the geometric design of these projects:

   a. <u>Horizontal and Vertical Alignment</u>. An existing bridge may have an alignment which does not meet ODOT's current criteria. For bridge replacement projects, the designer should evaluate the practicality of realigning the bridge to meet the applicable alignment criteria for new construction. For bridge rehabilitation projects, it is unlikely to be cost effective to realign the bridge to correct any alignment deficiencies.

b. Width. The bridge width should equal the full approach roadway width, including shoulders, as determined by the criteria in Chapter Twelve for the most likely level of future highway improvement on the approaches. If practical, this decision should be based on a capacity analysis for the selected DHV at the selected level of service. This analysis could determine the need for additional travel lanes and/or the need for wider shoulders. For example, if the predicted volume of trucks exceeds 250 DDHV, the future shoulder width on the approach should be 12 ft. Because freeway bridges represent major economic investments with lengthy design lives, it may be warranted to provide the wider widths as part of a bridge replacement or rehabilitation project.

As another example, a capacity analysis may indicate the need for an additional through lane to meet the level-of-service criteria for the design year. The decision may be made to widen the bridge as part of the replacement/rehabilitation project. Until the roadway approach is widened, it may be necessary to indicate with pavement markings that the additional width on the bridge cannot be used by through traffic.

c. Length. The length of the freeway bridge determines the width of the underpass for the facility passing beneath the freeway. Therefore, if practical, the freeway bridge should be long enough to accommodate any likely future widening of the underpassing roadway, based on at least a 20-year projection. This may involve an assessment of the potential for further development in the general vicinity of the underpass. The ODOT Planning Division should be consulted for design year traffic projections.

d. Bridge Rails. All existing bridge rails on the project should be evaluated to determine their adequacy. This will be the responsibility of the ODOT Bridge Division.

e. Approach Barrier Transitions. The approaching barrier transitions will be evaluated to determine if they meet ODOT's current criteria. If the transitions do not, they will be upgraded.

13.3.4.6 Roadside Safety

All 3R freeway projects will be evaluated for potential roadside safety improvements within the project limits. Except as noted in the next paragraph, the criteria in Chapter Eleven will fully apply to the evaluation. This includes roadside clear zones, barrier warrants, barrier design and drainage features.

Barrier warrants and design can present difficult problems (e.g., barrier height). The discussion in Section 13.2.11 on 3R non-freeway projects also applies to safety appurtenances on 3R freeway projects. Note that clear zones are the same as for new construction/reconstruction. See Section 11.2.

13.3.4.7 Interchanges

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. This may include lengthening acceleration/deceleration lanes, clearing the gore area, correcting the ramp superelevation, etc. In general, Chapter Ten will be used to design all interchange elements. The designer will determine, on a case-by-case basis, the proper application of the Chapter Ten criteria to a 3R freeway project.

One specific interchange design detail warrants discussion. Section 10.5.3 states that newly constructed slip ramps to a two-way frontage road are unacceptable. Existing slip ramps to two-way frontage roads may be within the limits of a proposed 3R freeway project. If practical, these should be removed and replaced by, for example, a buttonhook ramp design (see Section 10.5.3). If the designer decides to retain existing slip ramps to two-way frontage roads, the designer must prepare a written justification and attain written approval from the Division head.

13.4 SPOT IMPROVEMENTS

13.4.1 Objectives

Spot improvements are intended to correct an identified deficiency at an isolated location. This project scope of work is consistent with ODOT's responsibility to provide a safe driving environment for the motoring public which is free of unexpected demands on the driver. Research has demonstrated the need to improve specific roadway sections or spot locations with recognized 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,3R(non-freeway) or 3R (freeway). 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. One example is the 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 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 13.4.3.

13.4.2 Approach

ODOT 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 Chapter Twelve will provide the frame of reference for the spot improvement. Chapter Nine will apply to an intersection project. If a 3R project is considered the most likely level of improvement, then the criteria in Section 13.2 or 13.3 apply.

2. Design Speed. The design speed of the adjacent sections should be used for the spot improvement; however, a design speed less than the posted speed should generally not be used, unless a decision has been made to provide a signed reduction in the speed limit, or a design exception is approved. The selection of the applicable design speed will be left to the judgment of the designer.

   In summary, the selection of a project design speed will be one of the following:

   a. The existing posted or regulatory speed may be selected.

   b. The 85th percentile speed based on a speed study may be selected. This may be especially applicable to intersection modification studies (e.g., intersection sight distance, length of auxiliary turn lanes).

c. The designer may select a design speed which is higher than the anticipated posted or regulatory speed limit, where deemed to be appropriate.

d. On projects of less than one-half mile length on facilities with unposted speed limits (typically intersections or bridge replacement projects), the selection of design speed should be made considering the operating speed, the overall characteristics of the corridor and the regulatory limit. If the selected design speed is less than the unposted regulatory speed limit, the project must provide for either a posted speed limit consistent with the design speed, or warning signs in combination with appropriate advisory speed signing. Such projects will not require a design exception.

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., lengthening a culvert, flattening a curve, adding a left-turn lane). In addition, the designer should document and evaluate other geometric design deficiencies within the project limits, even if not related to the specific objective of the spot improvement.

4. Exceptions. For spot improvements, the exception process will apply only to those elements which are improved by the project. See Section 5.8 for a discussion on the design exception process.

13.4.3 Geometric Design of Highway Bridge Replacement/Rehabilitation Program (HBRRP) Projects

The spot improvement approach discussed in Section 13.4.2 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 ODOT's current criteria. For bridge replacement projects, the designer should evaluate the practicality of realigning the bridge to meet the applicable alignment criteria (reconstruction or 3R). For bridge rehabilitation projects, it is unlikely to be cost effective to realign the bridge to correct any alignment deficiencies.

2. Width. The bridge width should equal or exceed the full approach width, including shoulders, as determined from ODOT'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 Chapter Twelve, Section 13.2 or Section 13.3. If the decision is made not to provide the applicable width, the designer must comply with the exception process (Section 5.8).

3. 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.

4. Bridge Rails. All existing bridge rails on the project should be evaluated to determine their adequacy. This will be the responsibility of the ODOT Bridge Division.

5. Approach Barrier Transitions. The approaching barrier transitions will be evaluated to determine if they meet ODOT's current criteria. If the transitions do not, they will generally be upgraded.

13.5 REFERENCES

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

2. *Designing Safer Roads; Practices for Resurfacing, Restoration and Rehabilitation*, Special Report 214, Transportation Research Board, 1987.

3. "Developing Geometric Design Criteria and Processes for Non-Freeway RRR Projects," Technical Advisory T5040.28, Federal Highway Administration, October 17, 1988.

4. *Manual on Uniform Traffic Control Devices*, Federal Highway Administration, 1988.

5. *Highway Capacity Manual*, TRB Special Report 209, Transportation Research Board, 1985.

6. *Safety at Narrow Bridge Sites*, NCHRP 203, Transportation Research Board, 1979.

7. *Roadside Design Guide*, AASHTO, 1988.

8. *A Policy on Design Standards -- Interstate System*, AASHTO, 1991.

9. *Policy on Driveway Regulations for Oklahoma Highways*, Oklahoma Department of Transportation, revised January 1987.

10. *Utilities Manual*, Oklahoma Department of Transportation, Right-of-Way Division, current edition.

Chapter Fourteen

Traffic Engineering

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Chapter Fourteen

TRAFFIC ENGINEERING

The field of traffic engineering covers a wide range of design applications concerning the actual operation of vehicles on the highway system. With the variety of methodologies available yielding different solutions, there can be confusion among designers and consultants in determining ODOT's preferred methodology for these design applications. Therefore, Chapter Fourteen provides information on ODOT's traffic engineering operations. This includes the Traffic Engineering Division's organization, highway signing, pavement markings, traffic signals, highway lighting, traffic data field studies, accident surveillance systems, maintenance and protection of traffic through construction zones, and the highway safety improvement program.

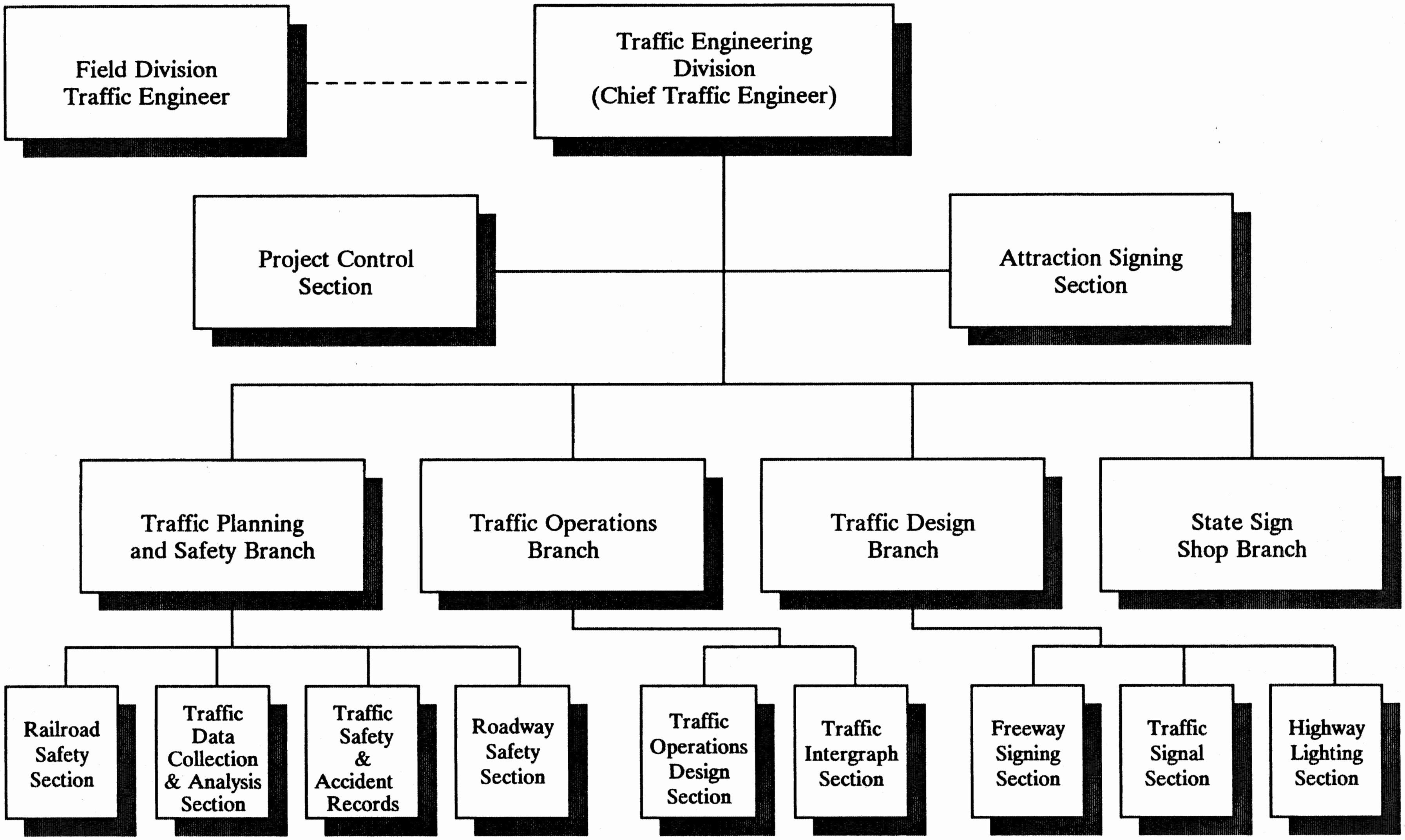
14.1 TRAFFIC ENGINEERING DIVISION

The Traffic Engineering Division is under the Assistant Director – Design. For additional information on ODOT's organization, see Chapter One. Figure 14.1A presents the organizational structure of the Traffic Engineering Division. The following sections discuss the specific functional responsibilities of the Chief Traffic Engineer, each branch within the Traffic Engineering Division and the Field Division Traffic Engineer.

14.1.1 Chief Traffic Engineer

The Chief Traffic Engineer, and his staff, is responsible for overseeing and implementing the administrative duties of the Traffic Engineering Division. To accomplish these tasks, the Chief Traffic Engineer's duties are to:

1. advise the Assistant Director – Design on traffic engineering issues and the Traffic Engineering Division's activities;
2. ensure that there is adequate planning and resources available for all projects assigned to the Division;
3. maintain adequate managerial controls to ensure that projects are processed according to plans and schedules;
4. develop and present traffic items to the Oklahoma Transportation Commission for approval;
5. develop standards for traffic control devices, highway lighting, pavement markings, signs, traffic signals and driveway controls;
6. review and process driveway permits requiring the Central Office's approval;
7. provide services to municipalities and other agencies relative to traffic design issues;
8. recommend and/or propose legislation concerning traffic laws; and
9. maintain ODOT's Attraction Signing Program.



TRAFFIC ENGINEERING DIVISION
(Organization)

Figure 14.1A

14.1.2 Traffic Planning and Safety Branch

The Traffic Planning and Safety Branch is responsible for developing and implementing the Division's policies, procedures and programs relating to traffic planning and safety. To accomplish these tasks, the Traffic Planning and Safety Branch's duties are to:

1. maintain data records on traffic statistics, traffic control devices and traffic movements in sufficient detail to meet the planning, research and operational needs of ODOT;

2. review traffic studies prepared by other organizations in connection with traffic control movements, regulations and safety;

3. coordinate with the Department of Public Safety to determine appropriate coding procedures for all traffic accidents that occur on the State Highway System and within municipalities throughout Oklahoma;

4. maintain an inventory of traffic accident data in sufficient detail to meet the planning, research and operational needs of ODOT;

5. analyze accident information and prepare summary reports reflecting significant findings which pertain to traffic accidents and related data;

6. perform continuing surveillance of the State Highway System in an effort to identify those locations where accident incidence rates are significantly higher than expected;

7. periodically publish an inventory of high-accident locations;

8. compile state-wide traffic accident data in specific formats so the data can be merged with other States' findings;

9. conduct pavement surface friction testing for ODOT's Pavement Management System and wet weather accident analysis;

10. conduct thorough evaluations of traffic control devices and recommend remedial signing improvements as needed;

11. assist in the preparation and review of urban and rural planning studies related to traffic control devices;

12. coordinate and perform research projects concerned with measuring the effectiveness of various traffic control devices;

13. prepare specifications for materials and equipment related to traffic control devices;

14. promote and encourage municipal governments throughout Oklahoma to actively upgrade traffic control devices within their respective jurisdictions;

15. distribute the FHWA *Manual on Uniform Traffic Control Devices* to counties and selected cities;

16. maintain ODOT's Safety Management System, incorporating traffic safety design considerations from all areas of highway development (planning, design, construction and maintenance);

17. perform roadway safety engineering analyses in order to recommend economically feasible safety improvements;

18. advise other State agencies, as necessary, in areas of traffic safety (e.g., Governor,

Highway Safety Office, Department of Public Safety, Department of Education, Attorney General);

19. provide accident data and traffic safety analyses for the Office of the General Counsel as required by Risk Management;

20. develop procedures to administer Federal-aid funds relating to traffic safety;

21. maintain the Railroad Crossing Safety Program for the approximately 4,800 public at-grade crossings within the State;

22. conduct spot speed studies to establish speed zones;

23. collect and assemble data to monitor highway traffic speed trends for ODOT and FHWA speed monitoring requirements; and

24. produce and maintain ODOT's video log system.

14.1.3 Traffic Operations Branch

The Traffic Operations Branch is responsible for coordinating the design process between the Field Divisions and the Central Office. To accomplish these tasks, the Traffic Operations Branch's duties are to:

1. provide guidance, coordination, liaison and technical assistance to the Field Divisions and other government agencies within the Field Divisions;

2. provide feedback of information from the Field Divisions to the other Branches within the Traffic Engineering Division and other ODOT design units;

3. act as a liaison between the Field Divisions, Planning Division, Transportation Commission and municipalities to determine speed zones and limits;

4. act as a liaison between the Field Divisions, Urban and Rural Design Divisions and Deputy Director on driveway applications and agreements;

5. review roadway design plans for proper use of traffic control devices, including maintenance and protection of traffic through construction zone plans;

6. review project planning reports and other special reports prepared within the Central Office;

7. prepare specifications for materials, equipment and installation procedures relative to traffic control devices;

8. maintain surveillance of the State Highway System for areas of congestion and/or high-accident locations;

9. coordinate the Traffic Division's Intergraph usage;

10. train and guide the traffic designers on the use of Intergraph;

11. prepare plans and drawings for the Traffic Operations Branch;

12. determine the Division's computer hardware requirements; and

13. review new software for possible applications by the Division and ODOT.

14.1.4 Traffic Design Branch

The Traffic Design Branch is responsible for developing and implementing the Division's policies, procedures and programs pertaining to signing, pavement markings, traffic signals, lighting and special projects. To accomplish these tasks, the Traffic Design Branch's duties are to:

1. review and/or prepare specifications for materials, equipment and installation of highway signs;

2. develop plans and standards for signing and pavement marking projects;

3. review signing plans prepared by others;

4. review, when requested, design plans for adequacy relative to the placement of highway signing;

5. provide guidance and assistance to municipalities on issues relative to highway signing;

6. act as a liaison between the Field Divisions, Traffic Signal Section, Transportation Commission and municipalities relative to signal warrants, designs, agreements and installations;

7. review and/or prepare specifications for materials, equipment and installation of traffic signals;

8. develop plans and standards for traffic signal projects;

9. review traffic signal plans prepared by others;

10. review, when requested, design plans for adequacy relative to placement of traffic signals;

11. provide guidance and assistance to municipalities on issues relative to traffic signal designs;

12. prepare traffic control device studies in urban areas;

13. review traffic control device and safety studies prepared by the Traffic Planning and Safety Branch;

14. review and/or prepare specifications for materials, equipment and installation of highway lighting;

15. develop plans and standards for highway lighting;

16. review highway lighting plans prepared by others;

17. review, when requested, design plans and utility permits for adequacy relative to the placement of highway lighting; and

18. provide guidance and assistance to municipalities on issues relative to highway lighting designs.

14.1.5 State Sign Shop Branch

The responsibilities of the State Sign Shop Branch are to prepare and manufacture all signs that are to be used along Oklahoma's State highways. In addition, they may also be requested to prepare and manufacture signs for other state agencies.

4.1.6 Field Division Traffic Engineer

The Field Division Traffic Engineer is responsible to the Transportation Division Engineer for all traffic operations functions in the Field Division. He is further accountable to the Chief Traffic Engineer with respect to traffic engineering policies. The implementation of policies of traffic engineering and traffic safety and design functions established by the Chief Traffic

Engineer is administered through the Transportation Division Engineer and, subsequently, through the Field Division Traffic Engineer. The Field Division Traffic Engineer's responsibilities are to:

1. handle complaints related to traffic engineering and traffic operations within the Field Division;

2. conduct studies and investigations for traffic signals and other traffic control devices;

3. conduct special studies such as parking studies, ramp studies, pedestrian studies, school crossing studies, as needed;

4. supervise the preparation of sign plans for new projects within the Field Division on the State Highway System for signs to be installed by State forces;

5. participate in field plan-in-hand reviews prior to completion of construction plans for determining proper functional capacity and maximum safety of geometric design;

6. comment on plans for signing, highway lighting, signals and other traffic control devices to be installed by the contractor;

7. participate in preconstruction conferences and final inspections providing input to ensure that proper traffic control devices, geometric design and handling of traffic is adequate during and after construction;

8. assist in establishing detours and necessary signing in cooperation with the Resident Engineer;

9. make field inspections of all new projects on the State Highway System during and after construction for functional operation and safety, including inspection of construction signing and handling of traffic;

10. inspect periodically all traffic control devices on State Highway System maintained by ODOT, including:

    a. Paint Lines — At least twice a year, in the spring and fall, review all painted lines on the System. Schedule for restriping all lines needing repainting as soon as practicable.

    b. Signs — Inspect all signs at least once each year including inspecting for night reflectivity. Schedule all deficient signs for replacement, posts to be straightened, and other maintenance work as required.

    c. Traffic Signals and Flashers — Inspect at least once each year for proper operation, including timing. Schedule required maintenance work such as painting of heads, poles and arms; periodic reflectors; focusing of heads; and other maintenance work as needed.

    d. Highway Lighting — Inspect at least once each year for proper operation, and schedule necessary maintenance work.

    e. Other Traffic Control Devices (such as islands, pavement buttons, barrels, barricades, etc.) — Check once each year for painting or replacement, as needed.

11. prepare special pavement marking designs in connection with channelization or other traffic control measures;

12. conduct speed studies to establish or adjust speed zones;

13. review permit applications for all driveway entrances;

14. provide technical supervision over the Division Sign Crews through the Traffic Superintendent;

15. conduct studies of accident prone locations;

16. review all fatal traffic collision reports and take such remedial action as may be required;

17. establish and maintain a liaison with municipal officials responsible for traffic control, and act in an advisory capacity where such service is required by the municipality; and

18. meet with governmental organizations, civic groups and other interested parties concerning traffic engineering and related matters.

14.2 MUTCD CONTEXT

Throughout the MUTCD the words "shall," "should" and "may" are used to describe the appropriate application for various traffic control devices. The MUTCD defines these terms as follows:

1. Shall -- a *mandatory* condition. Where certain requirements in the design or application of the device are described with the "shall" stipulation, it is mandatory when an installation is made that these requirements be met.

2. Should -- an *advisory* condition. Where the word "should" is used, it is considered to be advisable usage, recommended but not mandatory.

3. May -- a *permissive* condition. No requirement for design or application is intended.

In reference to the MUTCD definitions, ODOT has taken the following positions:

4. <u>Shall</u>. The designer will make every practical effort to follow the MUTCD criteria. For those few situations where it may be impractical to follow the "shall" criteria, the designer needs to obtain a Level One or Two design exception, as applicable.

5. <u>Should</u>. The designer is to follow the MUTCD with very few exceptions. In some situations, ODOT has determined that the MUTCD's "should" is to be changed to a "shall" to reflect a mandatory condition in Oklahoma. These situations are discussed throughout Chapter Fourteen.

   For those situations where it may be impractical to follow the "should" criteria, the designer needs to obtain a Level Two design exception.

6. <u>May</u>. The designer should make every reasonable effort to follow the MUTCD criteria. For those situations where it is impractical to follow the "may" criteria, no design exception will be necessary.

14.3 HIGHWAY SIGNING

14.3.1 General

The majority of the information required for the design and placement of highway signs along Oklahoma highways can be found in the FHWA *Manual on Uniform Traffic Control Devices* (MUTCD) and the ODOT Standard Drawings. The intent of this section is not to reiterate the information presented in these sources, but instead to supplement these references where necessary and to provide the user with additional guidance.

14.3.1.1 References

For additional information on highway signing, the designer is referred to the following publications:

1. *Manual on Uniform Traffic Control Devices*, FHWA;
2. *Traffic Control Devices Handbook*, FHWA;
3. *Standard Highway Signs*, FHWA;
4. *Standard Alphabets for Highway Signs and Pavement Markings*, FHWA;
5. *Traffic Engineering Handbook*, ITE;
6. *A Policy on Geometric Design of Highways and Streets*, AASHTO;
7. *Roadside Design Guide*, AASHTO; and
8. *Manual of Steel Construction*, AISC.

14.3.1.2 Computer Software

There are many computer software programs available to the designer that may be used in the design of highway signing including sign layouts, legends, quantities, structure supports, etc. The designer needs to be aware that not all software packages are applicable to Oklahoma. Therefore, the user should first contact the Traffic Engineering Division to determine which programs and versions are acceptable for use on ODOT projects.

Most of these software programs can be purchased from either McTrans Center, 512 Weil Hall, Gainesville, Florida 32611-2083; or from PC-TRANS, Kansas University Transportation Center, 2011 Learned Hall, Lawrence, Kansas 66045.

14.3.1.3 MUTCD Context

Throughout the MUTCD, the words "shall," "should" and "may" are used to describe the appropriate application for various traffic control devices. Section 14.2 presents ODOT's position on these clarifying words.

14.3.2 Applications

14.3.2.1 Sign Reflectorization

It is ODOT's policy that all signs need to be either illuminated or reflectorized. The following describes the reflective sheeting types available:

1. Type II. Enclosed lens type consisting of spherical lens elements embedded beneath the surface of a smooth, transparent, flexible plastic resulting in a nonexposed lens, optical reflecting system.
2. Type II-A. Same as Type II but with a higher quality and number of spherical lens elements.

3. Type III-A. Encapsulated lens type consisting of spherical lens elements adhered to a synthetic resin and encapsulated by a flexible, transparent waterproof plastic having a smooth surface.

4. Type IV. Cube corner (mircoprism) retroreflective elements integrally bonded to a transparent, flexible, smooth-surfaced, tough and weather-resistant polymeric film.

Although Type III-A reflective sheeting is more expensive than Type II-A, it provides much better reflectivity. For permanent sign installations, only the Type III-A sheeting should be used. The Type IV sheeting is generally only used on construction traffic control devices (e.g., plastic cones, barrels) or on other devices where flexible sheeting is required.

14.3.2.2 Overhead Signs

It is ODOT's policy that along Interstates and fully controlled urban expressways all guide signs will be placed on either overhead sign structures or overpasses. For other types of highways, the decision on whether to use overhead signing will be determined on a case-by-case basis. For a list of factors to consider in determining the need for an overhead sign, the designer should review the factors listed in the MUTCD.

Typically, all overhead signs are lighted, if an agreement can be reached with the local government for the sign lighting maintenance and if a power source is reasonably close to the overhead sign.

All existing overhead sign structures should have walkways on the front. New overhead sign structures should have walkways on the front and back.

14.3.2.3 Stop/Yield Signs

All local roads/streets intersecting a State-maintained highway with an at-grade intersection and which are not signalized will be stop controlled. A yield sign may be used if the intersection is operating in a merge condition (e.g., channelized intersection with a turning roadway) or at an entrance ramp to an access-controlled facility.

14.3.2.4 Speed Limit Signs

In accordance with Title 47 of the Oklahoma Statutes, the ODOT Transportation Commission is required to set all speed zones and limits under public jurisdiction. The Traffic Engineering Division is responsible for investigating a given site and recommending to the Transportation Commission an appropriate speed zone and limit. All requests for speed studies by local officials must first go through the Field Division Offices. The Field Division Office will then coordinate all such requests with the Traffic Engineering Division.

Typically, it is ODOT's practice to establish the speed limit at the 85th percentile speed as determined through speed studies. In addition, the set speed limit should not exceed the 85th percentile speed or the regulatory speed limit. If deemed necessary, a lower speed limit may be recommended to the Commission for school zones, high-accident locations, heavy congestion areas, etc.

14.3.2.5 No-Turn-On-Red Signs

Unless otherwise signed, right-turn-on-red is allowed at all intersections within the State. Where two one-way streets intersect, left-turn-on-red is allowed unless otherwise signed. The decision on whether to eliminate these maneuvers at an intersection should be based on an engineering study as defined in the MUTCD and must be coordinated with local and FHWA officials. Once the decision has been made to eliminate the turning movement, the proper no-turn-on-red sign shall be used as specified in the MUTCD.

14.3.2.6 Placement of Advance Warning Signs

Table 14.3A provides the recommended minimum distances for placement of advance warning signs. These distances are based on three conditions which are defined by the MUTCD as follows:

1. Condition A -- a higher driver judgement condition which requires the driver to use extra time in making and executing a decision because of a complex driving situation;
2. Condition B -- a condition in which the driver will likely be required to stop; and
3. Condition C -- a condition in which the driver will likely be required to decelerate to a specific speed.

If these distances cannot be met, then an oversized sign and/or flashing beacons may be used to adequately warn the driver. The need for an oversized sign and/or flashing beacon will be determined on a case-by-case basis. Table 14.3B provides a breakdown for the above-listed conditions of the typical warning signs used in Oklahoma. The following example illustrates how to use the two tables.

Example

Given: Stop-controlled intersection
Posted Speed 45 mph

Problem: Where to place a Stop Ahead Sign

Solution: From Table 14.3B it is determined that the stop ahead sign is a Condition B category (i.e., the driver must stop). From Table 14.3A the set-back distance is shown to be 300 ft.

If the sign cannot be adequately placed at a location of approximately 300 ft, then a 36" x 36" sign or flashing beacons may be appropriate.

14.3.2.7 Turn/Curve Signs

The MUTCD shows several alignment signs, but does not fully define when to use these signs. The decision on when to use an advance turn or curve warning sign is dependent upon many factors including speed, alignment, accident history, etc. It would be impractical and uneconomical to place an advance warning sign at every horizontal curve. Before using an advance turn or curve warning sign, the designer should consider the following:

1. Speed Determinations. In determining whether or not to place an alignment warning sign and/or advisory speed plates, the designer first needs to determine the appropriate safe speed of the curve. If the curve radius and superelevation are known (e.g., from construction plans) then Figure 14.3A can be used to determine the appropriate safe speed of the curve. If the radius of the curve is unknown, then a speed study is usually warranted. These types of studies are typically done using a ball-bank indicator.

   The ball-bank indicator test involves driving a test vehicle around a curve at

Table 14.3A

SUGGESTED DISTANCES FOR ADVANCE WARNING SIGN PLACEMENT (FT)[1]

Posted or 85 percentile speed (MPH)	Condition A High Judgment needed[2] (10 secs. PIEV)	General warning signs[3]					
		Condition B Stop Condition	Condition C — Deceleration condition to listed advisory speed — MPH (or desired speed at condition)				
		0	10	20	30	40	50
20	175	([4])	([4])	—	—	—	—
25	250	([4])	100	—	—	—	—
30	325	100	150	100	—	—	—
35	400	150	200	175	—	—	—
40	475	225	275	250	175	—	—
45	550	300	350	300	250	—	—
50	625	375	425	400	325	225	—
55	700	450	500	475	400	300	—
60	775	550	575	550	500	400	300
65	850	650	650	625	575	500	375

Source (1)

Notes:

1. Distances shown are for level roadways. Distance based on 36-inch signs. If 48-inch signs are used, the legibility distance may be increased to 200 feet. This would allow reducing the above distance by 75 feet.

2. In urban areas, a supplementary plate underneath the warning sign should be used specifying the distance to the condition if there is an in-between intersection which might confuse the motorist.

3. Distance provides for 3-second PIEV, 125 feet Sign Legibility Distance, Braking Distance for Condition B and Comfortable Braking Distance for condition C as indicated in *A Policy on Geometric Design of Highways and Streets*.

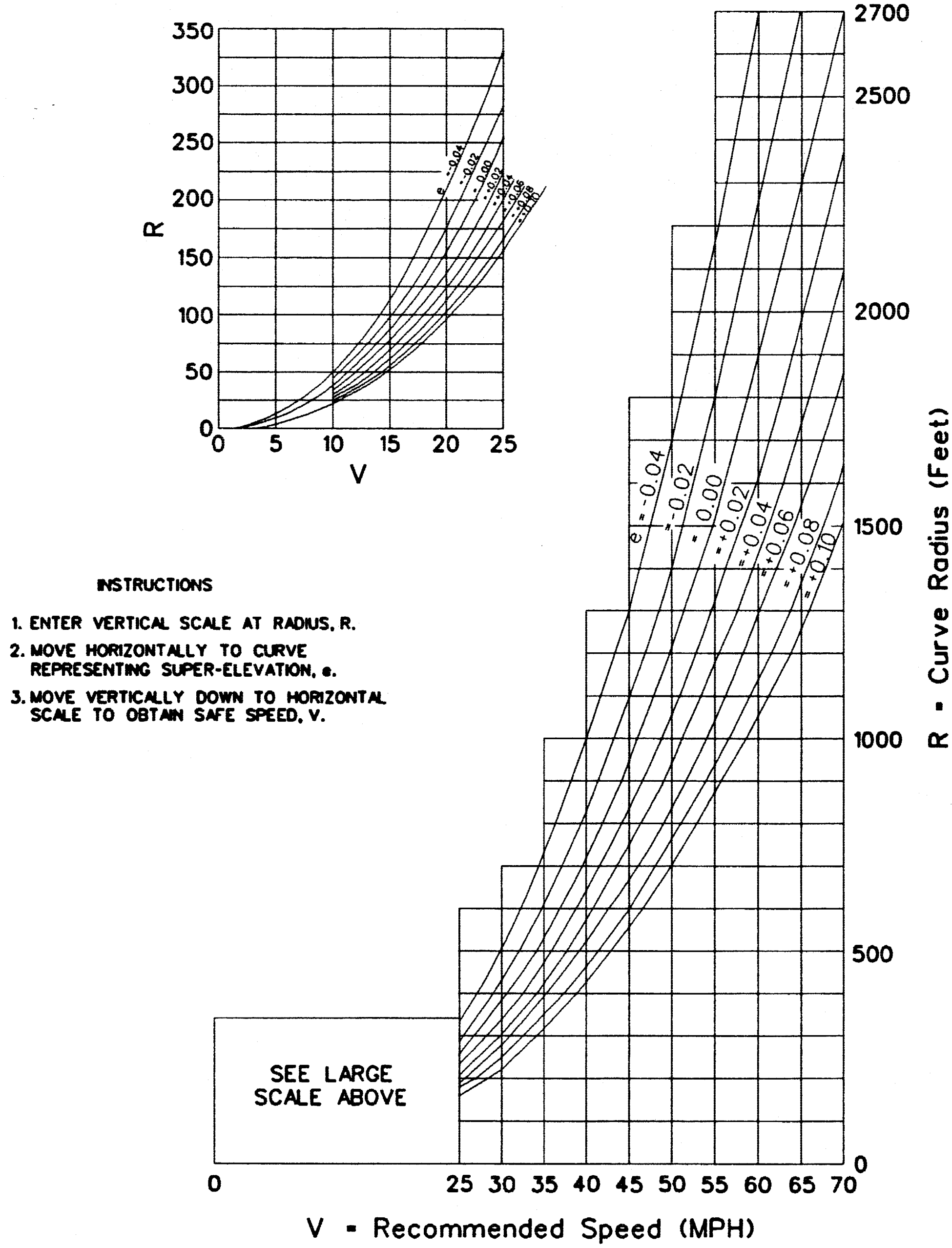
4. No suggested minimum distance provided. At these speeds, sign location depends on physical conditions at site.

Table 14.3B

CONDITIONS FOR ADVANCE WARNING SIGN PLACEMENT

CONDITION A	CONDITION B	CONDITION C
W4-1(L)	W1-7	W1-1 (L)
W4-1(R)	W2-1	W1-1 (R)
W4-2(L)	W2-2	W1-2 (L)
W4-2(R)	W2-3(L)	W1-2 (R)
W9-1(L)	W2-3(R)	W1-3 (L)
W9-1(R)	W2-4	W1-3 (R)
W9-2(L)	W2-5	W1-4 (L)
W9-2(R)	W3-1a	W1-4 (R)
W12-1	W3-2a	W1-5 (L)
W12-2	W3-3	W1-5 (R)
	W8-6	W1-6
	W8-6A	W1-8
	W8-6S	W6-1
	W10-1	W6-1 (Inv.)
	W11-2	W6-3
		W8-5
		W13-1
		W13-2
		W13-2a
		W13-3
		W13-3a
		W14-3

Note: This list only applies to those warning signs shown in the *ODOT Standard Drawings*. See Section 14.3.2.6 for determining conditions for other warning signs.



Source (2)

RECOMMENDED SAFE SPEED

Figure 14.3A

various speeds and reading a curved level to determine an appropriate safe speed for the curve. Table 14.3C presents the various recommended safe speeds for a curve based on several ball-bank readings. Test runs should be conducted in both directions with the selected speed applying to the lowest reading.

2. Highway Alignment. The designer should review the overall highway alignment to determine if warning signs are warranted. Unexpected curves after long tangent sections may be candidates for placement of an advance warning sign. Conversely, sharp curves on winding highways may not warrant the use of an advance warning sign as the driver will be expecting the turn.

3. Posted Speeds. Relative to the posted speed, the designer should consider the following:

   a. Highways with a posted or regulatory speed of less than 30 mph generally will not warrant an advance warning sign.

   b. A turn or curve sign should be considered when the recommended safe speed of the curve is found to be more than 10 mph below the posted speed.

4. Accident History. The accident history should be reviewed to determine if there are a high number of run-off-the-road accidents that can be contributed to the horizontal curve. High-accident locations will most likely warrant advance warning signs, advisory speeds plates and/or chevrons. On curves with few or no accidents, the use of advance warning signs typically will not be warranted.

5. Driver Familiarity. Highways serving local needs (e.g., collectors, local roads) will rarely warrant advance warning signs as the typical driver will be aware of the restrictive alignment. However, on arterials and recreational roads the typical driver may be less familiar with the highway and may require additional warnings.

6. Area Classifications. Urban areas will typically not warrant the use of advance warning signs as speeds tend to be lower and there is greater driver familiarity and awareness.

7. Public Reaction. Local residents generally have a good feel on how the drivers are reacting to the horizontal situation. If there are no complaints relative to near misses or accidents, the curve will probably not warrant the need for signing. Frequent complaints usually warrant further investigation.

8. Turn Versus Curve Sign. If it is determined that an advance alignment warning sign is warranted, the MUTCD recommends that a turn sign be used if the curve's recommended safe speed is 30 mph or less and that a curve sign be used if the curve's recommended safe speed is greater than 30 mph.

9. Advisory Speed Plates. If a turn sign is used, an advisory speed plate should also be used listing the recommended safe speed. For curve signs, an advisory plate should be used when the safe speed of the curve is more than 15 mph below the posted speed.

Table 14.3C

BALL-BANK INDICATOR READINGS

VEHICLE SPEED	BALL-BANK READINGS	RECOMMENDED SAFE SPEED OF CURVE
20 mph or less	14° or greater	Speed at which the 14° reading occurs
20 - 30 mph	12°	Speed at which the 12° reading occurs
35 mph or greater	10°	Speed at which the 10° reading occurs

Source (2)

14.3.2.8 Chevron Signs

Chevron signs should be used where there is a history of run-off-the-road accidents in conjunction with a horizontal curve. The minimum chevron sign size along State highways is 18" x 24"; however, to improve visibility the use of the larger 30" x 36" chevron sign is recommended.

14.3.2.9 Low Clearance Signs

In accordance with Title 47, Section 14-114 of the Oklahoma Statutes, ODOT is required to post the vertical clearances of all bridges on the State Highway System. New clearance signs are required if, after resurfacing operations, the vertical clearances have changed. In addition, it is ODOT's policy to install advance low clearance signs where the bridge clearance is less than 13′-6″. Chapters Twelve and Thirteen present the minimum clearances for bridges and sign structures over highways.

14.3.2.10 Logo Signing Policy

Information on ODOT's logo signing policy can be found in the ODOT publication, *Logo Signing*, or by contacting the Traffic Engineering Division.

14.3.2.11 School Zones

Part VII of the MUTCD and Section 11-801 of Title 47 of the Oklahoma Statutes provide explicit criteria relative to the determination of school zones and the placement of crossing signs near schools. In addition to the above-listed sources, the following should be considered:

1. Maximum Speed Limit. The maximum speed limit through a properly marked school zone shall be 25 mph.

2. School Zone. The areas for enforcement of the school zone speed limit is restricted to 100 yards on each side of a school adjacent to the highway. Restricted school zone speed limit signs

cannot be placed on highways which are not adjacent to the school.

3. Moveable Signs. The use of moveable speed restricted signs should meet the following conditions:

   a. they cannot be placed in the roadway more than 1 hour before assembly,

   b. they are to be removed from the roadway when school is in session,

   c. they are to be removed within 1 hour after school is dismissed,

   d. they are to be placed so that they do not impede normal traffic flow, and

   e. they are to be made of materials which are not hazardous to motorists or pedestrians when struck.

4. Flashing Beacons. Instead of moveable signs, a school speed limit sign with a flashing beacon may be used to restrict speeds through the school zone if the following conditions are met:

   a. the difference between the normal posted speed and the school zone posted speed is 25 mph or more, or the volume of the highway exceeds 500 vehicles per hour during both the school assembly and dismissal times;

   b. the school speed limit signs are permanently placed outside of the roadway; and

   c. the flashers are only on for the period one hour before school assembly and one hour after school is dismissed.

Normal time periods that a flashing beacon may be in operation for a typical school day are as follows:

1. 7:30 a.m. - 8:30 a.m.,
2. 11:30 a.m. - 1:30 p.m., and/or
3. 2:30 p.m. - 4:30 p.m.

14.3.2.12 Tourist Oriented Destination Signing

In accordance with Title 74, Section 18-91 of the Oklahoma Statutes, the Oklahoma Tourism Signage Advisory Task Force was established to review and issue recommendations to ODOT relative to all applications concerning directional signs for tourist and traveler-related attraction and enterprises. The Task Force membership is composed of the following ten members:

1. a person appointed by the Governor;

2. the Chairman of the Senate Tourism and Recreation Committee or designee;

3. the Chairman of the House of Representatives Tourism and Recreation Committee or designee;

4. the Director of the State Arts Council or designee;

5. a representative from each of the following organizations or State agencies to be selected by that organization or agency;

   a. Department of Transportation,

   b. Department of Tourism and Recreation,

   c. Oklahoma Historical Society,

   d. Oklahoma Museums Association,

   e. Oklahoma Lakes and Countries Association, and

f. Oklahoma Travel Industry Association.

In addition, Title 74, Section 18-92 of the Oklahoma Statutes allows ODOT, acting upon the recommendation of the Task Force, to design, construct, erect and select the location of the guide sign. Each person, firm, organization or entity approved for a guide sign shall be responsible for furnishing all materials for said signs in accordance with ODOT's "Attraction Signing Policy." ODOT will retain all jurisdiction over these signs. However, any maintenance and replacement, for whatever reason, shall be the sole responsibility of the entity sponsoring the sign. Additional information on the design and placement of these signs can be found in ODOT's "Attraction Signing Policy" or by contacting the Traffic Engineering Division.

14.3.2.13 Roadside Safety

In general, on State Highways, all sign posts within the clear zone should have either break-away supports or be protected. Chapter Eleven provides additional details on ODOT's roadside safety criteria.

14.4 PAVEMENT MARKINGS

14.4.1 General

The majority of the information required for the design and placement of pavement markings along State-maintained highways can be found in the *Manual on Uniform Traffic Control Devices* (MUTCD) and the *ODOT Standard Drawings*. The intent of this section is not to reiterate the information presented in these sources, but instead to supplement these references where necessary and to provide the user with additional guidance.

14.4.1.1 References

For additional information on pavement markings, the designer is referred to the following publications:

1. *Manual on Uniform Traffic Control Devices*, FHWA;
2. *Traffic Control Devices Handbook*, FHWA;
3. *Traffic Engineering Handbook*, ITE; and
4. NCHRP Synthesis 138, *Pavement Markings: Materials and Application for Extended Service Life*.

14.4.1.2 MUTCD Context

Throughout the MUTCD the words "shall", "should" and "may" are used to describe the appropriate application for various traffic control devices. Section 14.2 presents ODOT's position on these clarifying words.

14.4.2 Line Markings

Table 14.4A provides a listing of the typical pavement stripes used by ODOT.

14.4.3 Pavement Marking Materials

ODOT is presently using four types of pavement markings. Recommended locations for each pavement marking type are presented in Table 14.4B. All pavement marking materials must meet the criteria set forth in the Oklahoma *Standard Specifications for Highway Construction*. Each pavement marking type used by ODOT is described below:

1. Paint. Quick-drying paints are typically applied as a 4-inch or wider white or yellow stripe. Glass beads are dropped onto the wet paint which then bond to the paint surface when it dries. The use of glass beads greatly enhances the reflectivity of the paint stripe. Per unit cost, paint-applied markings are significantly cheaper than any other method. However, one of the major disadvantages of paint is that it can be quickly worn away on high-volume roadways and often needs to be reapplied more than once a year.

2. Thermoplastic. Thermoplastic markings are typically made from hydrocarbon or alkyd resins, pigment and filler. The materials are heated to high temperatures and are applied in thicknesses of 60 to 120 mils. After the material is applied to the surface, and while it is still hot, glass beads are dropped onto the mixture. When the material cools, the glass beads are then bonded to the surface. Thermoplastic markings must be applied to clean, dry pavements and on concrete pavements a primer is often required. Thermoplastic markers are significantly more expensive than paint, but often can last 5 or more years when applied properly. Thermoplastic is the preferred marking for high-volume roadways due to its long life.

3. Preformed Plastic. Preformed plastic markings are typically premade in a

Table 14.4A

TYPES OF PAVEMENT STRIPES

DESCRIPTION	COLOR	WIDTH	APPLICATION
Single Broken	White	4"	Separation of lanes on which travel is in the same direction, with crossing from one to the other permitted; i.e., lane lines on multilane roadways. The broken or dash line is formed by a pattern of segments and gaps. The typical pattern is a 12-foot segment followed by a 33-foot gap.
	Yellow	4"	Separation of lanes on which travel is in opposite directions, and where overtaking with care is permitted; i.e., centerline on 2-lane, 2-way roadways. The broken or dash line is formed by a pattern of segments and gaps. The typical pattern is a 12-foot segment followed by a 33-foot gap.
Single Solid	White	4"	Separation of lanes, or of a lane and shoulder, where lane changing is discouraged; i.e., lane lines at intersection approaches, right edge stripes.
		6"	Lane Lines separating a motor vehicle lane from a bike lane.
		8"	Delineation of locations where crossing is strongly discouraged; i.e., separation of special turn lanes from through lanes, gore areas at ramp terminals, paved turnouts.
	Yellow	4"	Delineation of left edge lines on divided highways, one-way roads and ramps.
Double Yellow	White	4-4-4"*	Separation of lanes on which travel is in same direction, with crossing from one side to the other prohibited; e.g., channelization in advance of obstructions which may be passed on either side.
	Yellow	4-4-4"*	Separation of lanes on which travel is in opposite directions, where overtaking is prohibited in both directions. Left-turn maneuvers across this marking are permitted. Also used in advance of obstructions which may be passed only on the right side.
Solid plus Broken	Yellow	4-4-4"*	Separation of lanes on which travel is in opposite directions, where overtaking is permitted with care for traffic adjacent to the broken line, but prohibited for traffic adjacent to solid line. Used on 2-way roadways with 2 or 3 lanes. Also used to delineate edges of a two-way left-turn lane — solid lines on the outside, broken lines on the inside.
Double Broken	Yellow	4-4-4"*	Delineates the edges of reversible lanes.
Single Dotted	Either	4"	Extension of lane lines through intersections. Color same as that of line being extended. Also used to extend right edge line of freeway shoulder lanes through off-ramp diverging areas in problem locations.
	White	8"	Separation of through lane and auxiliary lane or dropped lane.
Transverse	White	6" (min)	Crosswalk edge lines (minimum 6 ft apart) when not in the vicinity of school grounds.
		24"	Limit lines or STOP bars.
Diagonal	White	12"	Crosshatch markings, placed at an angle of 45°, at varying distances apart, on shoulders or channelization islands to add emphasis to these roadway features.

* 4-4-4" indicates typical width of stripes and gap between them. *Source (2)*

Table 14.4B

GUIDELINES FOR PAVEMENT MARKING PLACEMENT

ROADWAY CLASSIFICATIONS	PAVEMENT MARKINGS			
	PAINT	THERMOPLASTIC	PREFORMED PLASTIC	RAISED REFLECTIVE MARKER (ALL WEATHER)
Freeways		(2), (4)	(5)	(6), (7)
Urban Expressways		(2), (4)		(6), (7)
Rural Expressways		(2), (4)		(6), (7)
Urban Arterials	(1)	(3), (4)	(5)	(8)
Rural Arterials	(1)	(3), (4)		
Collectors (State Routes)	(1)	(3), (4)		

(1) Paint is typically used where traffic volumes are less than 5000 ADT.

(2) Typically used for these roadway types.

(3) Thermoplastic markings are typically used where traffic volumes are 5000 ADT or greater.

(4) If thermoplastic markings are used, all markings are to be thermoplastic (e.g., center lines, lane lines, edge lines, ramps.

(5) The Traffic Engineering Division is presently testing these markings at these locations.

(6) Raised reflective markings are used at all exit and entrance gores, see *ODOT Standard Drawings*.

(7) Raise reflective markings are used at the center and edge lines where traffic volumes are 15,000 ADT or greater.

(8) Raised reflective markings may be used at the centerline and lane lines on multilane and undivided highways.

factory from vinyl, pigment and fillers and can come in strips, words or symbols. Glass beads are commonly embedded into the surface of the markings at the factory. Application of the marking typically involves removing a protective strip, laying the marking in place and applying pressure with a roller or tire. Temporary tapes are commonly used in construction zones because the tapes can be easily removed. However, a common problem with some temporary preformed plastics is that they tend to break up easily and must be routinely checked for adequacy.

4. Raised Reflective Markers. Raised reflective markings are typically cube-cornered acrylic lenses, tempered-glass lenses, or glass-bead lenses, mounted in either a plastic or iron base. They are commonly placed with an adhesive to either the pavement surface or into a precut groove. Raised reflective markings are made to reflect the striping colors (e.g., white, yellow, red) and are often used in place of or as a supplement to other markings. To enhance the service life, recessed markers are designed to allow a snow plow to pass over the marker. The *ODOT Standard Drawings* provide additional details on the placement and color locations for raised reflective markers.

5. Experimental Markings. With the continued advancement of technology in pavement markings, there will always be new materials or methods in the placement of pavement markings. The designer is encouraged to pursue the use of these new materials or procedures. However, the use of any experimental pavement markings must be first approved by the Traffic Engineering Division.

14.4.4 Applications

14.4.4.1 Center Lines

It is ODOT's policy to place center lines on all State-maintained highways that have a surface width of 16 ft or more. For narrower widths, the decision on whether to use a center line will be determined on a case-by-case basis.

14.4.4.2 Edge Lines

It is ODOT's policy to place edge lines on all Interstates, multilane highways and paved roadways that have a surface width of 22 ft or more. For narrower roadway widths, the decision on whether to place edge lines will be determined on a case-by-case basis. The *ODOT Standard Drawings* also provide additional details on the placement of edge lines.

14.4.4.3 Parking Lot Markings

Section 17.1 provides information on the design and layout of parking stalls for both on and off street parking. White paint stripes of 4 inches are typically used for marking parking stalls.

14.4.4.4 Handicapped Markings

Section 17.4 provides information on the layout for handicapped parking stalls. The pavement markings will typically be white.

14.5 TRAFFIC SIGNALS

14.5.1 General

The design of traffic signals is one of the most dynamic fields of traffic engineering. Although this section will address several traffic signal design issues, it would be impractical for this section to constitute a traffic signal design guide. For detailed design information, the reader is encouraged to review the latest editions of the references listed in Section 14.5.1.1. The intent of this section is to provide the user with a brief overview of the traffic signal design issues and to present ODOT's positions, policies and procedures on these issues.

14.5.1.1 References

For additional information on traffic signal designs, the designer is referred to the following publications:

1. *Manual on Uniform Traffic Control Devices* (MUTCD), FHWA;
2. *Highway Capacity Manual*, TRB;
3. *Traffic Control Devices Handbook*, FHWA;
4. *Manual of Traffic Signal Design*, ITE;
5. *Traffic Engineering Handbook*, ITE;
6. *Traffic Control Systems*, NEMA;
7. National, State and Local Electrical Codes;
8. *Traffic Detector Handbook*, FHWA;
9. *Traffic Signal Installation and Maintenance Manual*, ITE;
10. *Traffic Signal Lamps*, ITE; and
11. Manufacturers' literature.

14.5.1.2 MUTCD Context

Throughout the MUTCD, the words "shall," "should" and "may" are used to describe the appropriate application for various traffic control devices. Section 14.2 presents ODOT's position on these clarifying words.

14.5.1.3 Definitions

The following is a brief list of the more common terms used in traffic signal designs:

1. Controller. (1) (Traffic) A device that controls the sequence and duration of indications displayed by traffic signals; (2) (computer) under computer supervision, a device that switches the signal circuits according to the computer's instructions.
2. Coordination. The establishment of a definite timing relationship between adjacent traffic signals.
3. Cycle Length. The time required for one complete sequence of signal indications.
4. Delay. (1) A measure of the time that elapsed between the stimulus and the response; (2) traffic delay — the time lost by vehicle(s) due to traffic friction or control devices.
5. Demand. The need for service; for example, the number of vehicles desiring to use a given segment of roadway during a specified unit of time.
6. Detection. The process used to identify the presence or passage of a vehicle at a specific point or to identify the presence of one or more vehicles in a specific area.

7. Detector. A device for indicating the presence or passage of vehicles or pedestrians (e.g., loop detector, microloop detector, calling detector, pushbutton, etc.).

8. Interval. A discrete portion of the signal cycle during which the signal indications remain unchanged.

9. Interval Sequence. Specifies the order in which the various intervals are displayed.

10. Interval Timing. The passage of time that occurs during an interval.

11. Loop Detector. A device capable of sensing a change in inductance of a loop sensor imbedded in the roadway caused by the passage or presence of a vehicle over the loop.

12. Offset. The time difference or interval in seconds between the start of the green indication at one intersection as related to the start of the green interval at another intersection or from a system time base.

13. Pattern. A unique set of traffic parameters (cycle, split and offset) associated with each signalized intersection within a predefined group of intersections.

14. Phase. A part of the traffic signal time cycle allocated to any combination of traffic movements receiving right-of-way simultaneously during one or more intervals.

15. Phase Overlap. Refers to a phase that operates concurrently with one or more other phases.

16. Phase Sequence. The order in which a controller cycles through all phases.

17. Point Detection. The detection of a vehicle as it passes a point or spot on a street or highway.

18. Preemption. The term used when the normal signal sequence at an intersection is interrupted and/or altered in deference to a special situation such as the passage of a train, bridge opening or the granting of the right-of-way to an emergency vehicle.

19. Presence Detection. The ability of a vehicle detector to sense that a vehicle, whether moving or stopped, has appeared in its field.

20. Recall. An operational mode for an actuated intersection controller whereby a phase, either vehicle or pedestrian, is displayed each cycle whether demand exists or not. Usually a temporary or emergency situation.

21. Split. A percentage of the cycle length allocated to each of the various phases in a signal sequence.

22. Yield. The action of allowing a semi-actuated controller, or an actuated controller operating in the semi-actuated mode, to terminate the main street phase so as to begin satisfying existing cross-street demand.

14.5.2 Preliminary Design Activities

In general, the Traffic Engineering Division is responsible for making the determination on the need for a new or existing traffic signal. This determination is based on several factors including traffic volumes, accident history, schools, pedestrians, local needs, driver needs, construction costs and maintenance costs. The following sections provide information on some of the guidelines, policies, procedures and factors that are used by ODOT in making these determinations.

14.5.2.1 Signal Study Requests

Requests for new signals can come from many sources — FHWA, Rural or Urban Design Divisions, Field Division, local officials, developers and/or local citizen groups. All requests for new traffic signal installations should be first forwarded to the appropriate Field Division Office. If the Field Division Office determines the request merits further investigation, it will then begin accumulating the necessary traffic data.

For in-house requests, the Field Division, in conjunction with the Planning Division, will conduct the appropriate traffic studies to obtain accurate and up-to-date traffic data and projections. For other requests, the latest traffic data and projections should be forwarded with the request. The data collector will need to refer to Part IV, Section C, of the MUTCD, which lists the 11 warrants for traffic signals, to determine the appropriate information required. For additional information on the collection of traffic data, the designer should review the ITE publication, *Manual of Traffic Engineering Studies*, or contact the ODOT Planning Division.

Once a legitimate request has been reviewed and the appropriate traffic data has been collected, the Field Division will then send the request and traffic data to the Traffic Engineering Division's Traffic Design Branch. The Traffic Engineering Division personnel (or consultant) will prepare a detailed analysis of the site using the warrants and considerations listed in the following sections.

14.5.2.2 Signal Warrants

At a minimum, all new traffic signals should meet the warrant criteria set forth in Part IV, Section C of the MUTCD. The 11 MUTCD traffic signal warrants are as follows:

Warrant 1 — Minimum vehicular volume
Warrant 2 — Interruption of continuous traffic
Warrant 3 — Minimum pedestrian volume
Warrant 4 — School crossings
Warrant 5 — Progressive movement
Warrant 6 — Accident experience
Warrant 7 — Systems
Warrant 8 — Combination of warrants
Warrant 9 — Four-hour volumes
Warrant 10 — Peak-hour delay
Warrant 11 — Peak-hour volume

14.5.2.3 Warrant Analysis

Even though traffic volumes may be sufficiently high, the installation of a traffic signal may not always be the most prudent choice. In addition to the MUTCD warrants, the designer should also consider the following:

1. Minimums. The MUTCD warrants are considered to be guidelines for determining the need for a traffic signal. The intent of MUTCD thresholds is to establish a minimum boundary below which a traffic signal should not be installed. Meeting or exceeding these thresholds does not automatically warrant a traffic signal.

2. Volume Warrants. MUTCD warrants 1 and 2 require the volume warrants to be satisfied for each of any 8 hours on an average day.

3. Benefits. The benefits of the traffic signal must outweigh its disadvantages. Traffic

signals will cause delays for at least one leg of the intersection when serving the needs of another. A traffic signal should be installed only if the operations of the whole intersection are improved and are more efficient.

4. Accidents. Traffic signals are often installed to reduce certain types of accidents (e.g., right-angle collisions, pedestrian crossings). However, the installation of a traffic signal may actually increase the number of rear-end collisions and may fail to reduce turning conflicts between vehicles and pedestrians. The designer needs to determine if a change in accident types and their severity will be an actual improvement for the intersection.

5. Geometrics. The geometric design of the intersection can affect the efficiency of the traffic signal. Installations of traffic signals at poorly aligned intersections may, in some cases, increase driver confusion and thereby reduce the overall efficiency of the intersection. If practical, the intersection should be properly aligned and have sufficient room to adequately lay out turning lanes, through lanes, etc. Chapter Nine provides additional information for at-grade intersection designs. Under some circumstances, the use of an interchange may be more economical than installing or upgrading traffic signals. Chapter Ten provides additional information on the design considerations for an overpass.

6. Costs. The installation and maintenance of traffic signals can be very expensive. A cost-effectiveness analysis may be required to determine if the benefits from the reduction in accidents and delays will actually exceed the costs for the signal.

7. Presentation Forms. Figure 14.5A illustrates an example of ODOT's preferred format for the presentation of the analysis results.

14.5.2.4 Local Responsibilities

It is ODOT's policy to assist in funding of the design and installation of a traffic signal only when the intersection is on a State or Federal highway or where a freeway exit or entrance ramp intersects with a local facility.

In accordance with the Oklahoma Statutes, all traffic signals, even on State highways, are to be maintained by the municipality or county. Only under rare circumstances, will the State take responsibility for the maintenance of a traffic signal (e.g., flashing beacons). For all traffic signals, an agreement is required between ODOT and the municipality or county to have the municipality or county take responsibility for the operation and maintenance of the signal equipment. The Traffic Engineering Division will be responsible for reviewing, preparing and/or coordinating these agreements.

14.5.3 Traffic Signal Equipment

All traffic signal equipment is to meet the criteria set forth in the MUTCD, NEMA *Traffic Control System*, *ODOT Standard Drawings* and *ODOT Standard Specifications*. The following sections provide additional information on the traffic signal equipment used in Oklahoma.

14.5.3.1 Traffic Controllers

There are two basic types of traffic controllers — pretimed and actuated. A pretimed controller operates according to pre-determined schedules. An actuated controller

TRAFFIC SIGNAL WARRANT SUMMARY FORM

TRAFFIC SIGNAL WARRANTS		INTERSECTION LOCATION					
		SITE 1 Maple Avenue & Main Street	SITE 2 First Avenue & Main Street	SITE 3 Grand & Main Street	SITE 4	SITE 5	SITE 6
* 1. Minimum Vehicular Volume		Yes	Yes	No			
2. Interruption of Continuous Traffic		Yes	Yes	No			
* 3. Minimum Pedestrian Volume		No	Yes	No			
* 4. School Crossing		No	No	No			
5. Progressive Movement		No	No	No			
* 6. Accident Experience		Yes	No	No			
7. Systems		No	No	No			
8. Combination of Warrants		No	No	No			
9. Four-Hour Volumes		Yes	Yes	Yes			
10. Peak-Hour Delay		Yes	Yes	Yes			
11. Peak-Hour Volume		Yes	Yes	Yes			
Signals Warranted	Yes	Yes	Yes				
	No			No			

* Include backup data with submission.

SAMPLE OF A TRAFFIC SIGNAL WARRANT SUMMARY FORM

Figure 14.5A

operates with variable vehicular and pedestrian timing and phasing intervals which are dependent upon traffic demands. If there is no demand for a phase, the actuated controller will omit that phase in the cycle (e.g., if there is not a demand for left turns, the left-turn phase will not be activated). The following sections provide general information on the various controllers used by ODOT.

14.5.3.1.1 Pretimed Controller

Pretimed controllers use a fixed, consistent predetermined cycle length, usually 40 to 120 seconds. They can be programmed to provide several different timing programs based on the time of day and/or day of week. Their use is best suited where traffic volumes and patterns are consistent from day to day (e.g., downtown areas). The controller can be either electromechanical or solid state. For new installation, the electromechanical controller is rarely used because the solid-state design has better expansion capabilities and replacement parts are simpler and easier to install. The following lists some of the advantages and disadvantages of the pretimed controller:

Advantages

1. They can be easily incorporated into a progression system.
2. They are not dependent on a detection device.
3. They are typically easier to operate and maintain than actuated controllers.
4. They have been commonly used in the past.
5. They require little additional training of local maintainers for proper operation and maintenance.
6. They can be readily upgraded to an actuated system.

Disadvantages

1. There is not an industry-wide interchangeability standard for replacement parts between different controller manufacturers.
2. They cannot compensate for short-term fluctuations in the traffic flows which can cause excessive vehicle delays.
3. They tend to be inefficient at isolated intersections.

Since the cost difference between a pretimed and actuated controller is minimal and actuated controllers can be set up to simulate pretimed controllers, ODOT has limited the use of pretimed controllers on State highways. Any proposed use of a new pretimed controller on a State highway must be first approved by the Traffic Engineering Division.

14.5.3.1.2 Semi-Actuated Controller

Semi-actuated controllers are based on vehicle detection from one or more approaches, but not on all approaches. Typically, vehicle detectors (e.g., loop detectors) are placed only on the minor approaches. The major approaches are kept in the green phase until a vehicle on the minor approach is detected. If there is a demand on the minor approach and the minimum green time for the major approach has elapsed, the right-of-way will then be given to the minor approach. To handle various fluctuations on the minor approach, the minor approach is given enough time to clear one vehicle with additional time added for each new detection up to the maximum green time. Once the minor approach demand has been satisfied or when its maximum green time has been reached the right-of-way is then returned to the major approach and the cycle begins again. If there is no minor approach demand the major approach will remain in the green phase indefinitely. The following lists some of the advantages and disadvantages of the semi-actuated controller:

Advantages

1. The major approach receives a green phase indefinitely until a vehicle is detected on the minor approach.

2. They can be easily incorporated into a coordinated system.

3. They can be effectively used at isolated intersections.

Disadvantages

1. Short continuous demands on the minor street (e.g., factory shift changes) can cause excessive delays to the mainline.

2. A detection device is required, typically loop detectors on the minor street.

3. They are typically more complex to operate than pretimed controllers.

14.5.3.1.3 Full-Actuated Controller

A full-actuated controller has detection devices on all approaches to the signalized intersection. The green phase on an approach will remain until a conflict call is received from the side or stopped approach. Continuous traffic on one street is not interrupted by an actuation demand from the side street until a gap appears or when the maximum green time has elapsed. Once the side street demand has been satisfied, right-of-way is then returned to the other street if a conflict detection has been registered. When there is a continuous demand for all approaches, the system tends to operate as a pretimed system. A full-actuated controller is an appropriate design choice when all approaches have equal traffic volumes and equal levels of hierarchy. The following lists some of the advantages and disadvantages of a full-actuated controller:

Advantages

1. They can handle high traffic volumes efficiently.

2. They are very efficient at isolated intersections.

3. They can handle varying traffic demands efficiently.

4. They can be programmed to operate as a pretimed signal or semi-actuated system.

5. They can be programmed to allow different phases to operate concurrently, if they are not conflicting phases.

Disadvantages

1. A detection device is required on all approaches, typically loop detectors.

2. They are more complex to operate and maintain.

3. With heavy traffic demands, they often provide only the minimum green time to each phase.

14.5.3.1.4 Volume-Density Controller

A volume-density controller is a more complex, full-actuated controller design. This type of controller is typically used at high-speed intersections. Additional detectors are placed well in advance of the intersection to determine vehicle gaps. The mainline vehicle gaps are adjusted to give priority to vehicle platoons versus conflict calls from the side streets. Once the gaps on the mainline are too long or the maximum green time has passed, the right-of-way is then given to the side streets to allow the waiting vehicles a chance to enter or cross the high-speed highway. The following lists some of the advantages and disadvantages of a volume-density controller:

<u>Advantages</u>

1. They are very efficient at high-speed intersections.
2. They can effectively handle large traffic volumes.
3. They can effectively clear stored traffic (e.g., stored vehicles in a left-turn bay).
4. They can be programmed to give higher priority to the mainline.
5. They can be programmed to allow different phases to operate concurrently, if they are not conflicting phases.
6. They can be programmed to handle local site conditions.

<u>Disadvantages</u>

1. Additional detection devices are required upstream of the traffic signal and on all approaches.
2. They are more complex to operate and maintain.
3. Maximum green times are routinely provided for each phase.
4. Typically, they have higher initial and maintenance costs.

14.5.3.1.5 Pedestrian Controller

A pedestrian controller commonly works in conjunction with one of the other controller types. These controllers allow for the timing of the "WALK" and "DON'T WALK" cycles. They are actuated by pedestrian push buttons or pedestrian sensors. The following lists some of the advantages and disadvantages of a pedestrian controller:

<u>Advantages</u>

1. They provide additional protection for crossing pedestrians.
2. When there is little pedestrian demand, disruption to the vehicular phases can be kept to a minimum.

<u>Disadvantages</u>

1. Pedestrian call buttons are required and must be located in a convenient location for both the able and handicapped.
2. Pedestrian cycles concurrent with green time may marginally delay right-turning vehicles.
3. Will significantly increase the required minimum green time on the minor street if the major street is significantly wider than the minor street.

14.5.3.1.6 Specialty Controllers

There are several other controllers that are used in traffic engineering designs (e.g., flashing beacons, emergency vehicle actuators, railroad grade-crossing signals). The use of these types of controllers is site specific and is to be designed on a case-by-case basis.

14.5.3.2 Detectors

The purpose for a detector is to determine the presence of a vehicle or pedestrian, or the passage of a moving vehicle. This presence or passage detection is sent back to the controller which adjusts the signal accordingly.

There are many types of detectors available which can detect the presence or passage of a vehicle. Typically, ODOT only uses the inductive loop and microloop detectors in its signal design. The inductive loop detector is preferred because it can be used for passage or presence detection, vehicular counts, speed determinations and is generally accurate and easy to maintain. Although the inductive loop detector is usually the system of choice, this does not prevent the designer from recommending the use of new devices in the future. If, in the designer's opinion, a new detector should be considered, its use must be first coordinated with the Traffic Engineering Division.

14.5.3.2.1 Inductive Loop Detector

An inductive loop detector (loop detector) design consists of two or more loops of wire embedded in the pavement surface. Figure 14.5B provides a schematic of a inductive loop detector. As a vehicle passes over the loop, it disrupts the current running through the wire. This disruption is recorded by an amplifier and transmitted to the controller as a vehicle detection.

The advantages of the loop detector are that they can:

1. detect vehicles in both presence and passage modes,
2. be used for vehicular counts and speed determinations, and
3. Be easily designed to meet the various site conditions.

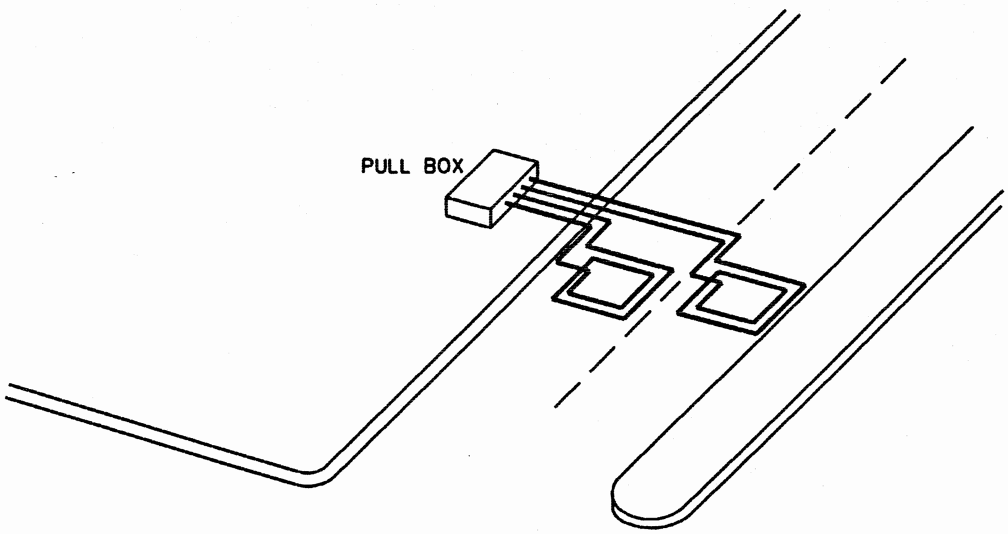
A major disadvantage of the loop detector is that it is very vulnerable to pavement surface problems (e.g., potholes) which can cause breaks in the loops. To alleviate this problem, a series of loops is often used.

There are basically two types of loop detector designs — the long loop design (6′x30′) and the short loop design (6′x6′). Figure 14.5C and the *ODOT Standard Drawings* illustrate typical loop detector design layouts and installation details. The designer needs to be aware that the typical layouts shown in Figure 14.5C and the Standard Drawings are for illustrative purposes only. Each intersection needs to be designed individually to meet local site conditions.

A series of loops (usually a 6'x 30') is used at the intersection itself for presence detection of vehicles stopped at the traffic signal. A series of short loops (6'x 6') is used to determine the passage of vehicles prior to the intersection. The spacing of these loops is determined from the design speed of the highway. Table 14.5A presents the typical loop detector spacings when they are used for passage detection.

14.5.3.2.2 Microloop Detectors

A microloop detector consists of a magnetic metal core with wrapped windings, similar to a transformer. This core is sealed in a



Notes: *1. All connections and splices shall be made in the pull box.*

2. See ODOT Standard Drawings for additional details.

Source (11)

SCHEMATIC OF INDUCTIVE LOOP DETECTOR

Figure 14.5B

cylinder about 1-inch in diameter and 4-inches long. Figure 14.5D illustrates a typical microloop installation.

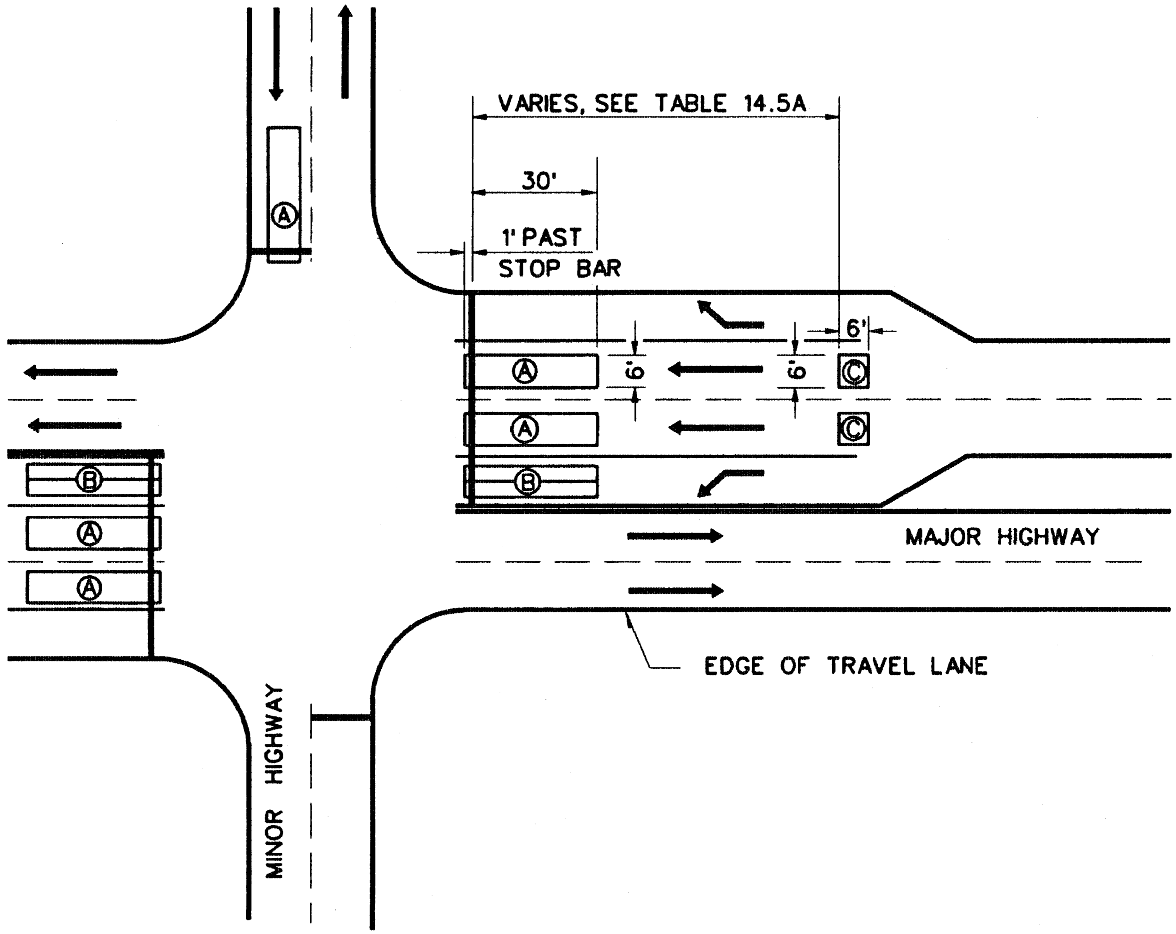
The microloop detector also can work with the standard inductive loop detector electronic units. The microloop is typically installed by drilling a 3-inch diameter hole 20 inches deep into the pavement structure or by securing it to the underside of a bridge deck (i.e., no drilling). A major disadvantage of the microloop detector is that it requires some motion to activate the triggering circuitry of the detector and does not detect stopped vehicles.

14.5.3.2.3 Pedestrian Detectors

The most common pedestrian detector is the pedestrian push or call button. These pedestrian call buttons should be placed so they are convenient to use, reachable by the handicapped and not placed in the direct path for the blind. Inconveniently placed pedestrian detectors is one of the reasons many pedestrians choose to cross the intersection illegally and unsafely.

14.5.3.3 Signal Poles

Under most circumstances, traffic signals are typically installed on mast arms which are placed on the far side of the intersection. This



Notes:

1. Detector loops A & B should operate in the presents mode.
2. Detector loop C should operate in the pulse mode. This detector unit should be furnished with a delay and extension time feature.
3. On the minor street where speeds are less than 30 mph, a 30′ x 6′ loop should be installed, detector loop A.
4. Where speeds are 30 mph and greater, the multi-loop design should be installed, detector loops A and C.
5. Detector loop B shall be quadruple.

TYPICAL LOOP DETECTOR LAYOUT

Figure 14.5C

Table 14.5A

LOOP DETECTOR SPACINGS FOR PASSAGE DETECTION[1]

Design Speed (mph)	First Detector[2] (ft)	Second Detector[3] (ft)
30-35	-1	180
40-45	-1	273
50-55	-1	386

Notes: 1. *Measured from front of STOP bar to front of loop detection.*

2. *Use 6′ x 30′ long loop design.*

3. *Use 6′ x 6′ short loop design.*

allows the signal heads to be placed directly over the turning and through lanes. In addition, the rigid mounting also allows for better control of the signal heads under wind-loading conditions. With the variety of materials available, mast arm designs can be designed to be aesthetically pleasing and made to match the neighborhood decor.

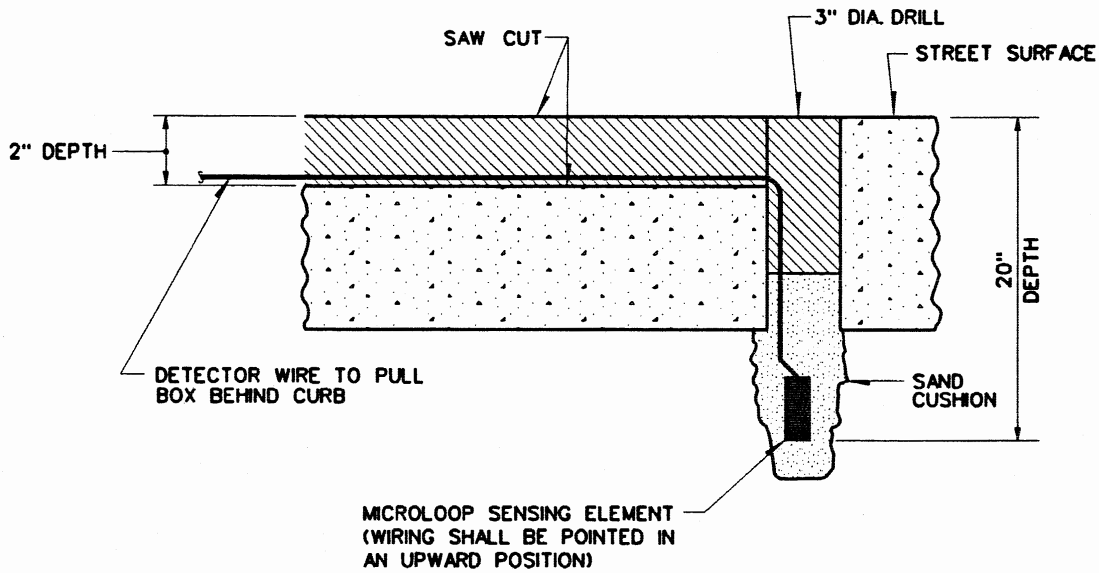
Pedestal- or post-mounted signals are often used if there is a left-turn signal in a median or on the near side of the intersection if the intersection is significantly wide. As with the mast arm, the post-mounted signal is a rigid design and can be designed to be aesthetically pleasing.

Under some temporary situations (e.g., temporary traffic signals in construction zones) signal heads may be placed on span wires. Some of the problems with the use of span wire designs include:

1. they do not provide enough rigidity under wind loading conditions,
2. the signal faces are harder to see on narrow streets,
3. pedestrians have a more difficult time seeing the signal faces, and
4. installations are often considered to be aesthetically unpleasing.

Consequently, it is ODOT's policy not to use span wire designs for permanent installations. The *ODOT Standard Drawings* provide typical layouts and design information on the temporary use of span wires for traffic signals installations.

It is also ODOT's policy to provide overhead highway lighting wherever traffic signals are used on State highways. Typically, the traffic signal post or mast arm pole is also used for the overhead highway lighting. The *ODOT Standard Drawings* present the design details for this signal pole. Section 14.6 provides information on the lighting design.



Note: The microloop detection is to be placed in a minimum 3-inch diameter x 20" deep hole in the pavement structure as by attaching it to the underside of a bridge deck without drilling.

MICROLOOP INSTALLATION

Figure 14.5D

14.5.3.4 Signal Display

The traffic signal display consists of many parts including the signal head, signal face, optical unit, visors, etc. The criteria set forth in Part IV, Section B of the MUTCD is to be followed in determining appropriate signal display arrangement and equipment. In addition, the following guidelines are provided for the selection of signal display equipment:

1. Signal Head Housings. Signal head housings can be made from either aluminum or plastic. Plastic is usually lighter and retains it color throughout its service life. However, plastic is not as strong as aluminum and tends to break when used in top- or bottom-mounted rigid installations.

2. Signal Faces. The Standard Drawings present ODOT's preferred signal face arrangements for use on State highways. It is ODOT's policy to place the signal lenses in a vertical line versus horizontally. Where protected left-turns are followed by permissive left-turns, the five-section cluster design is the recommended arrangement choice. Part IV, Section B of the MUTCD provides additional information on the arrangement of signal faces.

3. Lens Sizes. Although an 8-inch lens size is allowed by the MUTCD, ODOT's policy is to use only 12-inch lenses on State highways. When upgrading traffic signals, all signal faces with 8-inch lenses should be replaced with signal faces that have 12-inch lenses. In addition, it is ODOT's policy to only use glass lenses in its signal displays.

4. Backplates. It is ODOT's policy that backplates be installed on all signal heads. The Standard Drawings provide additional details on the backplates used in Oklahoma.

5. Signal Lamps. The designer is referred to ITE's *Traffic Signal Lamps* for ODOT's criteria on signal lamps. The ITE publication covers lamp illumination, light center length, rated "initial" lumens, lamp life and operating voltages.

6. Visors. ODOT's standard practice is to use a tunnel visor on all signals. In special cases, a cutaway visor may be used. These visors are typically used for two purposes — to direct the signal indication to the appropriate approaching traffic and to reduce "sun phantom." These visors are typically made of the same material as the housing. The decision on whether to use either the tunnel or cutaway visor is determined on a site-by-site basis.

7. Louvers. Louvers are sometimes used to direct the signal indicator to a specific lane (e.g., left-turn signal for a left-turn bay). Louvers are used where several signal heads may cause confusion for the approaching driver. One example of this problem is where a left-turn signal indicator is red, but the through lanes indicators are green. The decision on whether to use louvers depends on site conditions and will be determined on a case-by-case basis.

8. Optically Programmed Signals. Like louvers, optically programmed signals are designed to direct the signal indicator to specific approach lanes. A major advantage is that they can be narrowly aligned so that the adjacent lanes cannot see the indicator. Typical applications include closely spaced intersections, left-turn signals at skew intersections and left-turn signals on high-speed approaches. Optically programmed signals require

rigid mountings to keep the indicator properly directed. Although the initial cost may be higher than louvers, the advantage of being less confusing often makes them cost effective. The decision on whether to use a optically programmed signal depends on site conditions and will be determined on a case-by-case basis.

14.5.3.5 Conduit System

Electrical connections between the power supply, controller, detectors and signal heads are typically carried in a closed conduit system. The conduit system consists of electrical cables or wires, connectors, conduit and pull boxes. The designer should consider the following when developing the traffic signal wiring plan:

1. Service Connections. Service connections from the local utility lines should go directly to the controller and should be as short as practical. Preferably, these installations will be underground. Easy access to a shut off device in the controller is required to turn the power supply off when performing system maintenance.

2. Electric Cables. All electric cables and connections must meet national, state and local electrical codes in addition to the NEMA criteria. In general, the number of conductor cables should be kept to a minimum, usually only 3 or 4 combinations, to reduce inventory requirements.

3. Cable Runs. All electric cable runs shall be continuous between the controller, signal heads and pull boxes.

4. Pull Boxes. Pull boxes are to be located adjacent to the controller cabinet, each signal pole and each detector location. The *ODOT Standard Drawings* provide additional details on the design of pull boxes.

5. Underground Conduit. Underground conduit is used to connect the controller, traffic signals and loop detectors together. For conduit runs underneath the pavement and between the Size I and Size II pull boxes, 3-inch conduit should be used. Two 3-inch conduits are typically used between the controller cabinet and the Size II pull box. For runs between the traffic signal pull box to signal pole, 2-inch conduit should be utilized. For conduit runs between the detector pull boxes and traffic signal pull boxes, 1½-inch conduit is typically used. For runs with additional cables, the conduit size may need to be increased. The manufacturer's literature should be checked to determine the appropriate conduit size. Section 14.5.4 and the *ODOT Standard Drawings* provide additional details on the design and placement of underground conduit.

14.5.4 Traffic Signal Design

14.5.4.1 Design Criteria

In general, ODOT has adopted the MUTCD criteria for the placement and design of traffic and pedestrian signals. This includes, but is not limited to, signal indications, color requirements, number of lenses per signal face, number and location of signal faces, height of signal faces, location of signal supports, etc. In addition to the MUTCD, the *ODOT Standard Drawings* and references listed in Section 14.5.1.1 provide further details and information on the design of traffic signals.

14.5.4.2 Placement of Signal Equipment

For the most part, the designer has limited options available in determining acceptable locations for the placement of signal pedestals, mast arm signal poles, pedestrian detectors and controllers. From the roadside safety aspect, these elements should be placed as far back from the roadway as possible. However, due to visibility requirements, limited mast arm lengths, limited right-of-way, restrictive geometrics or pedestrian requirements, traffic signal equipment often must be placed relatively close to the travelway. The designer should consider the following when determining the placement of traffic signal equipment:

1. Clear Zones. If practical, the placement of traffic signal equipment should meet the clear zone criteria presented in Chapter Eleven.

2. Controller. In determining the location of the controller cabinet, the designer needs to consider the following:

    a. The controller cabinet should be placed outside the clear zone or in a position so that it cannot be struck by errant vehicles.

    b. The controller cabinet shall be located where the ease of access for maintenance is practical.

    c. Desirably, the controller cabinet should be located so that a technician working in the cabinet can see the signal heads in at least one direction.

    d. The controller cabinet shall be located where it is flood-proof or be raised above the flood stage for a Q_{100} storm.

    e. The power source should be reasonably close to the controller cabinet.

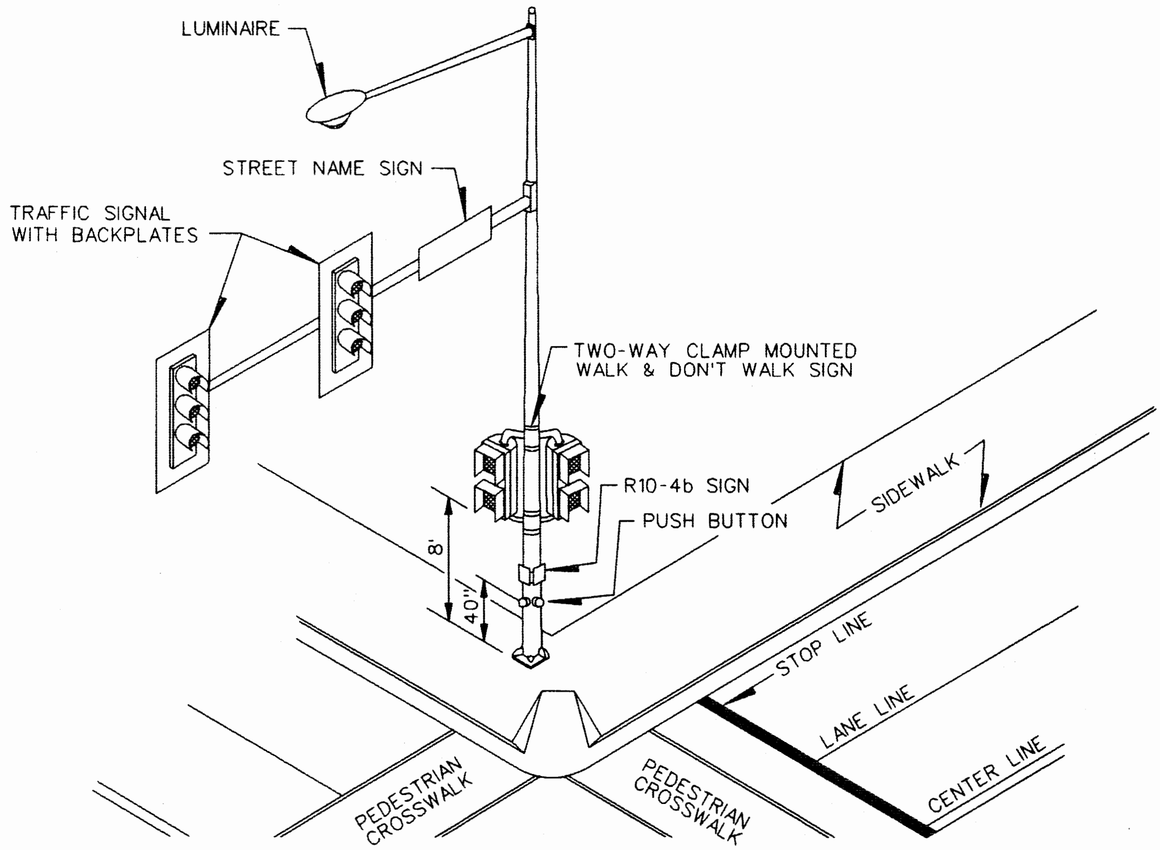
3. Signal Poles. If practical, all signal poles should be placed outside the clear zone. At a minimum, the following will apply:

    a. On urban curbed facilities and where the design speed is 45 mph or less, the center line of the signal pole should be at least 5 feet from the curb face. On non-curbed facilities, the signal pole should be 10 feet from the travelway.

    b. On rural, suburban or urban non-curbed facilities and where the design speed is 50 mph or greater, the signal pole should be at placed at least 2 feet outside of the shoulder and at least 10 feet from the travelway. Under these conditions, the signal pole may require some type of positive protection (e.g., guardrail, impact attenuators), see Chapter Eleven.

4. Pedestrians. If signal pole must be located in the sidewalk, it should not be placed in the direct path of pedestrians. In addition, the signal pole shall not be placed in a manner that will restrict the handicapped's access to curb ramps. Pedestrian call buttons must be located conveniently for both the able and the handicapped.

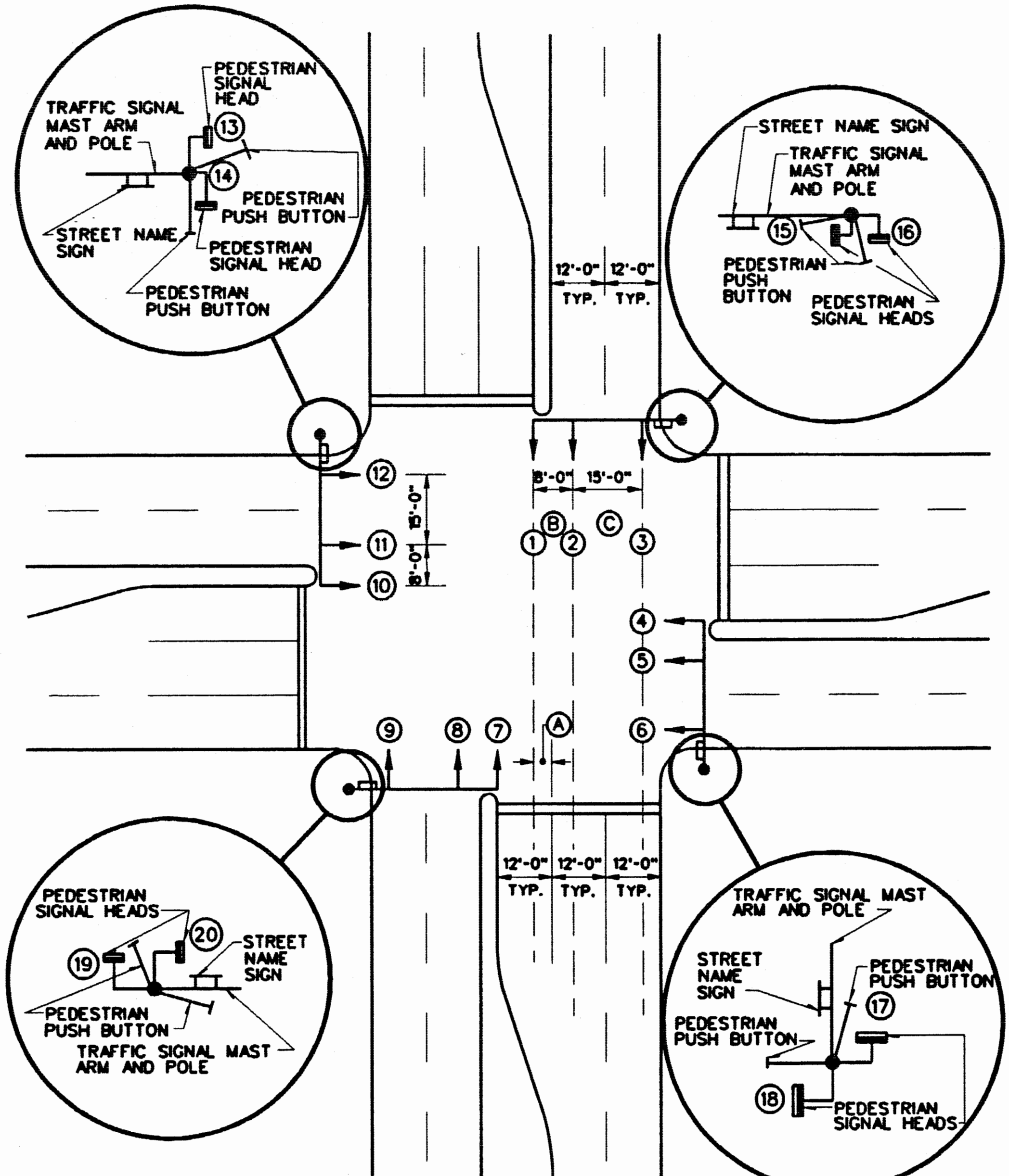
5. Layout. Figure 14.5E illustrates a typical traffic signal pole installation. Figure 14.5F shows a plan drawing schematic for traffic signal installations at an intersection. Figure 14.5G illustrates the typical placement for pull boxes and underground conduit.



Note: Where pedestrian traffic is proposed to walk between curb and signal pole, pushbutton and sign should be installed as shown. If proposed signal poles are located where pedestrian traffic will approach the pole from a different side, pushbutton and sign should be installed on pole facing pedestrian traffic or as shown on plans.

TYPICAL TRAFFIC SIGNAL POLE INSTALLATION

Figure 14.5E



Notes:

A. Left-turn signal head shall extend 2 to 3 ft in the left-turn bay.

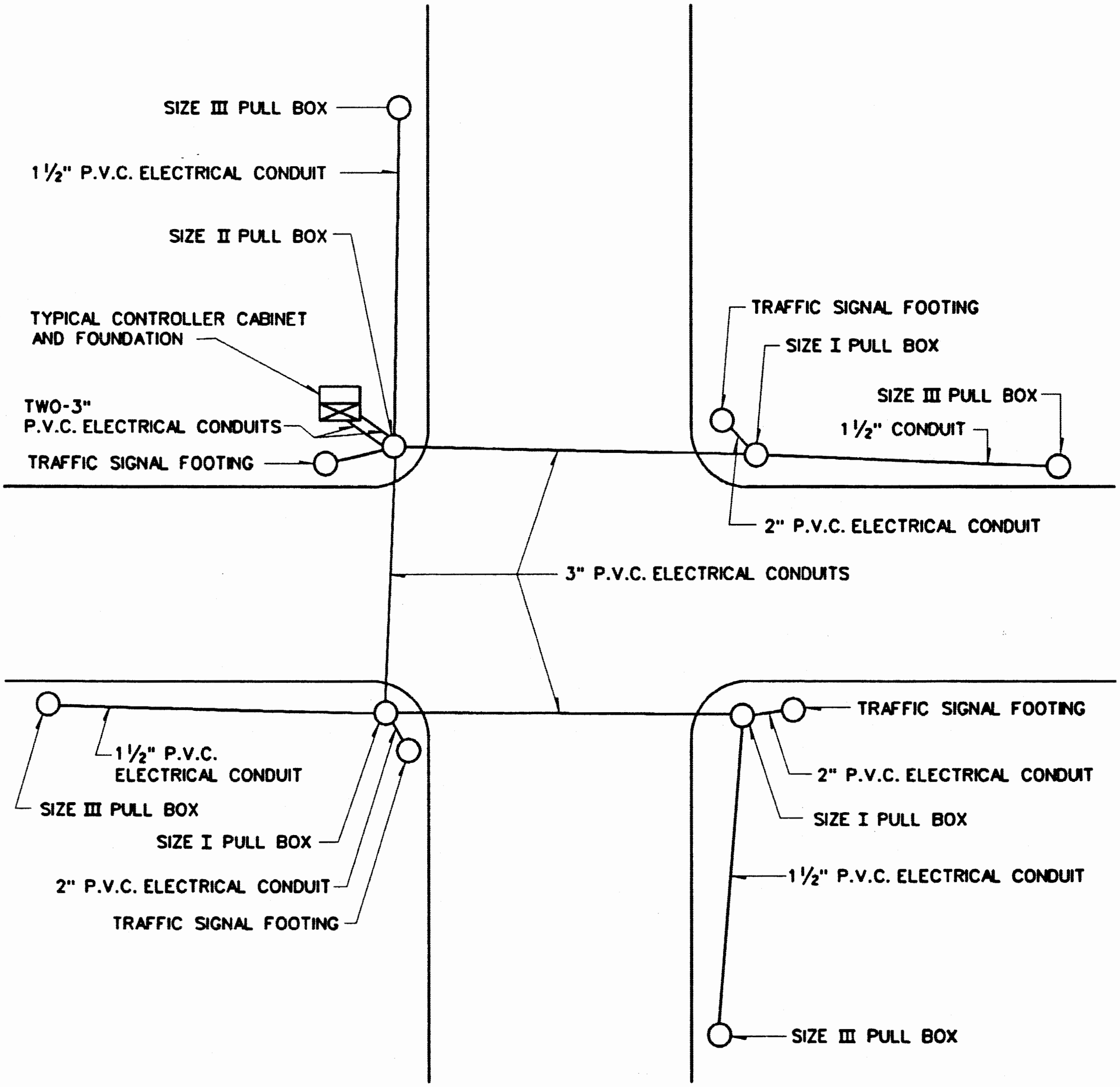
B. Spacing between signal heads #1 and #2 shall be a minimum of 8′-0″ (MUTCD).

C. Spacing for all through signal heads shall be a maximum of 15′-0″ and a minimum of 8′-0″ (MUTCD).

D. The minimum distance from the center of the traffic signal pole and the face of curb should be 5′-0″, see Section 14.5.4.

SCHEMATIC FOR TRAFFIC SIGNAL INSTALLATION

Figure 14.5F



Notes:

1. A Size I Pull Box shall be used at each quadrant of an intersection with the exception of quadrant where the controller unit will be located. In this quadrant, a Size II Pull Box shall be installed.

2. A Size III Pull Box shall be used for loop detector splicing.

3. See Section 14.5.4 for controller cabinet location requirements.

TYPICAL PULL BOXES AND CONDUIT PLACEMENT

Figure 14.5G

14.5.4.3 Audible Pedestrian Signals

Locations where sight handicapped pedestrians are anticipated to be may warrant the use of audible pedestrian "Walk/Don't Walk" signals. The use of audible signals will be determined on a site-by-site basis.

14.5.4.4 Local Considerations

After a signal is installed, it is the responsibility of the local municipality or county to operate and maintain the signal. The designer should review the local jurisdiction's existing traffic signal hardware and maintenance capabilities. Wherever practical, the designer should try to match the local jurisdiction's existing hardware, if it meets current ODOT criteria. This will reduce the municipality's need for additional resources and personnel training. However, this should not limit the designer's options, as there are several consultants who can help local governments operate and maintain any traffic signal.

14.5.5 Signal System Design

As traffic volumes continue to grow, the need for coordinated signal system designs becomes critical. By coordinating two or more traffic signals together, the capacity of the highway can be significantly increased. Although not a perfect solution, the use of a coordinated traffic signal system could satisfy the traffic needs of the highway for several years. It is also a relatively inexpensive method of improving capacity with minimal disruption to the highway as compared to constructing additional lanes. There are several different methodologies available to coordinate traffic signals. Most of these take advantage of recent advancements made in computer technology.

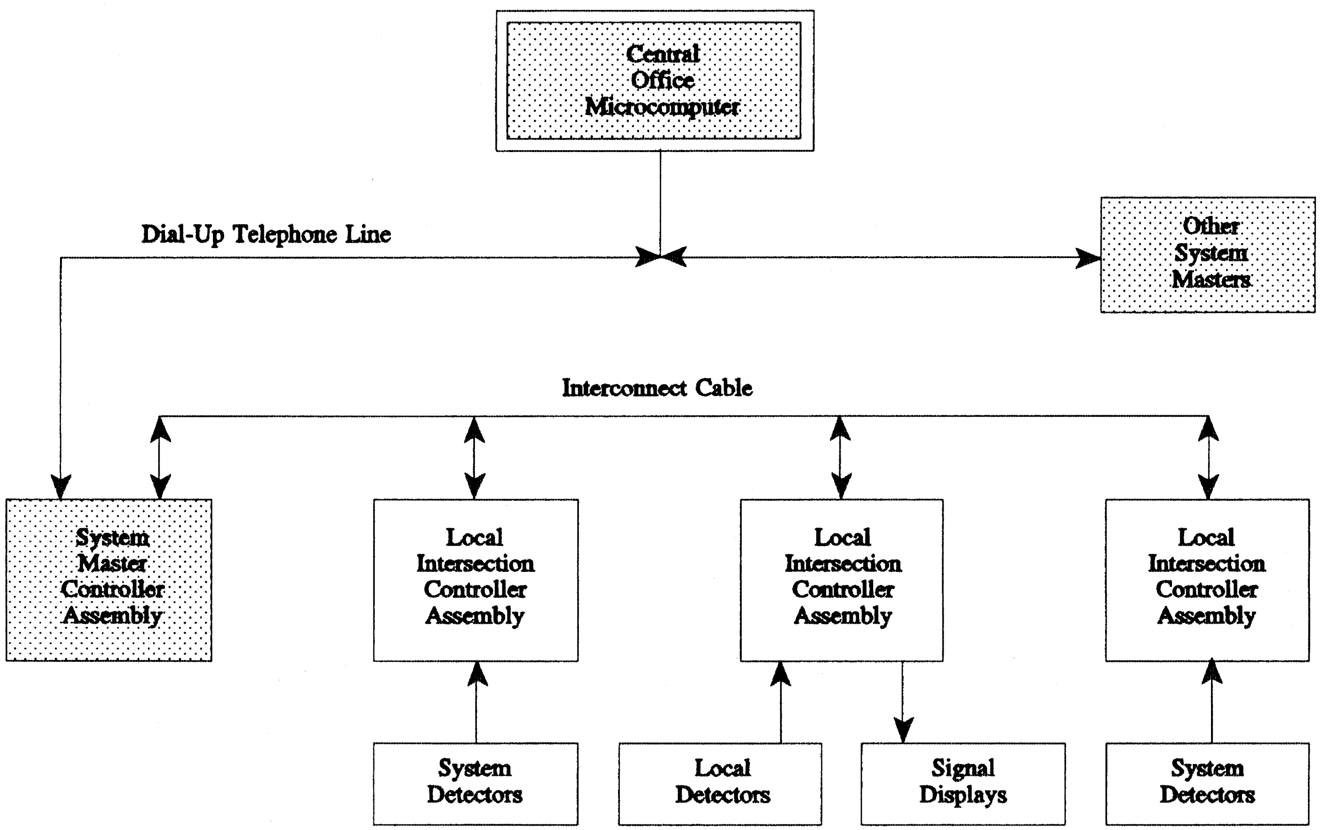
ODOT uses the "Distributed-Master" (Closed Loop) System. Figure 14.5H illustrates the schematics of this system. The closed loop system uses a master controller in the field to regulate the other local controllers in the system which are typically wired directly to the master controller. These local controllers feed information from their detectors back to the master controller. The master controller then adjusts the cycle lengths, splits and offsets for all the traffic signals in the system to efficiently move traffic through system. The master controller is also tied into a Central Office microcomputer through telephone lines. This allows the local authority, directly from a central office, to monitor the system, generate system reports or download additional commands and routines to the master controller. The Central Office's computer can control several coordinated systems at once.

As new signal controllers, computers and software are developed, the design of coordinated traffic signal systems will continue to improve. To maintain consistency, all traffic signal system designs are to be coordinated through the Traffic Engineering Division.

14.5.6 Operational Requirements

14.5.6.1 Phasing

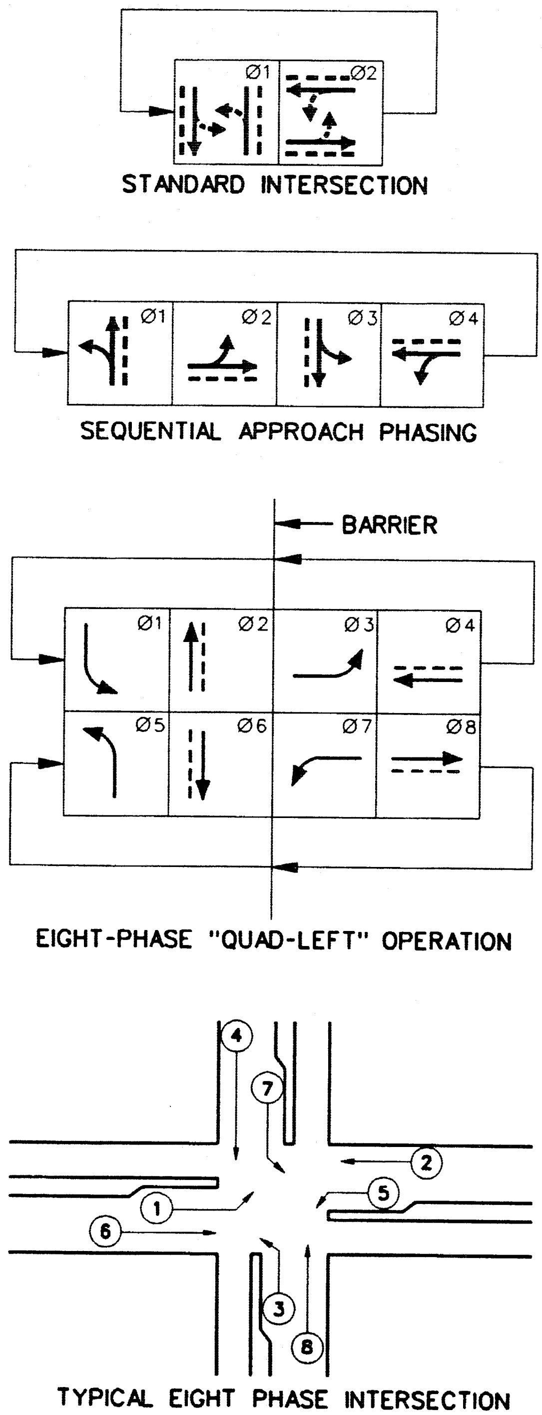
A signal phase is defined as the part of the traffic signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals. Each cycle can have 2 or more phases. For practicality, it is recommended that there be no more than 8 phases per cycle and desirably fewer. Figure 14.5I illustrates typical phasing schemes for single- and dual-ring operations.



Source (16)

DISTRIBUTED MASTER SYSTEM
(Closed Loop)

Figure 14.5H



Source (14, 15)

TYPICAL PHASING SCHEMES

Figure 14.5I

The most commonly added phases are for protected left-turns, i.e., left-turning vehicles are protected from opposing traffic. Left-turn phases can be either a lead left, where the protected left turn precedes the opposing through movements or a lag left, where the left-turn phase follows the opposing through movements. The decision on when to use either a lead-left or a lag-left turn will be determined on a case-by-case basis. Table 14.5B provides a comparison for each left-turn phase alternative.

Not all signalized intersections will require a separate left-turn phase. The decision on when to provide exclusive left-turn phases is dependent upon traffic volumes, delays and accident history, and is to be determined on a site-by-site basis. For intersections with exclusive left-turn lanes, the following are several guidelines that a designer may use to determine the need for a left-turn phase:

1. Volumes. A left-turn phase should be considered when the product of the left-turning and opposing peak-hour traffic volumes exceed 100,000 vehicles on a four-lane street or 50,000 vehicles on a two-lane street and where there are more than two left-turning vehicles per cycle during the peak hour.

2. Delay. A left-turn phase should be considered when there is a left-turn delay of 2.0 vehicle-hours or more on the critical approach during the peak hour, there are more than two left-turning vehicles per cycle during the peak hour and the average delay per left-turning vehicle is 35 seconds or more.

3. Accident History. The designer should review the accident history to determine the need for a separate left-turn phase. A left-turn phase may be warranted if there are several accidents that can be attributed to left-turning vehicles. This determination will be made on a site-by-site basis.

On approaches without an exclusive left-turn lane, the decision on whether to include a left-turn phase is determined on a site-by-site basis.

There are several computer programs available that can help the designer determine the appropriate phasing requirements, e.g., Signal Operations Analysis Package (SOAP). The Traffic Engineering Division can be contacted for more information on the latest software packages or versions used by ODOT.

14.5.6.2 Signal Timing

Developing an efficient signal timing plan is considered to be more an art that is developed over time than an exact science. For this reason, ODOT does not have a preferred signal timing methodology or program. In addition to transportation engineering text books, the sources listed in Section 14.5.4.1 provide information on several methodologies available to help a designer develop an appropriate signal timing plan. As a starting point in developing a signal plan, the following guidelines are offered:

1. Cycle Lengths. Initial cycle lengths of 60 seconds for rural intersections and 90 seconds for urban intersections are commonly used for planning purposes.

2. Phase-Change Intervals. Before yielding to the conflicting phase, sufficient time is required to adequately warn drivers of the pending change and to clear the intersection (i.e., yellow interval plus all red). Table 14.5C provides recommended clearance intervals for various design speeds and crossing street widths.

Table 14.5B

COMPARISON OF LEFT-TURN ALTERNATIVES

LEAD-LEFT-TURN PHASE	
ADVANTAGES	DISADVANTAGES
• Increases intersection capacity of one-or two-lane approaches without left-turn lanes when compared with two-phase traffic signal operation. • Minimizes conflicts between left-turn and opposing straight through vehicles by clearing the left-turn vehicles through the intersection first. • Drivers tend to react quicker than with lag-left operations.	• Left turns may preempt the right-of-way from the opposing through movement when the green is exhibited to the stopped opposing movement. • Opposing movements may make a false start in an attempt to move with the leading green vehicle movement.
LAG-LEFT-TURN PHASE	
• Both directions of straight through traffic start at the same time. • Approximates the normal driving behavior of vehicle operators. • Provides for vehicle/pedestrian separation as pedestrian usually crosses at the beginning of straight through green. • Where pedestrian signals are used, pedestrians have cleared the intersection by the beginning of the lag green interval. • Cuts off only the platoon stragglers from adjacent interconnected intersections.	• Left-turning vehicles can be trapped during the left-turn yellow change interval as through traffic is not stopping as expected. • Creates conflicts for opposing left turns at start of lag interval as opposing left-turn drivers expected both movements to stop at the same time. • Where there is no left-turn lane, an obstruction to the through movement during the initial green interval is created. • These disadvantages inherent in lag-left operations are such that its use is generally restricted to interconnected or pretimed operations or to a few specific situations in actuated control, such as "T" intersections.

Source (11)

Table 14.5C

RECOMMENDED CLEARANCE INTERVALS
(Yellow Plus All-Red Clearance)
(Seconds)

APPROACH SPEED (mph)	YELLOW CHANGE INTERVAL (seconds)	TOTAL CLEARANCE INTERVAL* FOR CROSSING-STREET WIDTHS (FEET)				
		30	50	70	90	110
20	3.0	4.2	4.9	5.5	6.2	6.9
25	3.0	4.2	4.7	5.3	5.8	6.4
30	3.2	4.3	4.8	5.2	5.7	6.2
35	3.6	4.5	4.9	5.3	5.7	6.1
40	3.9	4.8	5.1	5.5	5.8	6.1
45	4.5	5.1	5.4	5.7	6.0	6.3
50	4.7	5.3	5.6	5.9	6.2	6.4
55	5.0	5.7	5.9	6.2	6.4	6.7

Source (11)

* Based on following equation:

$$CP = T + \frac{V}{2a} + \frac{W + L}{V}$$

Where:
- CP = nondilemma change period (yellow plus all red), sec.
- t = perception - reaction time (1 sec.)
- V = approach speed, ft/s
- a = deceleration rate (10 ft/s^2)
- W = width of intersection, ft
- L = length of vehicle (20 ft)

3. Minimum Green Interval. The minimum green interval is the time required to clear any stored vehicles between the stop bar and detector and is only applicable for actuated signals. Table 14.5D presents the recommended minimum green intervals for semi- and full-actuated signals. For volume-density actuated signals, the minimum green interval is usually set at 10 to 15 seconds.

4. Maximum Green Interval. The maximum green interval is the maximum time a controller holds the green on the existing phase if there is an actuation detection on the conflicting phase. This time interval typically ranges from 30 to 60 seconds, but is determined on a site-by-site basis.

5. Pedestrian Clearance Intervals. The designer needs to consider pedestrian clearance intervals when there is an exclusive pedestrian phase, or if the pedestrian phase runs concurrently with traffic at wide intersections with short minimum green intervals. Walking rates of approximately 4 ft/sec are typically used at most intersections. However, in locations where very young children, elderly and/or handicapped people are known to be present, walking rates of 3.5 ft/sec are recommended. Table 14.5E presents recommended pedestrian clearance intervals for both 3.5 and 4 ft/sec walking rates.

   When using separate pedestrian displays (i.e., WALK, DON'T WALK), the MUTCD recommends that 4 to 7 seconds be provided for the "WALK" cycle. For Oklahoma, the use of 4 seconds is recommended where there are less than 10 pedestrians per cycle. The use of 7 seconds is recommended at intersections with high-pedestrian crossings (e.g., central business districts).

14.5.6.3 Computer Software

There are numerous software programs available to help the designer prepare traffic signal designs and timing plans. New programs, as well as updates to existing programs, are continuously being developed. Before using these programs the designer should contact the Traffic Engineering Division to determine which software package or version ODOT is currently using.

Most of these software programs can be purchased from either McTrans Center, 512 Weil Hall, Gainesville, Florida 32611-2083; or from PC-TRANS, Kansas University Transportation Center, 2011 Learned Hall, Lawrence, Kansas 66045. Many of these software programs can be purchased for either the mainframe or PC-based computer.

Table 14.5D

RECOMMENDED MINIMUM GREEN INTERVAL

DISTANCE BETWEEN STOP BAR AND DETECTOR, (Feet)	MINIMUM GREEN INTERVALS, (Seconds)
0 to 40	8
41 to 60	10
61 to 80	12
81 to 100	14
101 to 120	16

Source (11)

Table 14.5E

RECOMMENDED PEDESTRIAN CLEARANCE INTERVALS (Seconds)

CROSSING STREET WIDTH, (Feet)	MINIMUM CLEARANCE INTERVALS*	
	@ 3.5 ft/sec	@ 4.0 ft/sec
40	10.0	8.8
50	12.9	11.3
60	15.7	13.8
70	18.6	16.3
80	21.4	18.8

*Based on street width minus 5 feet for distance to center of furthest lane (assuming no parking).

Source (11)

TRAFFIC SIGNAL WARRANT SUMMARY FORM

TRAFFIC SIGNAL WARRANTS		INTERSECTION LOCATION					
		SITE 1	SITE 2	SITE 3	SITE 4	SITE 5	SITE 6
* 1. Minimum Vehicular Volume							
2. Interruption of Continuous Traffic							
* 3. Minimum Pedestrian Volume							
* 4. School Crossing							
5. Progressive Movement							
* 6. Accident Experience							
7. Systems							
8. Combination of Warrants							
9. Four-Hour Volumes							
10. Peak-Hour Delay							
11. Peak-Hour Volume							
Signals Warranted	Yes						
	No						

* Include backup data with submission.

14.6 HIGHWAY LIGHTING

14.6.1 General

The purpose of highway lighting is to provide a safe and comfortable environment for the night time driver. Due to the voluminous nature of highway lighting system designs, it would be impractical for this section to constitute a highway lighting design guide. For detailed design information, the reader is encouraged to the review the latest editions of the references listed in Section 14.6.1.1. The intent of this section is to provide the user with a synopsis of the highway lighting design issues and to present ODOT's positions, policies and procedures on these issues.

14.6.1.1 References

For additional information on highway lighting designs, the designer is referred to the following publications:

1. *An Informational Guide for Roadway Lighting*, AASHTO;
2. *Roadway Lighting Handbook*, FHWA;
3. *Roadway Lighting Handbook*, Addendum "Designing the Lighting System - Using Pavement Luminance," FHWA;
4. *Roadway Lighting*, RP-8, Illuminating Engineering Society;
5. NCHRP Report No. 152, *Warrants for Highway Lighting*, TRB;
6. NCHRP Report No. 256, *Partial Lighting of Interchanges*, TRB;
7. *Roadside Design Guide*, AASHTO;
8. *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*, AASHTO;
9. Technical Bulletin No. 270 *A Guide to Standardized Highway Lighting Pole Hardware*, ARTBA;
10. National Electrical Code;
11. National Electric Safety Code;
12. *Guide for the Design of Roadway Lighting*, Roads and Transportation Association of Canada; and
13. Various manufacturers' publications.

14.6.1.2 Definition of Terms

The following is a list of the more commonly used terms in highway lighting:

1. Average Initial Illuminance. The average level of horizontal illuminance on the pavement area of a traveled way at the time the lighting system is installed when lamps are new and luminaires are clean; expressed in average footcandles for the pavement area.
2. Average Maintained Illuminance. The average level of horizontal illuminance on the roadway pavement when the output of the lamp and luminaire is diminished by the maintenance factors; expressed in average footcandles for the pavement area.
3. Candela. The unit of luminous intensity. Formerly the term "candle" was used.
4. Candlepower. A measure of luminous intensity in a specified direction; expressed in candelas.

5. Equipment Factor (EF). A factor used in the illuminance or luminance calculations which compensates for light losses due to normal production tolerances of commercially available luminaires when compared with laboratory photometric test models.

6. Footcandle. The illuminance on a surface one square foot in area on which there is uniformly distributed a light flux of one lumen.

7. Footlambert. The unit of photometric brightness (luminance). It is equal to $1/\pi$ candela per square foot, or the uniform luminance of a perfectly diffusing surface emitting or reflecting light at the rate of one lumen per square foot.

8. Glare. The optical sensation produced by luminance within the visual field that is sufficiently greater than the luminance to which the eyes are adapted to cause annoyance, discomfort or loss in visual performance and visibility.

9. Illuminance. The density of the luminous flux incident on a surface. It is the quotient of the luminous flux by the area of the surface when the latter is uniformly illuminated.

10. Lamp Lumen Depreciation Factor (LLD). A depreciation factor that indicates the decrease in a lamp's initial lumen output over time. For design calculations, the initial lamp lumen value is reduced by a lamp lumen depreciation factor (LLD) to compensate for the anticipated lumen reduction. This factor is usually found in the manufacturer's test data.

11. Longitudinal Roadway Line. A line along the roadway parallel to the curb or shoulder line.

12. Lumen. A unit of measure of the quantity of light. One lumen is the amount of light which falls on an area of one square foot every point of which is one foot from the source of one candela (candle). A light source of one candela emits a total of 12.57 lumens.

13. Luminaire. A complete lighting unit consisting of a lamp or lamps together with the parts designed to distribute the light, to position and protect the lamps and to connect the lamps to the power supply.

14. Luminaire Dirt Depreciation Factor (LDD). A depreciation factor that indicates the expected reduction of a lamp's initial lumen output due to the accumulation of dirt on or within the luminaire over time.

15. Luminance. The luminous intensity of any surface in a given direction per unit of projected area of the surface as viewed from that direction.

16. Luminous Efficiency. The quotient of the luminous flux emitted by the total lamp power input. It is expressed in lumens per watt.

17. Maintenance Factor (MF). A combination of factors used to denote the reduction of the illumination for a given area after a period of time compared to the initial illumination on the same area (MF = EF + LLD + LDD).

18. Mounting Height. The vertical distance between the roadway surface and the center of the light source in the luminaire.

19. Nadir. The vertical axis which passes through the center of the luminaire light source.

20. Spacing. The distance in feet between successive lighting units.

21. Transverse Roadway Lane. Any line across the roadway that is perpendicular to the curb or shoulder line.

22. Uniformity of Illuminance. The ratio of average footcandles of illuminance on the pavement area to the footcandles at the point of minimum illuminance on the pavement. It is commonly called the uniformity ratio. A uniformity ratio of 3:1 means that the average footcandle value on the pavement is three times the footcandle value at the point of least illuminance on the pavement.

23. Uniformity of Luminance. The Average Level-to-Minimum Point method uses the average luminance of the roadway design area between two adjacent luminaires. The luminance uniformity (avg./min. and max./min.) considers traveled portion of the roadway, except for divided highways having different designs on each side.

24. Visibility. The quality or state of being perceivable by the eye. In outdoor applications, visibility is defined in terms of the distance at which an object can be just perceived by the eye.

25. Veiling Luminance. A luminance superimposed on the retinal image which reduces its contrast. It is this veiling effect produced by bright sources or areas in the visual field that results in decreased visual performance and visibility.

14.6.1.3 Project Initiation Procedures

If a municipality, agency or other local group desires a new lighting installation along a State-maintained highway, the following procedure should be employed:

1. The municipality, agency or other local group seeking the lighting system is required to submit a request to ODOT's Field Division Office petitioning ODOT to consider the installation of a new lighting system along the State highway.

2. If the Field Division Office determines the request justifies further action, it will submit the request to the Traffic Engineering Division for further consideration.

3. The Traffic Engineering Division will evaluate the site to determine if it meets the warranting criteria in Section 14.6.2.

4. After the Traffic Engineering Division has made its evaluation and determination, it will submit its recommendation to the Field Division Office.

5. Upon receipt of the Traffic Engineering Division's recommendation, the Field Division Office or the Traffic Engineering Division will take the following actions:

    a. If the recommendation is for approval, tell the requesting party that ODOT agrees that lighting is warranted. If the municipality agrees to pays its share of the installation and all of the operating and maintenance costs, the Field Division Office or the Traffic Engineering Division will begin the process of initiating a project to install lighting.

    b. If the recommendation is for disapproval, tell the requesting party that their petition has been denied. If the municipality or local group desires, they can still install the lighting under a Utility Permit if they agree to provide 100 percent of the cost for design, construction, operating and maintenance of the system. The system, however, still must meet ODOT's requirements for lighting utility permits and the roadside safety considerations presented in Section 14.6.4.

14.6.2 Warrants

Although beneficial, providing lighting along all highways is not practical or cost effective. ODOT's Traffic Engineering Division should determine that a lighting system is economically justifiable. However, any location meeting these warrants does not obligate ODOT to provide funding for the requested highway lighting projects. ODOT's objective is to identify those roadways which should be considered in the process of setting priorities for the allocation of available funding to roadway lighting projects. For a lighting system to be considered, it should meet the warrants provided in the following sections.

14.6.2.1 General Warrants

Any proposed lighting system should meet all of the following criteria:

1. The highway section must be within the city limits or jurisdiction of the local government unit making the request.
2. The local government unit must be willing to execute an agreement in which they agree to provide all operating and maintenance costs and to contribute their portion of the construction costs as specified by the Oklahoma Transportation Commission policies.
3. The proposed project area must indicate a need for highway lighting after being analyzed in accordance with the warranting conditions listed in the following sections

14.6.2.2 Freeways

ODOT's highway lighting warrants for freeways and expressways should conform to the recommended warrants presented in the AASHTO publication *An Informational Guide for Roadway Lighting*.

14.6.2.3 Interchanges

ODOT's highway lighting warrants for interchanges should conform to the recommended warrants presented in the AASHTO publication *An Informational Guide for Roadway Lighting*. It is ODOT's preferred practice that once it has been determined that lighting is warranted, complete interchange lighting will be provided. For additional information on interchange lighting warrants, the designer is referred to NCHRP Report No. 256, *Partial Lighting of Interchanges*. The final determination for complete or partial interchange lighting will be determined on a site-by-site basis.

14.6.2.4 Warrants for Streets and Highways, Other Than Freeways

All intersections which warrant traffic signals should, when practical, also include highway lighting by using a combination traffic signal and lighting pole.

Warrants for highway continuous lighting on highways and streets other than freeways should meet at least two of the following criteria:

1. <u>Traffic Volumes</u>. The highway or street should meet one of the following conditions:

a. A two-lane highway section should have an ADT in excess of 6,000 vehicles.

b. A four-lane highway section should have an ADT in excess of 12,000 vehicles.

c. An urban intersection should have a total ADT for all approaches in excess of 10,000 vehicles.

d. The main line at a rural intersection should have an ADT in excess of 4,000 vehicles.

2. Accidents. Lighting may be considered when a highway section or intersection has a night-to-day accident ratio of 1.5:1 or greater. This ratio is to be verified by analyzing the accident studies for the highway section and comparison with similar locations statewide.

   Lighting may also be considered where highway sections have the potential for numerous accidents, such as numerous driveways, channelized islands, significant commercial or residential development, a high percentage of trucks, or geometric deficiencies.

3. Surrounding Area. The area adjacent to the roadway section should be substantially developed and with considerable existing lighting visible from the roadway.

4. Cost Effective. The project should provide a positive benefit-cost ratio. In addition to the equipment and construction costs, the designer also needs to consider the operating and maintenance costs.

5. Social. In cases where the local government finds sufficient benefits in the form of convenience, safety, security, or other social or economic benefits, they may elect to pay an additional appreciable percentage of the installation costs required by Section 14.6.2.4.1 or they may elect to wholly finance the costs of construction and maintenance. If so, see Section 14.6.2.4.2.

14.6.2.4.1 Funding

GENERAL: Any location that meets the minimum warranting conditions does not obligate ODOT to provide funding for the requested highway lighting project.

1. The Oklahoma Transportation Commission may participate in the construction cost of warranted highway lighting projects erected on the state highway system in cities, towns or communities based on the following ratios;

   a. 50% - 50%, when Federal funds are not available;

   b. when Federal funds are utilized, the local government shall participate in the cost based on the funding ratio designated by the Federal-aid program requirements.

2. The local government's share of the estimated project costs shall be on deposit with ODOT's Comptroller prior to actual award of the contract or be in accordance with ARTICLE V-B, Section 5, of the ODOT's Commission Policies.

3. Lighting projects involving the expenditure of State or Federal funds shall become the property of the Oklahoma Department of Transportation. In the event the section of highway on which the lighting project is constructed is

transferred to another jurisdiction outside the area of ODOT's responsibility, the ownership of the lighting system shall also be transferred to the jurisdiction assuming responsibility for the highway section.

14.6.2.4.2 Design and Construction

1. Highway lighting projects that are funded in accordance with Section 14.6.2.4.1, shall meet the minimum design, construction standards and specifications of ODOT, set forth in this chapter of the *Design Manual*.

2. Highway lighting installations not requiring an expenditure of funds by ODOT shall be controlled by means of a permit system in accordance with ARTICLE V-B, Section 12, of ODOT's *Utilities Manual*. These "Utility" lighting systems shall follow the safety criteria established by Section 14.6.4 and should be designed in accordance with Section 14.6.6.

14.6.2.5 Highway Sign Lighting

In general, all overhead signs are to be lighted, if an agreement can be reached with the local government for the sign lighting maintenance, and if a power source is reasonably close to the overhead sign.

14.6.2.6 Special Areas

Lighting should be considered at the following locations:

1. rest areas,
2. weigh stations,
3. tunnels,
4. underpasses in lighted areas,
5. commuter park-and-ride lots,
6. bikeways,
7. walkways, and
8. other pedestrian facilities.

The need for lighting at these locations will be determined on a case-by-case basis.

14.6.3 Lighting Equipment

There are a variety of options available to the designer in selecting luminaire equipment that will meet the desired design criteria. In addition to the *ODOT Standard Drawings*, the following sections provide guidance on ODOT's preferred lighting equipment.

The designer should ensure that the selected equipment meets standard hardware designs. Specialized equipment and designs can significantly increase the installation and maintenance costs, thereby reducing the cost effectiveness of the lighting system. ARTBA Technical Bulletin No. 270, *A Guide to Standardized Highway Lighting Pole Hardware*, provides suggested details and specifications for standard lighting hardware. The designer may also contact the manufacturers for details and specifications on their standard hardware stocks.

14.6.3.1 Light Sources

There are numerous light sources available that could possibly be used for highway lighting. However, there are only a few practical choices when considering availability, size, power requirements and cost effectiveness. The following provides information on the light sources that are acceptable for use along State highways:

1. High Pressure Sodium (HPS). Due to its excellent luminous efficiency, power usage and long life, the HPS is the most commonly used light source with new installations. ODOT is almost exclusively

using HPS for all its installations of conventional and high mast lighting. The HPS lamp produces a soft, pinkish-yellow light by passing an electric current through a sodium and mercury vapor.

2. Low Pressure Sodium (LPS). Low pressure sodium is considered to be one of the most efficient light sources on the market. Its disadvantage is that it requires very long tubes and has poor color quality. Generally, low pressure sodium installations are limited to tunnels and underpasses. The low pressure sodium lamp produces a yellow light by passing a electrical current through sodium vapor.

3. Mercury Vapor (MV). Prior to the introduction of HPS, the mercury vapor was the most commonly used light source. Its usage by ODOT is generally limited to overhead sign lighting. The mercury vapor lamp produces a bluish-white light.

14.6.3.2 Light Poles

A major factor in highway lighting design is the selection of the luminaire and the mounting height. Higher mounting heights usually reduce the number of poles required. The *Standard Drawings* provide ODOT's criteria for light poles. The following describes the light poles commonly used by ODOT:

1. Conventional. This pole type is the most commonly used in lighting along highways. These poles have mounting heights ranging from 30 to 80 ft. ODOT's typical practice is to use a light pole with a mounting height between 40 to 50 ft. The recommended minimum mounting height is 40 ft. In selecting the mounting height, the designer should consider the local government's ability to maintain the lighting system.

2. High Mast. High mast poles range from 80 to 150 ft. This pole is an excellent choice where there is a large area that requires lighting (e.g., interchanges). The use of high mast lighting and higher watt lamps greatly reduces the number of poles, but yet still retains the quality of the lighting. The designer should consider using high mast lighting wherever practical.

3. Materials. Light poles for permanent installations are typically made from galvanized steel or aluminum. The *ODOT Standard Drawings* provide information and design guidance on the typical galvanized steel poles utilized by ODOT. Wood poles are normally used for service poles or on temporary lighting projects (e.g., construction zones), or may be allowed on highway lighting Utility Permits.

4. Breakaway Bases. Unless otherwise protected, all light poles within the clear zone along rural and high-speed urban highways should be provided with breakaway bases. However, where pedestrians are commonly present, breakaway designs should not be used. Typical breakaway supports include frangible bases (cast aluminum transformer base), slip bases and frangible couplings (couplers). ODOT's preferred practice is to use the cast aluminum transformer base. All breakaway bases shall meet the breakaway criteria set forth in the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*.

14.6.3.3 Mast Arms

Mast arms allow the light source to be placed near the edge of the driving lane. The use of the longer mast arms are recommended, although the initial costs may be higher. Longer mast arms allow the poles to be placed further from the travelway, thus providing a safer roadside environment. Mast arms are typically made from galvanized steel. The *ODOT Standard Drawings* provide information and design guidance on the typical mast arms used by ODOT.

14.6.3.4 Luminaires

A luminaire is defined as a complete lighting unit consisting of a lamp or lamps together with the parts designed to distribute light. The *Standard Drawings* provide ODOT's specifications on luminaire hardware. For additional information, the designer is also encouraged to contact the Traffic Engineering Division for the latest approved products list and by contacting the appropriate manufacturers.

14.6.3.5 Other Equipment

In developing a highway lighting system there are numerous components of the equipment design that can affect the design. These include ballasts, fuses, photoelectric controls, wiring, conduit, pull boxes, breakaway bases, etc. In addition to the *ODOT Standard Drawings*, the designer is encouraged to contact the Traffic Engineering Division for the latest information on the manufacturers' equipment specifications contained in an approved products list.

14.6.4 Roadside Safety Considerations

The placement of light poles should be installed in a manner that will not reduce the roadside safety. However, the physical roadside conditions often dictate the placement of light poles. It is important that the designer evaluate these limitations in the design process. Overpasses, sign structures, guardrail, roadway curvature, right-of-way limitations, gore clearances, the proximity of other existing roadside obstacles and the limitations of the lighting equipment are all factors which must be taken into consideration during the design. The designer needs to consider many other factors such as the roadway and area classification, design speed and/or posted speed limits, safety aesthetics, economics, environmental impacts, etc., while accounting for the physical limitations.

There should be adequate right-of-way, driveway control and utility clearance to allow the placement of the proposed lighting system in accordance with these safety requirements. Otherwise, the local government may be requested to provide additional right-of-way, driveway control and/or utility relocations.

The designer should consider the following when determining the location of light poles:

1. Whenever practical, poles should be placed outside the roadside clear zone, see Section 11.2.

2. Poles placed within the clear zone shall be provided with a breakaway device and/or maintain the minimum clearance distance specified in Table 14.6A.

3. All breakaway devices shall comply with all applicable AASHTO requirements for structural supports and may be one of the several forms that have been approved for

Table 14.6A

MINIMUM SUPPORT POLE CLEARANCES

Speed Limit or Posted Speed (mph)	Minimum Clearance Distance		
	Roadways Without Non-Mountable Curb With Breakaway (ft)	Roadways With Non-Mountable Curb With Breakaway (ft)	Roadways With Non-Mountable Curb Without Breakaway (ft)
55 or more	16	16	NR
50	14	14	NR
45	12	10	14
40	10	8	12
35 or less	8	4	8

NR = Not Recommended

Note: The above clearance distances are minimum and a greater distance is to be preferred, but are dependent on the roadside features and right-of-way. Therefore, each lighting system must be evaluated individually. A breakaway design is not required when the pole is placed beyond the recommended clear zones as presented in Section 11.2

use as a breakaway device, see Section 14.6.3.

4. All poles that require a breakaway device should be served by underground wiring.

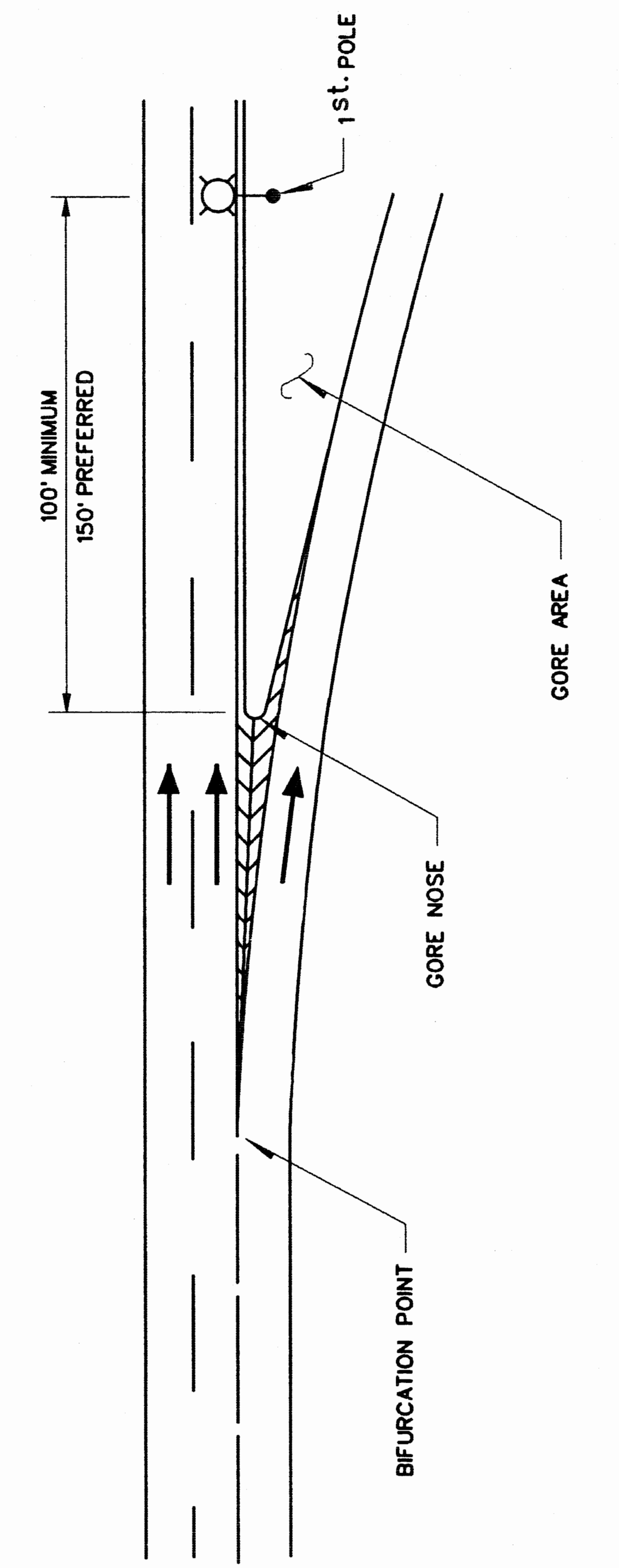
5. Poles should desirably be located to provide adequate safety clearance in the gore areas of the exit and entrance ramps. Figure 14.6A illustrates ODOT's criteria for placement of light poles within exit ramp gore areas.

6. Poles should desirably be placed on the inside of sharp curves and loops. However, they shall have sufficient clearance to avoid being struck by trucks if the radius is superelevated.

7. The hazards to be encountered while performing future maintenance on the lighting equipment should be considered in determining pole locations.

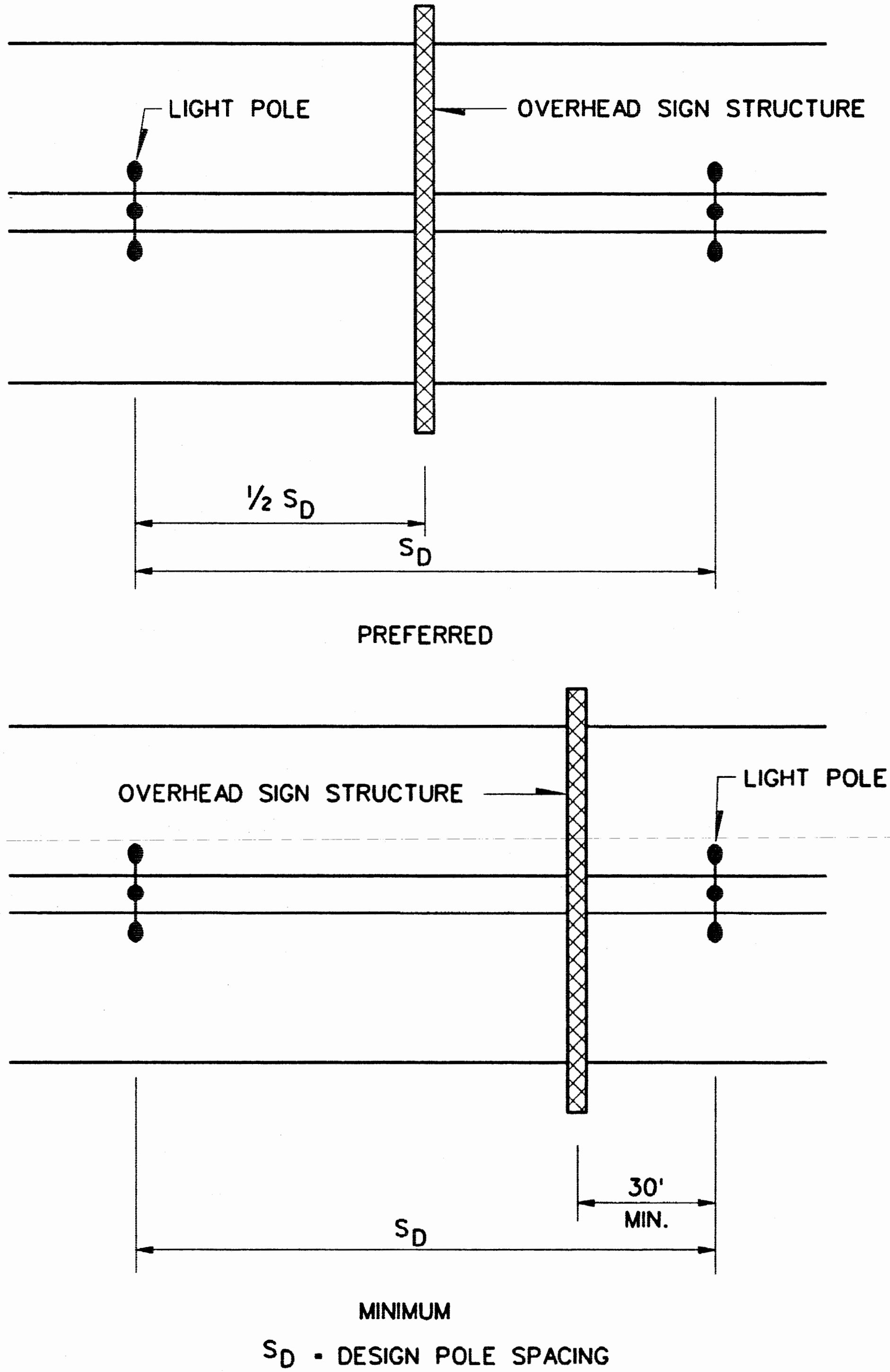
8. Poles should be placed to minimize interference with the driver's view of the roadway or any highway signs.

9. Poles should be placed sufficiently far enough away from overhead bridges or overhead sign structures so that the light from the luminaire will not cast distracting shadows on the roadway surface; detract from the sign's legibility, day or night; or produce unnecessary glare for the motorist. Figure 14.6B illustrates ODOT's criteria for light pole placement



POLE CLEARANCE AT EXIT RAMPS

Figure 14.6



LIGHT POLE PLACEMENT NEAR OVERHEAD SIGN STRUCTURES

Figure 14.6B

next to overhead signs and Figure 14.6C illustrates light pole placement next to bridges.

10. Existing overhead sign lights should be tied into any new lighting system circuits.

11. Trees should be sufficiently pruned so that they do not cause shadows on the roadway surface or reduce the luminaire's efficiency.
12. Poles should be placed behind flexible railings or rigid barriers, if they are present.

13. Poles that are shielded by a guardrail should be offset a distance of at least 4′ to allow the railing to deflect without hitting the pole. If this clearance distance is not available, such as in extreme side slope conditions or if the pole is located within 75′ of the approach end of the flexible railing, a breakaway device should be added.

14. Poles that are shielded by a rigid or non-yielding barrier type will not require a breakaway device unless the pole is located within 75′ of the approach end of the barrier.

15. Poles may be located either on top of or behind retaining walls. Poles mounted on top of retaining walls will require that special consideration be given to the retaining wall design.

16. Poles should not be installed with less than a 3-ft distance from the face of the barrier curb to the centerline of the pole.

17. Poles located in an area where pedestrian traffic exists or is expected should not be mounted on a breakaway device.

18. Poles may be placed in median locations where the width of the median is appropriate or if median barriers are to be used. Normally, the median width should be equal to or greater than the pole mounting height to be acceptable.

19. Twin poles should have the same mast arm lengths.

20. Poles with a breakaway device should have the top of the footing constructed as close to ground level as possible to assure the proper action of the breakaway device and to prevent damage to the foundation or underside of an impacting vehicle. In order for the breakaway device to perform properly, it must be installed in accordance with the latest AASHTO *Breakaway Specifications* and the manufacturer's recommendations, see Section 14.6.3.

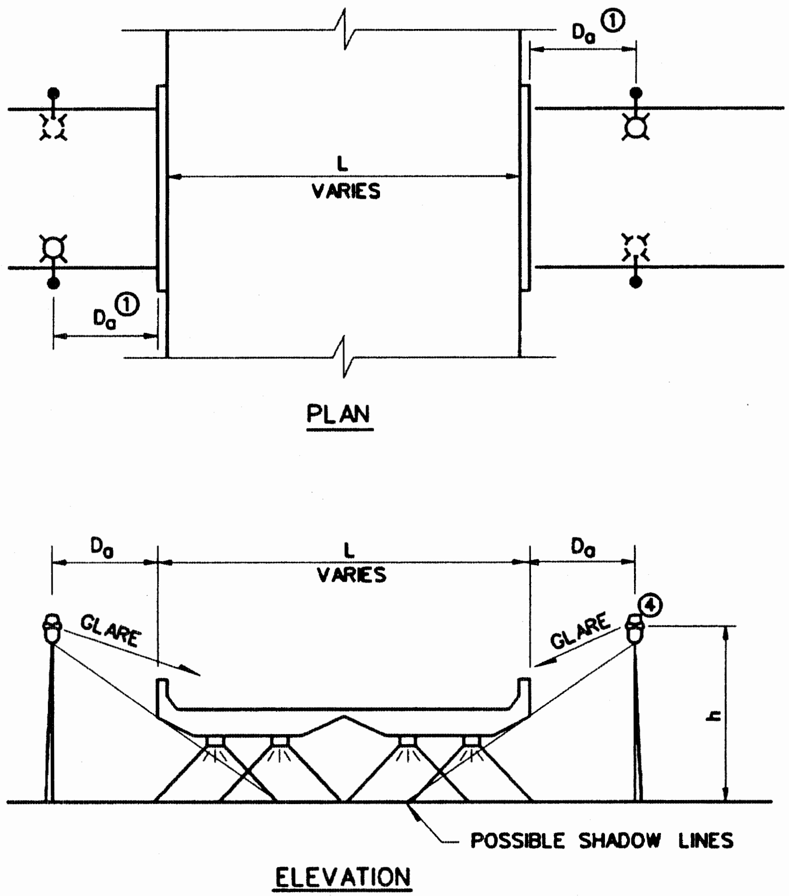
21. Poles, either with or without breakaway devices, should be located in such a manner that they will not interfere with the functional operation of any impact attenuator or other safety breakaway device.

22. The construction of a special feature such as a curb, barrier or other obstacle primarily to protect a light pole will not be allowed, unless approved by ODOT prior to its construction.

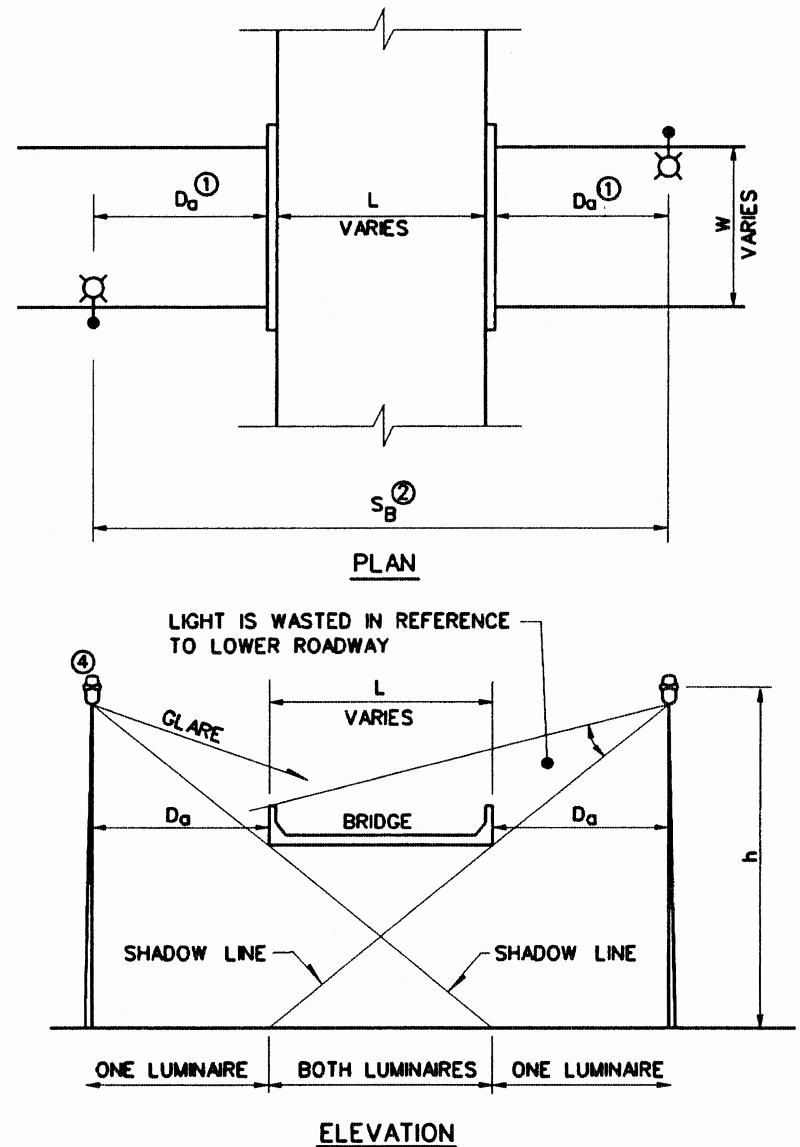
23. Where practical, consideration should be given to providing transitional lighting by using decreasing or lower wattage luminaires.

24. Unprotected high mast towers should be at least 50′ from the edge of the travel lanes or the clear zone distance, whichever is greater, see Figure 14.6D.

25. Access for service vehicles shall be provided for high mast towers and service poles.



LIGHT POLE PLACEMENT FOR WIDE BRIDGES
REQUIRING UNDER STRUCTURE LIGHTS

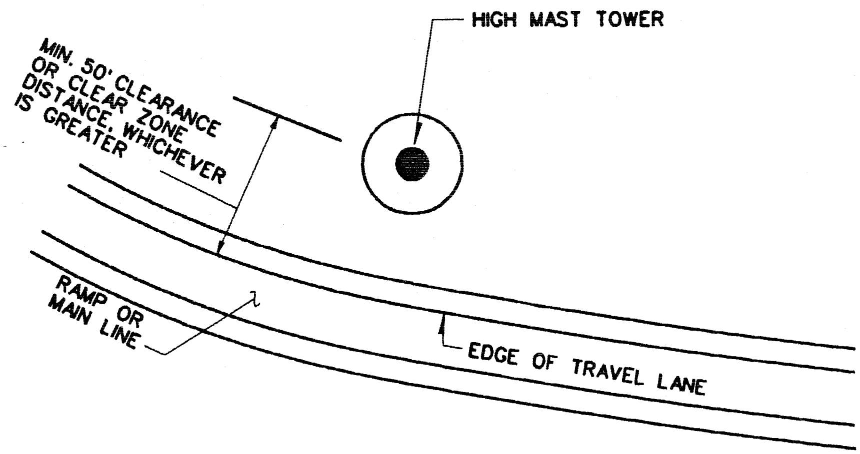


LIGHT POLE PLACEMENT FOR
NARROW BRIDGES & STRUCTURES

Notes:
1. $1h \leq Da \leq 2h$
2. S_B shall not be less than .90 S_D
3. S_D = Design Pole Spacing
4. Glare shields or cutoff type luminaire may be required.

LIGHT POLE PLACEMENT NEAR OVERHEAD BRIDGES

Figure 14.6C



HIGH MAST TOWER PLACEMENT

Figure 14.6D

14.6.5 Lighting Methodologies

There are three lighting design methodologies that are commonly used in the United States — illuminance methodology, luminance methodology and small target visibility methodology. For the most part, the Illuminating Engineering Society of North America (IES) has been the leader in the development of these procedures. The following sections briefly describe each of these three methodologies. For additional information on these procedures, the designer is encouraged to review the references listed in Section 14.6.1.1.

ODOT's policy is to use a combination of the luminance and illuminance methodologies in its design of highway lighting, hereby known as the "luminance-illuminance method." The initial layout design is developed using the conventional illuminance methodology and then analyzed against the luminance criteria. If the luminance criteria are not met, the designer changes the initial parameters (e.g., the light pole spacing, mounting heights) and rechecks to see if the design then meets the luminance criteria. This process is repeated until the design is optimized and the criteria are met. Lighting system designs often require several iterations to produce an acceptable design, see Section 14.6.6.

14.6.5.1 Illuminance

The illuminance methodology is the oldest and simplest to use of the three methodologies. The methodology is concerned with the measurement of the light's intensity striking a particular point on the pavement. The brightest spot will normally occur directly under the luminaire and diminishes the farther the driver is away from the source. The problem with this methodology is that basically one does not "see" incident light; but instead sees the light reflected from an object or surface. This sensation is known as "brightness" with objects distinguished by the difference in brightness or "contrast." Brightness can be expressed mathematically as "luminance"; the luminous intensity per unit area directed towards the eye.

The important factors in the illuminance design are the measurement of average maintained horizontal illumination (E_{avg}), the uniformity ratio of the average illuminance to the minimum illuminance and the ratio of the maximum to minimum illuminance.

14.6.5.2 Luminance

Compared to the illuminance methodology, the luminance design is considered to be a more accurate representation of the driver's visibility requirements. The luminance methodology is concerned with the measurement of light from the luminaire reflecting off the pavement surface to the motorist's eyes. This measurement is affected by the pavement's reflectivity characteristics, see Section 14.6.6. To get the lighting measurements for the roadway, readings are taken from a set of observation points spread across the roadway in a grid pattern.

The design factors in luminance design include average maintained luminance (L_{avg}), minimum luminance (L_{min}), maximum luminance (L_{max}), maximum veiling luminance (L_v) and ratios of L_{avg} to L_{min}, L_{max} to L_{min}, and L_v to L_{avg}. For design purposes, the designer should use the R-3 road surface classification to determine the appropriate luminance values.

14.6.5.3 Small Target Visibility (STV)

IES has proposed the STV methodology in an effort to better define actual visibility requirements of the driver. The STV methodology is similar to the luminance methodology in measurement of the light's reflectivity, but instead of measuring the pavement's reflectivity, it measures a small object's reflectivity against the pavement background. The STV methodology is significantly more complex than the other methodologies and is considered to be impossible to calculate manually, i.e., a computer is required. ODOT does not recommend the use of the STV methodology.

14.6.6 Design Procedures

The following sections provide guidelines on the lighting design procedures used by ODOT. The designer should contact the Traffic Engineering Division for ODOT's latest design procedures and criteria. For additional information, the designer should also review the references listed in Section 14.6.1.1.

14.6.6.1 Computer Design

To determine an acceptable lighting system requires numerous iterations using complex equations. The chance for error calculating these equations manually is very high. Therefore, the designer is encouraged to use one of the several commercial computer software packages that are available.

Each software package generally requires the same input and performs the same calculations. However, the method of input varies significantly. With the proliferation of software programs, the user should check with the Traffic Engineering Division to determine which programs are currently acceptable to ODOT. The designer should provide the Traffic Engineering Division with the design data (inputs and reports) in both a hard copy format and on an electronic media.

14.6.6.2 Design Process

The following is a listing of the procedural steps required in the design of a lighting system:

1. Assemble Information. The designer first needs to assemble the necessary information. This includes:

   a. contacting the Traffic Engineering Division for the current design policies and procedures applicable to the project, the latest copies of sample plans, schedules, pay quantities, plan notes and example calculations;

   b. gathering roadway and bridge plans including plans, profiles and special details (e.g., overhead signs) sheets;

   c. determining existing and expected utility locations;

   d. discussing any special considerations with the highway or bridge designer;

   e. conducting field reviews;

   f. talking with local officials;

   g. reviewing local maintenance capabilities; and

   h. gathering luminaire equipment specifications from manufacturers.

2. Determine Classifications. The designer needs to determine the roadway classification, area classification, existing and expected future pavement types, and environmental conditions. If not already included in the project report, this information can be obtained from the Planning Division.

3. Select Design Criteria. Based on the above information, the designer needs to select the pertinent design criteria, see Sections 14.6.6.3 and 14.6.6.5. For most roadway lighting projects, ODOT's luminance-illuminance design method shall be used.

4. Select Equipment. In the preliminary design, the designer needs to make some initial assumptions on the equipment composition. This includes mounting heights, pole setback distances, light sources, mast arm lengths, lamp wattage, etc. See Sections 14.6.3 and 14.6.6 for additional details. After selecting the luminaire equipment, the designer also needs to obtain the photometric data sheet from the manufacturer for the luminaire(s) selected.

5. Determine Layout Arrangements. Section 14.6.6.3 provides information on the commonly used lighting arrangements. Selection of the appropriate layout design depends upon local site conditions and the engineer's judgment.

6. Preliminary Calculations. The illuminance methodology should be used for the initial calculations. Experience has shown that if an arrangement does not meet the illuminance criteria, it will not meet the luminance criteria. These calculations should be compared to the criteria selected in Step 3.

7. Luminance Calculation. If the design meets the illumination criteria, it then should be recalculated using the luminance methodology and checked against the luminance criteria selected in Step 3.

8. Select Optimum Design. Since computer recalculations are relatively quick and easy, the designer should try several

alternatives even if one design meets the criteria. There are often several alternatives that will work. Typically, the most cost-effective design will be the one with the highest mounting heights and longest pole spacings. Table 14.6B provides guidance on how changing the luminaire location affects the luminance design criteria.

9. ODOT's Design Approval. The designer will provide and discuss the optimum alternatives with the Traffic Engineering Division in order to expedite the development of the project. Upon approval from ODOT, FHWA, city and the local utility company, the final development of the plans may proceed.

10. Prepare Plans. Once the final design has been selected, the lighting designer will prepare and submit to the Traffic Engineering Division the plan sheets, quantities and notes, cost estimate, voltage drop calculations, circuit schematic layouts, and any special provisions that are required for review.

11. Maintenance Agreement. For most projects, the Traffic Engineering Division will prepare a maintenance agreement with the local officials to have the local government maintain the system after it is installed.

14.6.6.3 Design Considerations

In laying out a lighting system, there are many elements or factors the designer needs to consider. To help the designer in this process, the IES has standardized many of these layout elements. However, not all of these elements are appropriate to Oklahoma. In addition to the following, Table 14.6C provides additional guidance on the design values used by ODOT for lighting designs:

1. Classifications Determinations. In selecting the appropriate design criteria, the designer first needs to determine the highway's functional and area classifications. Definitions for these functional and area classifications are provided in the IES publication, *Roadway Lighting*, RP-8.

2. System Configurations. Figure 14.6E illustrates the typical layout arrangements used by ODOT for its lighting system designs. Before using other layout arrangements, the designer will need to receive prior approval for their use from the Traffic Engineering Division.

3. Light Distribution. In determining the lighting design layout, the designer needs to know the expected light distribution. Section 14.6.6.4 presents information on the various light distribution defined by IES.

4. Mounting Heights. Higher wattage bulbs allow the designer to use higher mounting heights, less luminaires, less support poles and still maintain the lighting quality. In general, higher mounting heights tend to produce the most efficient design. Table 14.6D presents ODOT's typical practice for determining mounting height for various conditions. For practical and aesthetic reasons, the mounting height should be kept at a constant height throughout the system.

5. Luminaire Geometry. Figure 14.6F illustrates the common terms used in defining and designing luminaires (e.g., mounting heights, overhang, rotation).

6. Pavement Classifications. To calculate the luminance design values, the designer needs to know the pavement's reflectivity. Table 14.6E presents the roadway surface classifications developed by the

Table 14.6B

EFFECT OF LUMINAIRE LOCATION CHANGE

	L_{avg}	L_{min}	L_{max}	L_v
Decrease Spacing	II	II	I	I
Increase Overhang	II	II	—	—
Decrease Mounting Height	I	DD	II	I
Increase Lamp Size	I	I	I	I
CO to SCO Classification	D	—	—	I
SCO to NCO Classification	D	I	—	I
Short to Medium Classification	D	II	D	I
Medium to Long Classification	D	I	D	I
Staggered to Opposite Arrangement	—	D	—	D

Source (19)

I	Increase in absolute value.
D	Decrease in absolute value.
II or DD	Faster increase or decrease than single letter.
—	Little or no change with moderate variation.
SCO	Semi-Cutoff
NCO	Non-Cutoff
CO	Cutoff

See Section 14.6.6 for classification definitions.

Table 14.6C

ODOT LIGHTING DESIGN PARAMETERS

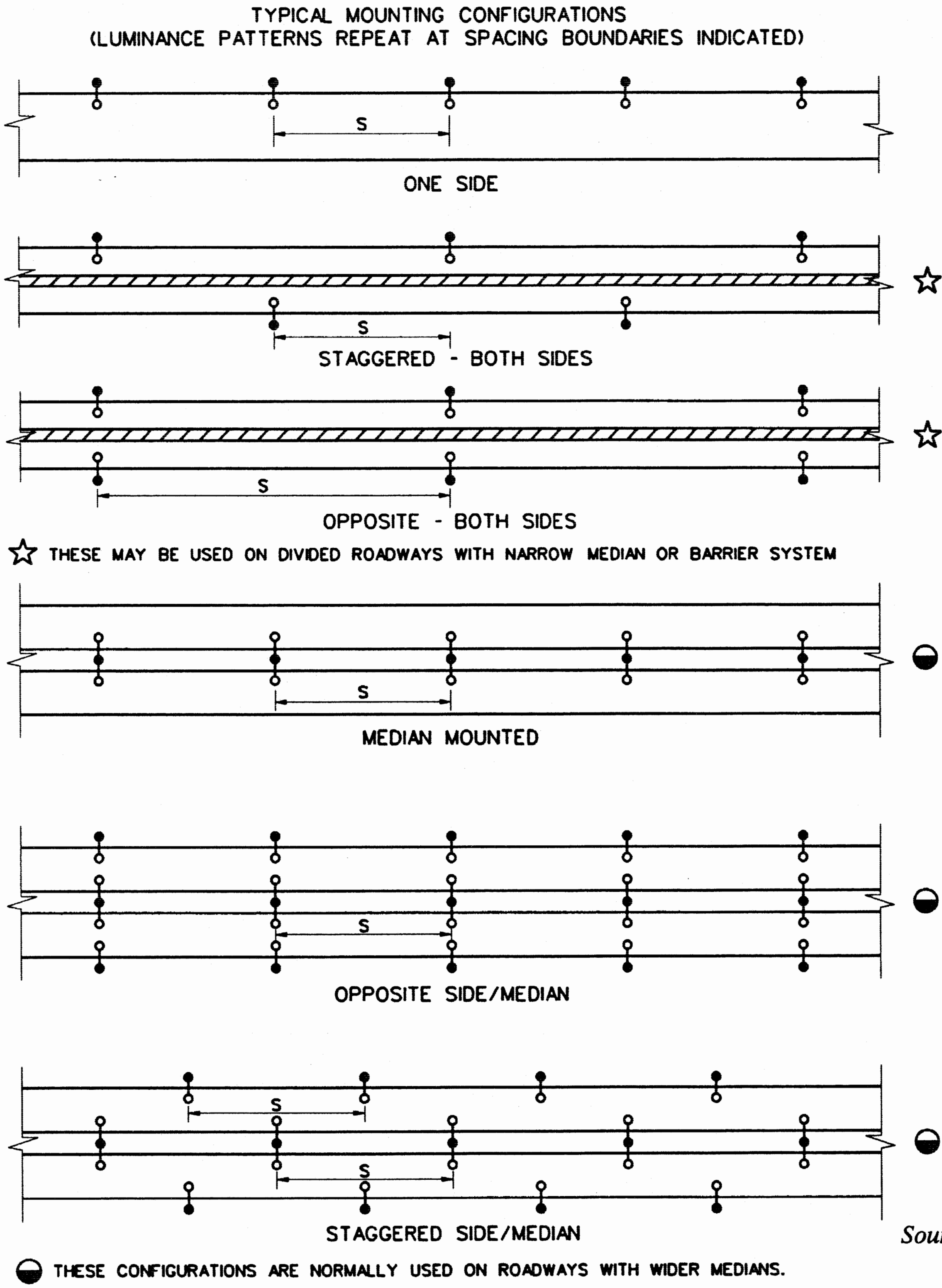
DESIGN PARAMETERS	
Maintenance Factor	.65
Percent of Voltage Drop Allowed	5%
Pavement Design Classification	R3
Power Lines Clearance	10 ft (min)

Table 14.6D

ODOT MOUNTING HEIGHT PRACTICES

FREEWAYS		
Mainlines	40′ - 50′ Mounting Height	15′ Mast Arms
Ramps	40′ Mounting Height	12′ Mast Arms
MOUNTING HEIGHT - LUMINAIRES		
40′ Mounting Height	200w/250w HPS	
45′ Mounting Height	310w HPS	
50′ Mounting Height	400w HPS	
Underpass 15′ Mounting Height	100w HPS	
Overhead Signs	175w M.V.	
HIGH MAST TOWERS		
Mounting Heights	Luminaires 1000w HPS[1]	
100′ - 110′	3-4	
115′ - 130′	4-5	
135′ - 140′	5-6	
145′ - 150′	6-8	

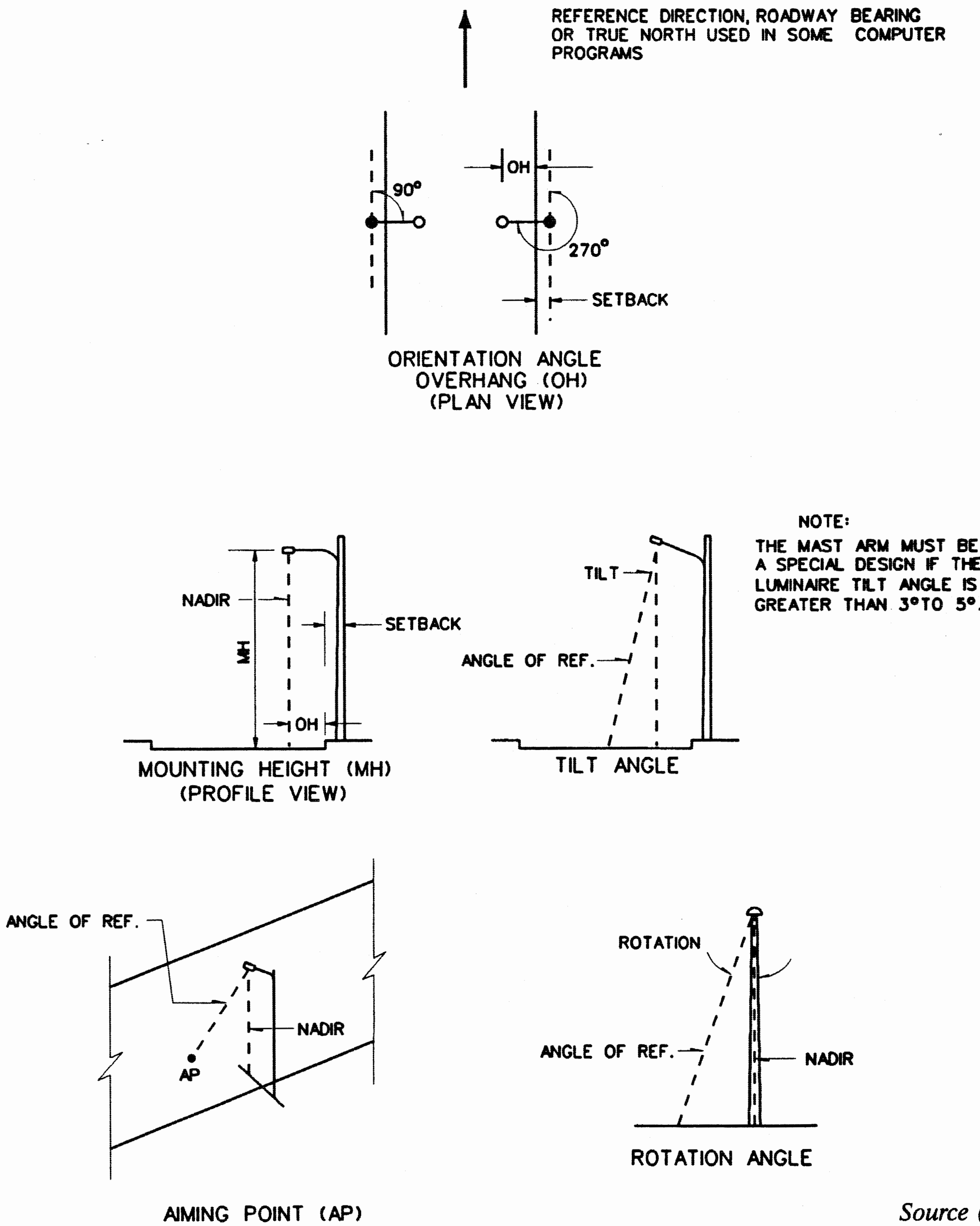
[1] Typical maximum of 12.



Source (19)

LIGHTING SYSTEM CONFIGURATIONS

Figure 14.6E



Source (19)

LUMINAIRE GEOMETRY

Figure 14.6F

Table 14.6E

ROAD SURFACE CLASSIFICATIONS

Class	Q_o	Description	Mode of Reflectance
R-1	0.10	Portland cement concrete road surface. Asphalt road surface with a minimum of 15% of the aggregates composed of artificial brightener (e.g., Syncopal) aggregates (e.g., labradorite, quartzite).	Mostly Diffuse
R-2	0.07	Asphalt road surface with an aggregate composed of a minimum 60% gravel (size greater than 10 millimeters). and Asphalt road surface with 10 to 15% artificial brightener in the aggregate mix. (Not normally used in North America.)	Mixed Diffuse and Specular
R-3	0.07	Asphalt road surface (regular and carpet seal) with dark aggregates (e.g., trap rock, blast furnace slag); rough texture after some months of use (typical highways).	Slightly Specular
R-4	0.08	Asphalt road surface with very smooth texture	Mostly Specular

Q_o = representative mean luminance coefficient.

Source (19,20)

Commission Internationale d'Eclairage (CIE). For design purposes, lighting systems on Oklahoma highways should be based on the R-3 pavement classification.

7. Photometric Data. Designers may obtain photometric data from luminaire manufacturers. This information is required by ODOT to review the proposed lighting design, unless otherwise specified by the Traffic Engineering Division.

8. Maintenance Factor (MF). As time passes, the efficiency of the luminaire is reduced. The designer needs to estimate this reduction to properly estimate the light available at the end of maintenance life of the lamp. The maintenance factor may range from 0.50 - 0.90, with the typical range between 0.65 - 0.75. Table 14.6C presents the typical maintenance factor used by ODOT for designing lighting systems. The maintenance factor is a combination of the following factors:

   a. Equipment Factor (EF). To compensate for the normal production tolerances of commercially available luminaires, it is common practice to estimate the equipment loss at 5 to 10 percent (i.e., EF = .95 - .90). This information should be provided by the manufacturer.

   b. Lamp Lumen Depreciation Factor (LLD). As the lamp progresses through its service life, the lumen output of the lamp decreases. This is an inherent characteristic of all lamps. The initial lamp lumen value is factored by a lumen depreciation factor to compensate for the anticipated lumen reduction. This assures that a minimum level of illumination will be available at the end of the assumed lamp life, even though lamp lumen depreciation has occurred. This information should be provided by the manufacturer.

   c. Luminaire Dirt Depreciation Factor (LDD). Dirt on the exterior and interior of the luminaire, and to some extent on the lamp, reduces the amount of light reaching the roadway. Various degrees of dirt accumulation may be anticipated depending upon the area in which the luminaire is located. Industry; exhaust of vehicles, especially large diesel trucks; dust; etc., all combine to produce the dirt accumulation on the luminaire. Higher mounting heights, however, tend to reduce the vehicle-related dirt accumulations. Information on the relationship between the area and the expected dirt accumulation is discussed in the IES publication, *Roadway Lighting*, RP-8.

14.6.6.4 Light Distribution Considerations

Proper distribution of the light from the luminaire is a major factor in the design of efficient lighting. Table 14.6F presents three IES classifications for luminaire light distributions -- width, spacing and glare control. Table 14.6G provides additional guidance on the selection of luminaires based on these classifications. Figure 14.6G shows a plan view of a roadway which has been modified to show a series of Longitudinal Roadway Lines (LRL) and Transverse Roadway Lines (TRL) and how these distribution factors are interrelated to each other. The following briefly describes these classifications:

1. Vertical Light Distributions. There are three groups of vertical light distributions -- short, medium and long. Selection of a vertical light distribution is dependent

Table 14.6F

LUMINAIRE CLASSIFICATION SYSTEM

Spacing Classification	**Definition**	**Normally Used For Spacings**
S	Short	Up to 4 times Mounting Height
M	Medium	Up to 5 times Mounting Height
L	Long	Over 5 times Mounting Height
Width Classification	**Mounted at Pavement**	**Normally Used For Roadway Widths**
Type I	Center	Up to 2 times the MH in width.
Type II	Edge	Up to 1 times the MH for one side mounting. Up to 2 times the MH for both side mounting.
Type III	Edge	Up to 1.5 times the MH for one side mounting. Up to 3.0 times the MH for both side mounting.
Type IV	Edge	Up to 2 times the MH for one side mounting. Up to 4 times the MH for both side mounting.
Type V	Center	Up to 4 times the MH in total width.
Glare Control Classification	**Definition**	**Normally Used For**
CO	Cutoff	Strict control of light above 80 degrees vertical.
SCO	Semi-cutoff	Medium control of light above 80 degrees vertical.
NCO	Non-cutoff	No control requirements above 80 degrees vertical.

Source (18)

Notes:

1. The complete luminaire classification consists of the three terms in sequence. Example: M-III-SCO

2. There is no assurance that the criteria values will be achieved by a luminaire which meets the classification requirements and is used as shown above.

3. ODOT does not use all of the IES classifications listed above. The designer should review Section 14.6.3.4 or contact the Traffic Engineering Division to determine the luminaire classifications used by ODOT.

Table 14.6G

GUIDE FOR LUMINAIRE LATERAL LIGHT TYPE AND PLACEMENT

One Side or Staggered	Staggered or Opposite	Twin Mast Arm (Median Mounting)	At-Grade Intersections or High Mast
Pavement Width up to 1.5 MH Types II,III,IV	Pavement Width beyond 1.5 MH Types III and IV	Pavement Width up to 1.5 MH (each pavement) Types II and III	Pavement Width up to 2.0 MH Types IV and V

Source (18); Revised

upon the mounting height and light source. Pavement brightness is generally increased when the vertical light angle is increased. The following defines these three vertical light distribution types:

a. Short Distribution. The maximum candlepower strikes the roadway surface between 1 and 2.25 mounting heights from the luminaire. The theoretical maximum spacing, using the short distribution, is 4.5 mounting heights.

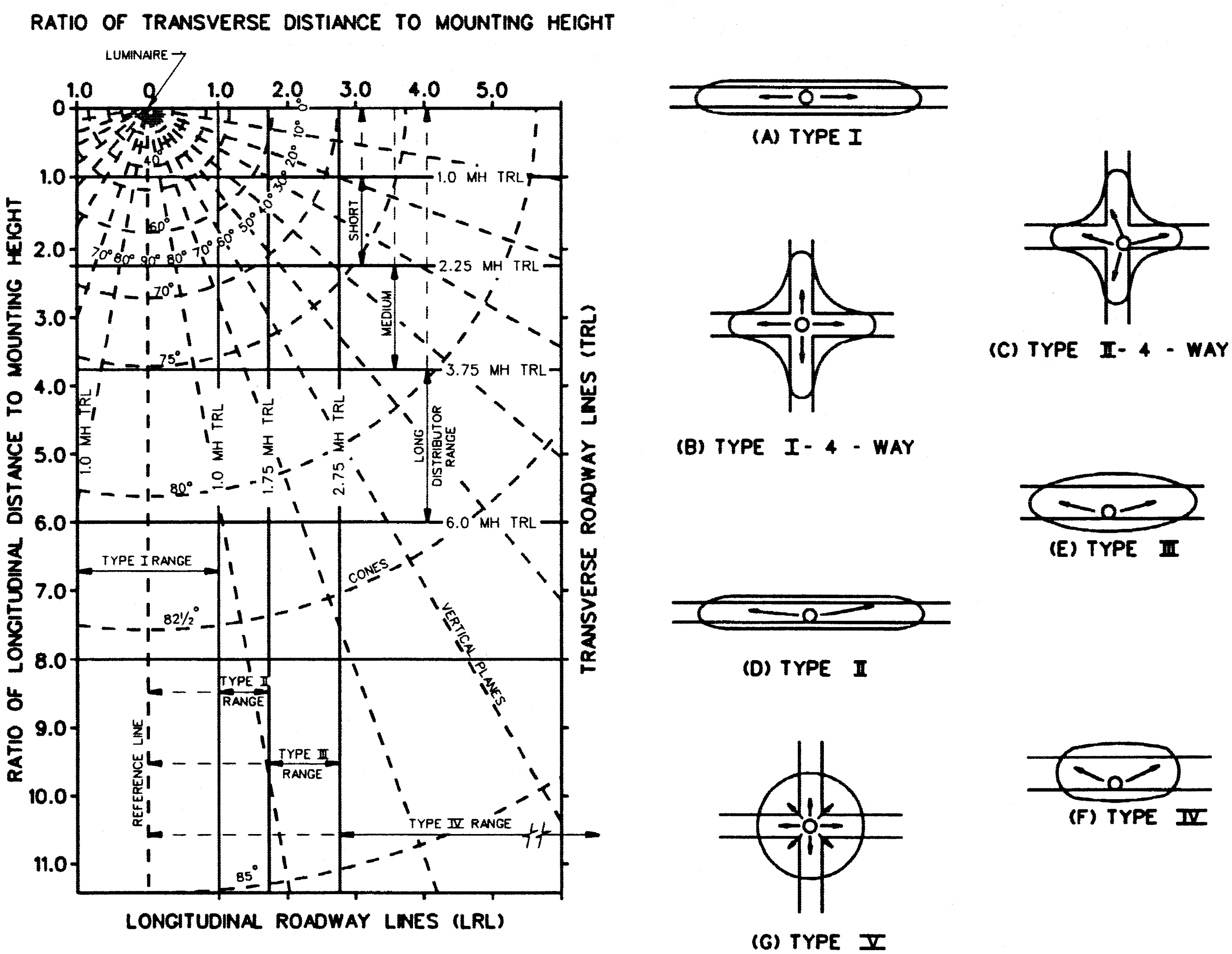
b. Medium Distribution. For medium distribution, the maximum candle-power is between 2.25 and 3.75 mounting heights from the luminaire. The theoretical maximum spacing is 7.5 mounting heights.

c. Long Distribution. For long distributions, the maximum candlepower is between 3.75 and 6.0 mounting heights from the luminaire. The theoretical maximum spacing is 12 mounting heights. This vertical light distribution is typically not utilized by ODOT in its lighting designs.

2. Lateral Light Distributions. The IES has developed seven lateral light distributions which are presented in Figure 14.6F. In addition, the following provides additional information on the placement for these lateral light distributions:

   a. Type I. The luminaire is placed in the center of the street or area where lighting is required. It produces a long-narrow oval-shaped lighted area. Some types of high mast lighting are also considered to be a modified form of Type I. ODOT generally does not use this lateral light distribution in its lighting systems designs.

   b. Type I - 4-Way. The luminaire is placed in the center of the inter-section and distributes the lighting down the four legs of the intersection.



Source (20)

PLAN VIEW FOR LUMINAIRE COVERAGES

Figure 14.6G

ODOT generally does not use this lateral light distribution in its lighting system designs.

c. Type II. The luminaire is placed on the side of the street or edge of the area to be lighted. It produces a long, narrow, oval-shaped lighted area which is usually applicable to narrower streets.

d. Type II - 4-Way. The luminaire is placed at one corner of the intersection and distributes the lighting down the four legs of the intersection. This lateral light distribution is generally not used by ODOT in its lighting system designs.

e. Type III. The luminaire is placed on the side of the street or edge of area to be lighted. It produces an oval-shaped lighted area and is usually applicable to medium width streets.

f. Type IV. The luminaire is placed on the side of the street or edge of area to be lighted. It produces a fatter oval-shaped lighted area and is usually applicable to wide streets.

g. Type V. The luminaire is placed in the center of the street, intersection or area where lighting is required. It produces a circular lighted area. Type V often applies to high mast lighting.

3. Control of Distribution. As vertical light angles increase, discomforting glare also increases. To distinguish the types of glare effects on the driver from the light source, IES has defined these glare effects as follows:

a. Cutoff. The cutoff design is where the luminaire light distribution is less than 25,000 lumens at an angle of 90° above nadir (vertical axis) and 100,000 lumens at a vertical angle of 80° above nadir.

b. Semi-cutoff. For the semi-cutoff design, the candlepower numbers become 50,000 lumens for 90° above nadir and 200,000 lumens at a vertical angle of 80° above nadir.

c. Non-cutoff. This classification is when there is no limitation on the zone above the maximum candlepower.

14.6.6.5 Design Criteria

The lighting criteria varies according to the design methodology, highway classification, area classification and pavement type. Section 14.6.6.3 defines the variables used in the criteria tables. The following tables presents AASHTO's, IES' and ODOT's lighting design criteria:

1. Table 14.6H presents the recommended AASHTO and IES luminance design criteria which is utilized by ODOT.

2. Table 14.6I presents the recommended AASHTO and IES illuminance design criteria which is utilized by ODOT.

3. Table 14.6J presents ODOT's illuminance design criteria for underpasses, ramps, intersections, parking areas, weigh stations, rest areas, frontage roads and interior roadways within rest areas and weigh stations. The criteria presented in Table 14.6J exceeds the recommended AASHTO values.

4. Table 14.6K presents the recommended AASHTO and IES illuminance design criteria for high mast lighting which is utilized by ODOT.

Table 14.6H

LUMINANCE DESIGN CRITERIA

Road Classification	Area Classification	L_{avg} (Footlamberts)	L_{avg} to L_{min}	L_{max} to L_{min}	Maximum Ratio L_v to L_{avg}
Freeway		0.17	3.5:1	6:1	0.3:1
Expressway*	Commercial	0.29	3:1	5:1	0.3:1
	Intermediate	0.23	3:1	5:1	
	Residential	0.17	3.5:1	6:1	
Arterial	Commercial	0.35	3:1	5:1	0.3:1
	Intermediate	0.26	3:1	5:1	
	Residential	0.17	3.5:1	6:1	
Collector	Commercial	0.23	3:1	5:1	0.4:1
	Intermediate	0.17	3.5:1	6:1	
	Residential	0.12	4:1	8:1	
Local	Commercial	0.17	6:1	10:1	0.4:1
	Intermediate	0.15	6:1	10:1	
	Residential	0.09	6:1	10:1	
Alleys	Commercial	0.12	6:1	10:1	0.4:1
	Intermediate	0.09	6:1	10:1	
	Residential	0.06	6:1	10:1	

* Applied to both the main line and ramps. Expressways with full control of access are considered as freeways.

Source (17, 20) Revised

Note: For design of pedestrian walkways and bike paths, use the illuminance criteria shown in Table 14.6L.

Table 14.6I

ILLUMINANCE DESIGN CRITERIA

		Average Maintained Horizontal Illuminance*			
Roadway and Walkway Classification	Area Classification	R1 (Footcandles)	R2 & R3 (Footcandles)	R4 (Footcandles)	Uniformity avg/min
Freeway		0.6	0.8	0.7	3:1
Expressway**	Commercial	0.9	1.3	1.2	
	Intermediate	0.7	1.1	0.9	3:1
	Residential	0.6	0.8	0.7	
Arterials	Commercial	1.1	1.6	1.4	
	Intermediate	0.8	1.2	1.0	3:1
	Residential	0.6	0.8	0.7	
Collector	Commercial	0.7	1.1	0.9	
	Intermediate	0.6	0.8	0.7	4:1
	Residential	0.4	0.6	0.5	
Local	Commercial	0.6	0.8	0.7	
	Intermediate	0.5	0.7	0.6	6:1
	Residential	0.3	0.4	0.4	
Alleys	Commercial	0.4	0.6	0.5	6:1
	Intermediate	0.3	0.4	0.4	
	Residential	0.2	0.3	0.3	
Sidewalks	Commercial	0.9	1.3	1.2	3:1
	Intermediate	0.6	0.8	0.7	4:1
	Residential	0.3	0.4	0.4	6:1

* Average illuminance on the travelled way or on the pavement area between curb lines of curbed roadways. See definition of average maintained illuminance in Section 14.6.1.

** Only applies to the main line. For ramps and rest areas, see Table 14.6.J. Expressways with full control of access are considered to be freeways.

Source (17, 20) Revised

Table 14.6J

ODOT'S MAINTAINED[1] ILLUMINANCE DESIGN CRITERIA

	Minimum Footcandles	Average Footcandles	Uniformity Ratios	
			Average/ Minimum	Maximum/ Minimum
Underpasses	.4	.6	3:1	8:1
Entrance Ramps	.4	.6	3:1	8:1
Exit Ramps	.4	.6	3:1	8:1
Intersections	.4	.7	3:1	8:1
Interior Roadways[2]	.4	.6	3:1	8:1
Parking Areas	.4	1.0	3:1	8:1
Weigh Stations	.4	1.0	3:1	8:1
Externally Illuminated Overhead Signs	2.0	10.0	N/A	6:1[3]
Rest Areas (Major)[4]	.4	1.0	3:1	8:1
Rest Areas (Minor)[4]	.2	.5	6:1	10:1

1 Maintained illuminance indicates the lamp lumen has depreciated to its lowest maintained values.

2 For frontage roads and roadways within rest areas and weigh stations.

3 6:1 maximum, 4:1 preferred.

4 Major and minor areas as defined in AASHTO *An Information Guide for Roadway Lighting*. Rest areas may have both types of activity.

Table 14.6K

RECOMMENDED MAINTAINED ILLUMINANCE DESIGN LEVELS FOR HIGH MAST LIGHTING

(Horizontal Illuminance (E_{avg}) in Footcandles)

Road Classification	Commercial Area	Intermediate Area	Residential Area
Freeways	0.8	0.6	0.6
Expressways	0.9	0.7	0.6
Major	1.1	0.8	0.6
Collector	0.6	0.6	0.6

Source (20) Revised

Note: 1. Recommended uniformity of illumination is 3 to 1 or better; average-to-minimum for all road classifications at the illuminance levels recommended above.

2. These design values apply only to the traveled portions of the roadway. Interchange roadways are treated individually for purposes of uniformity and illuminance level analysis.

5. Table 14.6L presents the recommended IES illuminance design criteria for pedestrian walkways and bikeway paths.

14.6.6.6 Calculations

The designer is referred to the references listed in Section 14.6.1.1 for information on the equations and procedures used to develop the calculations for the illuminance and luminance methodologies. Computer software packages are commercially available that will perform these calculations.

14.6.6.7 Metric Conversions

IES exclusively utilizes the metric system in its lighting publications. For conversion purposes, Tables 14.6M and 14.6N are provided for the designer's convenience.

Table 14.6L

RECOMMENDED AVERAGE MAINTAINED ILLUMINANCE LEVELS FOR PEDESTRIAN WALKWAYS[1]

(Footcandles)

Walkway and Bikeway Classification	Minimum Average Horizontal Levels (E_{avg})	Average Vertical Levels For Special Pedestrian Security (E_{avg})[2]
Sidewalks (roadside) and Type A bikeways:		
Commercial Areas	0.9	2.0
Intermediate Areas	0.6	1.0
Residential Areas	0.2	0.5
Walkways distant from roadways and Type B bikeways:		
Walkways, bikeways, and stairways	0.5	0.5
Pedestrian Tunnels	4.0	5.0

Source (20)

Note: 1. Crosswalks traversing roadways in the middle of long blocks and at street intersections should be provided with additional illumination.

2. For pedestrian identification at a distance. Values are 6 feet above walkway.

Table 14.6M

LUMINANCE CONVERSION FACTORS

Multiply Number of → To Obtain Number of ↓ By	Footlambert	Candela/ square meter	Candela/ square inch	Candela/ square foot	Candela/ square centimeter
Footlambert	1	0.2919	452	3.142	2,919
Candela/square meter	3.426	1	1.550	10.76	10
Candela/square inch	0.00221	0.000645	1	0.00694	6.45
Candela/square foot	0.3183	0.0929	144	1	929
Candela/square centimeter	0.00034	0.0001	0.155	0.00108	1

Table 14.6N

ILLUMINANCE CONVERSION FACTORS

Multiply Number of → To Obtain Number of ↓ By	Footcandles	Lux	Phot
Footcandles	1	0.0929	929
Lux	10.76	1	10,000
Phot	0.00108	0.0001	1
Milliphot	1.076	0.1	1,000

1 lumen = 1/683 light-watt
1 lumen-hour = 60 lumen-minutes
1 footcandle = 1 lumen/square foot
1 watt-second = 10^7 ergs
1 phot = 1 lumen/square centimeter
1 lux = 1 lumen/square meter = 1 metercandle

Source (20)

14.7 MISCELLANEOUS PROGRAMS AND STUDIES

The Traffic Engineering Division is responsible for conducting several traffic engineering programs and studies. The following sections provide a synopsis of what is involved in these programs and studies.

14.7.1 Spot Speed Studies

Spot speed studies are typically conducted by the Traffic Planning and Safety Branch to:

1. determine speed zones,
2. set posted speed limits,
3. select an appropriate design speed,
4. determine no passing zones,
5. determine traffic sign locations,
6. determine traffic signal locations and timing plans,
7. analyze high-accident sites for possible corrective action,
8. provide speed data at high-accident sites for the Hazard Elimination Program data base,
9. investigate public complaints,
10. gather traffic data for research, and
11. calculate road-user costs for economic analyses.

Numerous statistical analyses can be performed on the speed data collected from spot speed studies. ITE's *Manual of Traffic Engineering Studies* and other college engineering text books provide information on conducting a spot speed study and how to statistically analyze the data. The following describes common terms used in spot speed studies:

1. Design Speed. Design speed is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern. The selected design speed should equal or exceed the anticipated posted/regulatory speed limit of the facility after construction.

2. 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 average running speed is the distance summation for all vehicles divided by the running time summation for all vehicles.

3. Average Travel Speed. Average travel speed is the distance summation for all vehicles divided by the total time summation for all vehicles. (Note: Average running speed only includes the time the vehicle is in motion. Therefore, on uninterrupted flow facilities which are not congested, average running speed and average travel speed are equal.)

4. 85th-Percentile Speed. The 85th-percentile speed is the speed below which 85 percent of vehicles travel on a given highway. The most common application of the value is its use as one of the factors, and usually the most important factor, for determining the posted legal speed limit for a highway section. Field measurements for the 85th-percentile speed are to be conducted during off-peak hours when drivers are free to select their desired speed.

5. Spot Speed. Spot speed is the instantaneous measure of speed at a specific location on a roadway.

6. Modal Spot Speed. Modal spot speed is the speed value that occurs most frequently in a sample of speed measurements.

7. Sample Size. Sample size is the minimum number of readings required to get a desired level of confidence.

8. Frequency Distribution. Frequency distribution is a convenient way to condense and summarize data. It can quickly show at what speeds the majority of the drivers are traveling for a given location. It can also be used to quickly compare two or more sample sites.

9. Arithmetic Mean. The arithmetic mean is the most common measure of central tendency. It is determined by summing all the data points and dividing it by the sample size. For spot speed studies, it represents the 50th-percentile driver.

10. Variability. This value represents the spread of the data points. Common measures for variability include range and standard deviation. Spot speed studies with small ranges and standard deviations indicate that a majority of drivers are driving a particular section of highway at relatively the same speed.

11. Reliability. The reliability of a study is determined by the degree of confidence that the sample study represents the population as a whole. Typical confidence levels are 90, 95 or 99 percent.

12. Significance Testing. Significance testing determines if differences between two studies are statistically significant. Significance testing is commonly done in before-and-after studies. An example would be to determine if the increase in speeds after a reconstruction project was due to the reconstruction project or due to chance.

14.7.2 Skid Testing

Skid testing is routinely conducted by the Traffic Planning and Safety Branch as part of ODOT's Pavement Management Program. This data is used to determine projects for inclusion into the pavement resurfacing program. Interstate routes are tested annually, arterials biennially and other routes as necessary.

Skid testing is also done to evaluate wet weather accidents. Test locations are determined by conducting a computer analysis of the accident data bases to find locations with reoccurring wet weather accidents. Testing may also be conducted to check the pavement surface for a particular accident.

Skid testing as performed by ODOT is a measure indexing a worst-case condition. The skid test trailer utilizes a treadless tire in a locked brake condition moving over a wetted pavement.

A skid number of less than 35 is usually the threshold used to alert other Divisions or Sections of the relative pavement performance, either dry or wet weather performance. This data is forwarded, as appropriate, to the Field Division, the Design Divisions and to the Pavement Management Branch for further action.

14.7.3 Videolog

The Traffic Planning and Safety Branch maintains ODOT's videolog of the State highway system. Each direction of the State's 13,000 route miles are covered resulting in 26,000 highway miles on tape. Each route is relogged every 2 to 4 years or when major changes occur (e.g., new interchange). A videolog system can be found in the Central Office as well as in each Field Division Office.

To effectively prepare a set of plans, field reviews are often conducted to evaluate the existing site conditions. The designer should review the videolog prior to the field review to become familiar with the site. This will allow the designer to determine, in advance, the important features to review eliminating wasted time in the field. As the design progresses and memories fade, the videolog system allows the designer to review the site again or determine locations for specific items that may have been missed in the initial review without having to conduct another field trip. The videolog system can be used to determine the condition and locations for:

1. horizontal curves,
2. vertical curves,
3. intersections,
4. bridges,
5. railroad crossings,
6. rivers,
7. signs,
8. signals,
9. guardrail,
10. impact attenuators,
11. hazards or obstacles, and
12. accident locations.

14.7.4 Accident Surveillance System

ODOT has developed and maintains a highway safety improvement program; see Section 14.8. This program exceeds the recommended program outlined by the FHWA. Major components of this program include collecting accident data records and maintaining a surveillance system. The Traffic Planning and Safety Branch has developed a comprehensive computerized data base covering the accident history for all highways and streets in Oklahoma. Accident data are fed into the computer from the "Official Police Traffic Collision Report." The designer may request a copy of this report from the Department of Public Safety.

In determining a cost-effective design, the designer needs to know the accident history record of the highway. The designer should request an accident history report for any project on an existing highway. An example accident history request form is shown in Figure 14.7A. This accident history data base can provide the designer with valuable information including:

1. identification of high-accident locations;
2. documentation for before-and-after studies;
3. information for determining traffic control devices (e.g., signs, signals);
4. information for evaluating different geometric designs;
5. identification of access control problems;
6. establishing priorities for high-accident location improvements;
7. justification for improvements;
8. justification for design exceptions;
9. identification of driver/pedestrian actions causing accidents;
10. wet weather accident analysis;
11. identification of need for roadway lighting; and

OKLAHOMA DEPARTMENT OF TRANSPORTATION

DATE ______

TO: TRAFFIC ENGINEERING DIVISION
TRAFFIC SAFETY AND ACCIDENT STUDIES SECTION

FROM: ______
(DIVISION, DEPARTMENT, OR AGENCY)

SUBJECT: REQUEST FOR ACCIDENT INFORMATION

COUNTY: ______ HIGHWAY NUMBER OR ON-STREET NAME ______

CITY: ______ BEGINNING STREET NAME OR C.S. NO. ______ ENDING STREET NAME OR MILE POINT(S) ______

LOCATION DESCRIPTION: ______

TYPE OF INFORMATION REQUIRED

NORMAL REQUESTS:

- ☐ COMPUTER TABULATION - SHOWS NO. AND TYPE OF ACCIDENT, CAUSE, TIME, CASENUMBER, SEVERITY, ETC.
- ☐ COLLISION DIAGRAM(S) - INTERSECTIONS ONLY
- ☐ COMPUTER PRINTOUT OF EACH INDIVIDUAL ACCIDENT
- ☐ BRIEF ANALYSIS - LOCATION AND CASENUMBERS ONLY
- ☐ NUMBER AND SEVERITY OF ACCIDENTS ONLY
- ☐ CASE NUMBERS OF ACCIDENTS ONLY
- ☐ OTHER - TREND STUDIES, FIXED OBJECTS, ETC.

______

REQUESTS REQUIRING SPECIAL AUTHORIZATION:

- ☐ CONCENTRATION LISTING - CITY-WIDE, COUNTY-WIDE, STATE-WIDE, ETC.
- ☐ ACCIDENT RATE ANALYSIS
- ☐ COST / BENEFIT ANALYSIS - MUST PROVIDE PROJECT COST AND TYPE OF PROJECT IN 'REMARKS'
- ☐ BEFORE AND AFTER STUDY - MUST PROVIDE PROJECT COMPLETION DATE AND COST IN 'REMARKS'
- ☐ HAND-DRAWN SPOT MAP/LISTING
- ☐ INVESTIGATING OFFICERS' REPORTS - HARD COPIES OF REPORTS

YEARS OF HISTORY NEEDED: 1, 2, 3, OTHER ______
(THREE YEARS IS THE STANDARD RUN)

SPECIFY TIME FRAME DESIRED: ☐ MOST CURRENT AVAILABLE OR ______ TO ______

PURPOSE NEEDED/REMARKS ______

REQUESTED BY ______
(SIGNATURE)

DATE REQUIRED ______
OR PRIORITY: ☐ ROUTINE ☐ ASAP ☐ RUSH

FOR TRAFFIC SAFETY AND ACCIDENT STUDIES SECTION USE ONLY

ASSIGNED TO: ______ DATE RECEIVED ______ DATE COMPLETED ______

CLASS =/NE	______	CLASS =/NE	______	CLASS =/NE	______	CLASS =	______
COUNTY =	______	COUNTY =	______	COUNTY =	______	CNTYCITY =	______
CITY =	______	CITY =	______	CITY =	______	NSROAD =	______
CONTROL =	______	CONTROL =	______	CONTROL =	______	EWROAD =	______
MILEPOST =	______	MILEPOST =	______	MILEPOST =	______	INTREL =	______
SPECIAL =/NE	______	SPECIAL =/NE	______	SPECIAL =/NE	______		

PHONE ☐ IA MAIL ☐ REQUEST FOR ACCIDENT REPORTS FROM DPS MADE ______ RECEIVED ______

INTERSEC ______ SKETCH # ______ STUDYLEN ______ BEGENDJULN ______

REMARKS: ______ STUDY #:

TE-2-13 REV. 01/87

ACCIDENT HISTORY REQUEST FORM

Figure 14.7A

12. justification for parking restriction.

Based on the requested information, a computer generated reply will provide an accident history listing, a collision diagram and accident information on:

13. accident severity (e.g., fatal, injury, property damage);

14. accident type (e.g., rear end, sideswipe);

15. traffic control devices (e.g., malfunctioning signal, no signing);

16. driver intentions (e.g., turning, crossing);

17. traffic conditions (e.g., speed, volumes);

18. environmental conditions (e.g., rain, ice, snow, dry, day, night);

19. time (e.g., hour, day, week, month); and

20. location (e.g., city, county, intersection, route, street).

For more information on ODOT's accident surveillance system, the designer should contact the Traffic Planning and Safety Branch. For information on how to interpret the data, the designer is referred to:

21. ODOT's Highway Safety Improvement Program, Section 14.8;

22. ITE's *Manual of Traffic Engineering Studies*;

23. ITE's *Transportation and Traffic Engineering Handbook*; and

24. FHWA's *Manual on Identification, Analysis and Correction of High-Accident Locations*.

14.8 HIGHWAY SAFETY IMPROVEMENT PROGRAM

The Highway Safety Improvement Program (HSIP) is a special Federal monetary category set aside to fund the elimination of localized hazards on existing highways. A copy of ODOT's HSIP can be obtained from the Traffic Engineering Division. The following sections provide brief synopses on the two subprograms under the HSIP.

14.8.1 Hazard Elimination Program

The purpose of the Hazard Elimination Program is to identify, prioritize and correct known or potential high-accident locations. ODOT's Accident Surveillance System, as discussed in Section 14.7, is used to identify these locations. ODOT's HSIP documentation presents ODOT's procedures for prioritizing these accident locations. This prioritization takes into account accident severities, accident reductions, funding categories, economic benefits, construction costs, etc. Once the priority list has been developed and, as funds become available, projects are initiated. ODOT's Roadside Safety Upgrade Program, Bridge Rail Retrofit Program and Delineation Program are all a part of the Hazard Elimination Program. Hazard elimination projects may include:

1. upgrading guardrail,
2. flattening slopes,
3. upgrading horizontal or vertical alignment,
4. installing pavement markings,
5. improving signing,
6. upgrading bridge rails,
7. realigning intersections,
8. installing turn lanes,
9. upgrading or installing signals,
10. installing delineation, and/or
11. roadway lighting.

14.8.2 Railroad/Grade Crossing Program

The Railroad Safety Section in the Traffic Engineering Division is responsible for the implementation of ODOT's At-Grade Railroad Crossing Program. This program provides guidelines for evaluation of at-grade railroad crossings throughout the State to determine what, if any, corrective action needs to be done to improve the safety of the crossings. ODOT's procedures for prioritizing the various crossings in need of improvement are based on established formulas which use as input parameters exposure, existing warning systems, accident history, etc.

After it has been determined that a crossing warrants further consideration, it is given to the designer to prepare the engineering studies and designs. The designer should consider the following:

1. Identify Alternatives. First the designer should identify the available alternatives. Available alternatives could include do nothing, upgrade the crossing, close the crossing, or build a bridge over the crossing. The designer should always first try to reduce the number of crossings. Before choosing an alternative, the designer needs to work with the railroad company and local officials to determine a solution which is satisfactory to everyone involved.

2. Traffic Control Devices. If the crossing is to remain as an at-grade crossing, the designer needs to determine the appropriate traffic control devices for the crossing. These include passive controls, flashing signal lights, gates, bells, etc. The MUTCD provides information on the selection, placement, installation and operation of these devices.

3. Geometrics. The designer needs to review the adequacy of the existing

horizontal and vertical alignment at the crossing. Desirably, the highway will cross the railroad at a right angle on a nearly level roadway. Chapters Six and Seven present ODOT's horizontal and vertical alignment criteria. The designer is referred to the AASHTO *Policy on Geometric Design of Highways and Streets* for information on sight distance requirements at railroad grade crossings.

4. Crossing Surface. The selection, installation and maintenance of the railroad crossing surfaces is jointly the responsibility of ODOT and the railroad company in accordance with current ODOT practice.

5. Railroad Coordination. All decisions involving railroads should be coordinated with the railroad companies, appropriate ODOT Divisions and Sections and other Federal or State agencies having regulatory or permit authority over this work.

14.9 MAINTENANCE AND PROTECTION OF TRAFFIC THROUGH CONSTRUCTION ZONES

Traveling through a construction work zone can be difficult and confusing to unfamiliar drivers. A well-designed traffic control plan with strategically placed warning signs, pavement markings, channelization devices, etc. can alleviate many of these difficulties and confusions.

14.9.1 Traffic Control Plans (TCP)

14.9.1.1 General

Highway construction will almost always disrupt normal traffic operations; therefore, ODOT requires every project to address, in some manner, traffic control through construction zones. With much of ODOT's highway program involved in upgrading existing highways, a well-conceived traffic control plan is essential. The TCP should minimize the operational and safety problems through the construction zone.

The Traffic Operations Branch is responsible for preparing the TCP for all in-house projects. It is also responsible for reviewing and approving the TCP prepared by consultants. On most in-house projects, the roadway designer should request the Traffic Operations Branch to begin preparation of the TCP as early as possible during the plan preparation. The roadway designer should also include the proposed construction sequence with the request. For consultant-designed projects, the Traffic Operations Branch should be provided with their first review opportunity when the construction plans are 50 to 75 percent complete. The consultant should also include the proposed construction sequence. On large projects the Traffic Operations Branch should be involved much earlier in the design process; i.e., at the planning stage.

14.9.1.2 Evaluations

The objective of the TCP is to provide a strategy that will efficiently and safely move traffic through or around the construction zone. To accomplish this strategy, the designer should evaluate the following when preparing a TCP:

1. Engineering. Some of the engineering aspects to consider include:

   a. Highway Capacity. The TCP should provide adequate capacity to handle the expected traffic volumes through the work zone or detour. This may require converting shoulders to travel lanes, eliminating on-street parking, constructing temporary lanes, opening additional lanes during peak periods, constructing HOV lanes, or providing public transportation.

   b. Geometrics. The TCP should have suitable geometry so that a driver can safely maneuver through the work zone, day or night. Section 14.9.4 presents the geometric design criteria for construction zones.

   c. Roadside Safety. The TCP needs to provide a reasonably safe environment both for the traveling public and construction workers. Section 14.9.3 addresses roadside safety concerns through construction zones.

   d. Traffic Control Devices. Appropriate traffic control devices need to be included in the TCP to safely direct vehicles through the construction zone or detour. Section 14.9.2 provides guidance on the selection of traffic control devices.

e. Constructability. The roadway designer needs to evaluate the proposed construction sequence to determine if the project can be constructed with the proposed TCP.

2. Economic. The designer needs to consider the additional economic costs a TCP can have on users and adjacent businesses including extra driving time; wasted fuel; wear and tear on vehicles; increased air pollution; restricted patron access to restaurants, gas stations, stores and other attractions; restricted access for employees and shipments to manufacturing companies; etc. The designer should review the TCP to make sure that it does not restrict access to businesses during peak retail periods. For example, road closures should not be timed to occur during peak seasonal shopping periods (e.g., from Thanksgiving to Christmas).

3. Community. The TCP needs to consider the impacts on neighborhoods, parks, schools, etc. Detours may significantly increase traffic through a community to a point that local traffic can no longer cross the detour route. The designer needs to consider how the TCP will affect fire, ambulance, police and school bus routes. Response times for emergency vehicles should not be noticeably reduced.

4. Social/Political. The designer needs to consider the concerns of local governments, agencies, politicians, or organizations (e.g., homeowner groups). If reasonable, changes should be made to the TCP that will address their concerns. Working with local officials or organizations early in project development can significantly reduce the animosity or opposition to a project.

14.9.1.3 Content

The size and project type greatly affect the amount of information required in the TCP. For example, on signing projects the TCP may only be a listing of the appropriate Standard Drawings and on freeway reconstruction projects it could contain full-scale drawings, special details, special provisions, special task forces, etc. The TCP content will be determined on a project-by-project basis. The TCP may include:

1. Construction Plan Sheets. Reconstruction projects will typically require very detailed plans for handling traffic at each construction stage. These plans may include geometric layout details, positive protection schemes and/or placement for traffic control devices. Smaller projects such as resurfacing, signing, signal, or spot improvements projects will rarely require this type of detail.

2. Pay Items (Quantities). A complete listing of the various traffic control devices required to direct traffic through the work zone should be included in the TCP. This may require estimating or calculating the number of barrels, barricades, cones, signs, warning lights, flashing arrow boards, or other devices that may be required to construct the project.

3. Construction Sequence/Time. The roadway designer should develop a construction sequence and an initial estimate of the construction time in calendar days.

4. Work Schedules. The TCP should clearly spell out any restricted work schedules a contractor is required to follow (e.g., no construction work during peak hours).

5. Phone Numbers. The TCP should inform the contractor that he/she is required to provide emergency phone numbers to the Field Engineer. These phone numbers are required in case there is an accident during non-construction time periods (e.g., replacement of sand barrels).

6. Permits. The Department of Public Safety, Size and Weight Section, is responsible for providing oversize or overweight permits.

7. Special Provisions. Special provisions are utilized to explain special procedures, materials, or equipment used in the TCP that are not covered in ODOT's *Standard Specifications for Highway Construction*. Prior to developing a new special provision, the designer should first make sure that it is not already covered in the *Standard Specifications*, Supplemental Specifications, or reoccurring special provisions. Chapter 18 provides information on the requirements for preparing special provisions.

8. Agreements/Legal Releases. An agreement and/or legal release may be required prior to starting construction for ODOT to use local facilities for highway detour purposes.

9. Media. The designer or other ODOT officials should inform newspapers, television stations, radio stations, etc. when there will be major road closures or detours. This will allow users to make an informed decision on when to use alternative routes or transportation.

10. Pedestrians. The TCP needs to address the safe accommodation of pedestrians through the construction zone. In addition, construction should be scheduled around non-peak pedestrian times (e.g., summer construction around schools).

11. Local Businesses/Residents. At least one reasonable access should always be maintained to businesses and residential neighborhoods. The designer should also make sure these individuals are kept informed of any planned street or driveway closures.

12. Inspections. ODOT and the contractor are responsible for routine inspections of the construction zone. This involves day and night inspections of the pavement markings, signs, lighting, traffic patterns, etc.

13. Traffic Control Enforcement. Research has shown that the presence of law enforcement officials at a work zone site significantly reduces traffic speeds though the work zone. However, the automatic inclusion of law enforcement officials on a project is not always prudent or cost effective. On projects with law enforcement, the designer needs to determine the type of enforcement (e.g., state police, local police), number of officials, number of hours present, etc. These calculations should be included as part of the pay quantities, with enforcement times spelled out in the plan notes.

14.9.2 Traffic Control Devices

The proper use of traffic control devices through construction zones is critical to the safety of the traveling public and construction workers. The following provide ODOT's guidelines and policies relative to the placement of traffic control devices along construction zones and detours:

1. MUTCD (Part VI). ODOT's policy is to comply with the criteria and guidelines presented in the MUTCD, except as noted in comment number 3.

2. Changeable Message Signs. ODOT has found the use of changeable message signs on freeways to be very effective and recommends their use prior to the construction zone. The use of changeable message signs on other highways will be determined on a project-by-project basis. The following guidelines should be considered when using changeable message signs:

   a. Display. The display should provide the maximum amount of information that can be read in a quick glance, i.e., no rolling messages. Studies have shown that three-line presentations with two message phases provide the best readability and comprehension. Figure 14.9A illustrates an example of a two-phase message.

   b. Location. The display should be located approximately ¾ of a mile in advance of the construction zone or application (e.g., lane closure).

   c. Traffic Control Devices. The changeable message sign is considered to be a supplement and not a substitute to the proper use of traffic control devices.

   d. Arrow Boards. Changeable message signs are not to be used as an alternative to arrow boards.

3. Flashing Arrow Boards. It is ODOT's practice to follow the criteria and guidelines set forth in the MUTCD on the use of flashing arrows boards, except on freeways. On freeways it is ODOT's policy that flashing arrow boards shall be used for all lane closures.

4. Warning Signs. ODOT follows the criteria set forth in the MUTCD relative to the type and placement of warning signs. However, the designer should not always adhere to the criteria if a better alternative is available. When warning signs are provided, ODOT also recommends the use of advisory speed plates as defined in the MUTCD.

5. Sight Distance. Desirably, decision sight distance should be provided to all locations requiring a driver to make a maneuver change (e.g., lane closures, crossovers, lane shifts). For example, moving the signing pattern to a sag or to the near side of a crest vertical curve can often provide the desirable sight distance.

6. Speed Limits. In determining a posted speed limit through construction zones, the following should be considered:

   a. Desirable. The construction zone posted speed limit should desirably be the posted speed limit existing prior to construction.

   b. Freeways. The freeway posted speed limit may be reduced from the existing speed limit by 10 mph in areas of freeway congestion.

   c. Non-freeways. On non-freeways highways, the speed limit may be reduced by 10 mph to meet restrictive conditions. Under special circumstances, the Field Engineer may further lower the construction zone speed limit (e.g., accidents).

   d. Design Speed. The construction zone posted speed limit should not exceed the design speed of the construction zone, see Section 14.9.4.

6. Channelization Devices. The need for channelization devices is determined on a case-by-case basis. The following provides

Phase I

Phase II

EXAMPLE OF CHANGEABLE MESSAGE SIGN

Figure 14.9A

a list of the channelization devices typically used by ODOT:

a. barrels and drums,
b. barricades,
c. cones,
d. tube channelizers,
e. longitudinal pavement markings, and
f. portable concrete median barrier.

The Standard Drawings provide additional information on the selection and placement of these channelization devices.

7. Pavement Markings. The pavement marking selection through construction zones depends upon the project's duration, length and traffic volumes. For traffic patterns lasting a year or more, the use of permanent pavement markings should be considered. Section 14.4 provides information on permanent pavement markings. For temporary pavement markings the following should be considered:

    a. Temporary Paint. Due to their low cost, quick-drying paints are the preferred temporary pavement marking choice on most construction projects. To improve the paint stripe's reflectivity, glass beads are to be added to the temporary paint markings. ODOT does not normally allow the use of temporary paint markings on the final pavement surface.

    b. Raised Temporary Pavement Markings. In high-volume locations, or in other locations as deemed necessary, the use of raised temporary pavement markings may be used as a supplement to other markings or by themselves to improve delineation through the construction zone.

    c. Temporary Preformed Plastic Tapes. Temporary preformed plastic tapes are an excellent material choice when there are changes made to the traffic patterns during construction (e.g., crossover switches). These markings can be easily and quickly installed and if necessary, easily removed. A disadvantage is that they tend to move or breakup under heavy traffic volumes. Therefore, routine inspections are required to check their serviceability.

    d. Thermoplastic Markings. Thermoplastic markings are generally only used in construction zones if the traffic volumes are large and the traffic pattern will be in place for a long duration (e.g., over a year).

8. Temporary Traffic Signals. The use of temporary traffic signals in construction zones will be determined on a site-by-site basis. Common locations for temporary traffic signals include intersections where an existing traffic signal is moved or relocated, at temporary access sites, or at one-lane bridges where there is alternating traffic.

9. Temporary Lighting. The designer should consider the use of temporary lighting along highways that have high traffic volumes, high speeds, heavy congestion and/or require complicated maneuvers (e.g., freeway crossovers). In addition, if existing lighting is removed due to construction, temporary lighting should be provided until the permanent lighting is replaced. Section 14.6 provides additional information on highway lighting.

14.9.3 Roadside Safety

As drivers traverse through a construction zone, they are often exposed to numerous hazards including restrictive geometrics, construction equipment and two-way traffic. Elimination of these hazards is often impossible or impractical. Regardless, consideration needs to be given to reducing their exposure.

14.9.3.1 Positive Protection

During the planning and design of a project, careful consideration should be given to traffic control plan alternatives which do not require the use of temporary barriers. This can often be accomplished by using detours, constructing temporary roadways, minimizing exposure time and maximizing the separation between traffic and workers. Even with proper project planning and design there will still be many instances where positive protection should be considered.

Since each site should be designed individually, ODOT has not developed a list of specific warrants for providing positive protection in construction zones. The following provides a list of factors that should be considered:

1. duration of construction activity,
2. traffic volumes,
3. nature of hazard,
4. design speed,
5. highway functional class,
6. proximity between traffic and construction workers,
7. proximity between traffic and construction equipment,
8. adverse geometrics which may increase the likelihood of run-off-the-road vehicles,
9. two-way traffic on one barrel of a divided highway,
10. transition areas at crossovers, and/or
11. lane closures or lane transitions.

14.9.3.2 Two-Way Traffic on Divided Highways

ODOT does not have a specific practice or policy for when to use two-way traffic on a single barrel of a divided highway. This decision should be made on a project-by-project basis. In making this decision, the following factors should be considered:

1. alternate detour routes,
2. traffic characteristics,
3. shoulder widths,
4. room for temporary lanes in median,
5. stage construction (e.g., one-lane at a time),
6. project length,
7. construction efficiency, and
8. height and width restrictions.

Studies have found that the optimum segment length of two-way traffic on divided highways is less than 3.5 miles. When segments exceed 3.5 to 5 miles, operational efficiency is often reduced as traffic backs up behind slower vehicles.

Due to the complex maneuvers required by drivers at crossovers, it is ODOT's policy to use portable concrete median barriers (PCMB) through the crossover; see *ODOT Standard Drawings*. In addition, PCMB should also be used the entire length of the project if the project length is two miles or less and the average daily traffic (ADT) is 10,000 vehicles or greater. When a project is longer than two miles, PCMB will normally only be used in the crossovers.

It is also ODOT's policy to install portable modular glare screens on top of the PCMB through the crossover. The use of glare screens in other locations will be determined on a case-by-case basis. Section 14.9.3.3 provides additional guidance on use of glare screens.

The designer also needs to consider the effect that directing traffic onto the opposing roadway will have on the roadside appurtenances. For example, trailing ends of bridge parapets may require approach guardrail transitions or impact attenuators, or blunt guardrail terminals may need to be converted to an acceptable treatment.

14.9.3.3 Appurtenances Types

The designer's first objective is to design a traffic control plan that eliminates the need for temporary barriers. However, this is often not practical. In addition to Chapter Eleven and the *ODOT Standard Drawings*, the following provides general information on the roadside safety appurtenances used by ODOT through construction zones:

1. Guardrail/Median Barrier. For most construction projects the installation of a new temporary guardrail/median barrier is usually not cost effective due to the short project life. Temporary guardrail/median barrier installations are to meet the permanent installation criteria set forth in Chapter Eleven and the *ODOT Standard Drawings*, expect where modified in Section 14.9.3.5.

2. Portable Concrete Median Barrier. PCMB provides the greatest protection to the work zone and between two-way traffic, but it is also the least forgiving to the driver. To eliminate head-on accidents at crossovers, ODOT requires PCMB be used at all crossovers on two-lane divided highways; see *ODOT Standard Drawings*. For other locations, the decision on when to use PCMB in construction zones will be determined on a site-by-site basis.

3. Glare Screens. Portable modular glare screens should be installed on top of PCMB through crossovers on divided highways. In addition to crossovers, glare screens should be considered where:

   a. the travel lane is within 2 feet of the PCMB,

   b. a high amount of side ambient light exists,

   c. there is a high volume of truck traffic, and/or

   d. the vertical or horizontal alignment of the roadway creates an apparent headlight glare problem.

   If glare screens are used on curvilinear alignments, the designer should ensure that the glare screen installation will not produce subminimal stopping sight distances.

4. End Treatments. Even when protected or otherwise mitigated, the ends are the most hazardous element of any barrier system. Therefore, the unprotected

terminal ends for guardrail and PCMB should be located as far as practical from the roadway or be protected with an appropriate end treatment. This includes breaks in the barrier for crossovers and/or contractor access openings. Chapter Eleven provides information on various end treatments used by ODOT. The safest end treatment should be provided consistent with cost effectiveness and geometric considerations.

4. Sand Barrels. Sand barrels are commonly used to protect the driver from point obstacles (e.g., bridge priers, PCMB ends). Due to the size of the array, sand barrels should only be used outside of the travelway (e.g., shoulders, medians). Arrays vary according to the obstacle width and the design speed. The *ODOT Standard Drawings* illustrate typical sand barrel applications; however, the designer needs to confirm that the *Standard Drawings* are applicable to his/her site. Information on alternative array arrangements can be obtained from the Traffic Engineering Division or from the manufacturers' literature. The designer should also note that no single row arrays are allowed.

5. GREAT_{CZ}. The construction zone GREAT_{CZ} is commonly used to protect the blunt end of the PCMB when the PCMB cannot be properly flared away from the oncoming traffic. It is also used where space does not allow the use of sand barrels at point obstacles (e.g., bridge piers). The *ODOT Standard Drawings* provide additional details on the GREAT_{CZ}.

6. Truck-Mounted Attenuators. Truck-mounted attenuators are typically used on moving or short term construction projects. The following section provides information on the recommended usages for truck-mounted attenuators.

14.9.3.4 Truck-Mounted Attenuators

The hazardous nature of construction and maintenance work zones on streets and highways has been recognized for many years. The use of traffic control plans, improved signing, channelization and pavement markings, portable barriers, better training of flaggers, arrow panels, changeable message signs, and improved scheduling have all combined to produce safer work zones. However, the intrusion of a vehicle into the work area may present a significant hazard to workers and occupants of the intruding vehicle. To overcome this hazard, truck-mounted attenuators (TMA) may be used. Figure 14.9B provides an illustration of a typical TMA. The following factors should be considered when determining the need for a TMA on a construction project:

1. time of work - peak or off-peak periods,

2. distance of work from edge of pavement,

3. traffic volumes,

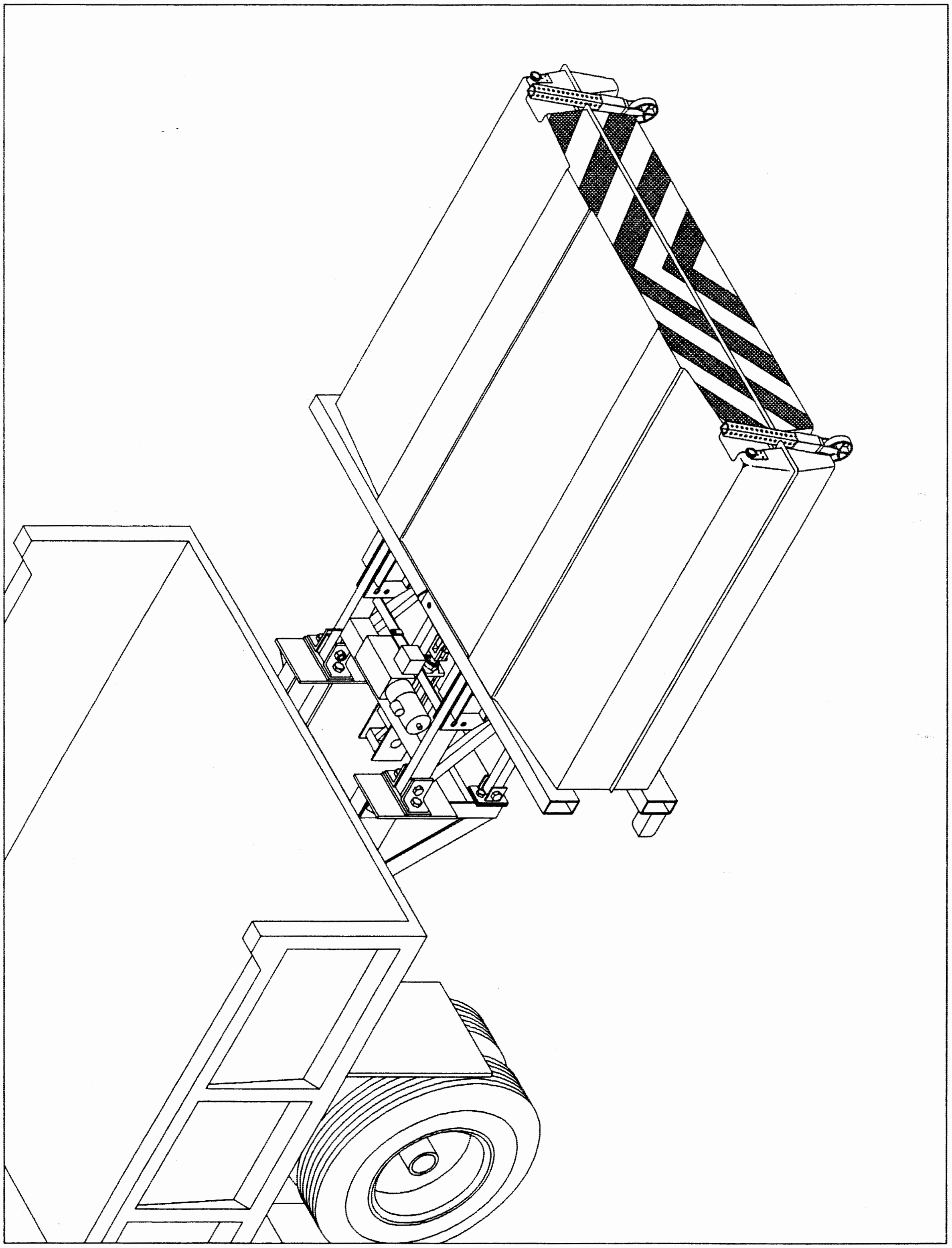
4. moving operations (e.g., painting operations, sweeping),

5. visibility limitations due to restrictive geometrics,

6. when flag persons are anticipated not to be effective,

7. when warning signs are anticipated not to be effective,

8. when flashing arrow boards might create a sudden and unsafe reaction from motorists,



Source (Energy Absorption Systems, Inc.)

EXAMPLE OF TRUCK-MOUNTED ATTENUATOR

Figure 14.9B

9. when cone tapers are anticipated to be inadequate,

10. highway operating speeds,

11. the number of workers to be protected,

12. the need to shield expensive equipment,

13. the functional class of the highway,

14. public reaction (assumed to be positive) to this additional protective device, and

15. improved worker morale.

In addition to the above-listed factors, Table 14.9A provides a listing of example TMA usages. Table 14.9B presents usage guidelines for TMA.

14.9.3.5 Design/Layout

In general, when designing and laying out temporary roadside safety appurtenances in construction zones, the criteria set forth in Chapter Eleven should be followed. However, due to the limited time exposure, it may not always be cost effective to meet the permanent installation criteria. The following provides several alternatives the designer may use in designing and laying out temporary roadside safety appurtenances:

1. Clear Zones. Applying the new or reconstruction clear zone distances, as presented in Chapter Eleven, in construction work zones is often impractical. ODOT has developed revised distances for clear zones through construction zones, which are presented in Table 14.9C. Due to the hazardous conditions which typically exist in construction zones, the designer still must use considerable judgment in applying these clear zone distances.

2. Length of Need. As with new installations, sufficient distance of a full-strength barrier should be provided prior to the hazard to eliminate run-behind accidents. For temporary layouts, the length of need can be determined by using an angle of 15° from the back of the hazard or the clear zone distance off the travelway.

3. Shy Distance. Shy distance recommendations presented in Chapter Eleven are not applicable to temporary barriers in construction zones.

4. Shoulder Widening. When a temporary barrier is placed next to the shoulder, the designer should not provide the extra 2-ft widening.

5. Flare Rates. Desirably, the PCMB terminus should be flared away from the travelway to a point outside of the clear zone. Table 14.9D presents the desirable flare rates for PCMB based on design speed. The designer should provide these flare rates unless under extenuating circumstances it is impossible to do so (e.g., stop conditions, driveways, intersections).

6. End Terminals. Unless the blunt ends of PCMB and guardrail can be sufficiently flared away from oncoming traffic, they shall be protected with an appropriate crash-worthy end terminal or impact attenuator. Chapter Eleven provides additional information on end terminals.

7. Sand Barrels. Sand barrel array designs for temporary installations are the same as for permanent installations, except for the shielding of the hazard. Permanent sand barrel installations require that there be at least 30 inches between the hazard and the outside edge of the sand barrel array. For temporary installation, this

Table 14.9A

EXAMPLES FOR TYPICAL TMA USAGE

CLOSURE/EXPOSURE CONDITION	EXAMPLES OF TYPICAL CONSTRUCTION/MAINTENANCE ACTIVITIES
No Formal Lane Closure	
Shadow Vehicle for Operation Involving Exposed Personnel	Crack pouring, patching, utility work, striping, coning
Shadow Vehicle for Operation Not Involving Exposed Personnel	Sweeping, chemical spraying
No Formal Shoulder Closure	
Shadow Vehicle for Operation Involving Exposed Personnel	Pavement repair, pavement marking, delineator repair
Barrier Vehicle for Operation Not Involving Exposed Personnel	Open excavation, temporarily exposed bridge pier
Formal Lane Closure	
Barrier Vehicle for Operation Involving Exposed Personnel	Pavement repair, pavement marking
Barrier Vehicle for Condition Involving Significant Hazard	Open excavation
Formal Shoulder Closure	
Barrier Vehicle for Operation Involving Exposed Personnel	Pavement repair, pavement marking, guardrail repair
Barrier Vehicle for Condition Involving Significant Hazard	Open excavation

Definitions:

A FORMAL CLOSURE condition (either lane or shoulder) includes a full complement of advance warning devices, a closure taper of channelizing devices, and channelizing devices to define the work area as required.

A NO FORMAL CLOSURE condition (either lane or shoulder) includes limited (if any) advance warning signs and channelizing devices.

A SHADOW VEHICLE is a moving vehicle traveling a short distance upstream from a moving operation giving physical protection from approaching traffic.

A BARRIER VEHICLE is a vehicle parked a short distance upstream from a stationary operation giving protection from approaching traffic.

Source (30)

Table 14.9B

USAGE GUIDELINES FOR SHADOW/BARRIER VEHICLES (TMA)

CLOSURE/EXPOSURE CONDITION	RANKING *			
		NON-FREEWAY WITH SPEED LIMIT		
	FREEWAY	≥50 MPH	40-45 MPH	≤ 35 MPH
No Formal Lane Closure				
Shadow Vehicle for Operation Involving Exposed Personnel	A	A	A	A
Shadow Vehicle for Operation Not Involving Exposed Personnel	E	E	E	E
No Formal Shoulder Closure				
Shadow Vehicle for Operation Involving Exposed Personnel	B	B	C	C
Shadow Vehicle for Operation Not Involving Exposed Personnel	E	E	E	E
Formal Lane Closure				
Barrier Vehicle for Operation Involving Exposed Personnel	B	B	C	D
Barrier Vehicle for Condition Involving Significant Hazard	E	E	E	E
Formal Shoulder Closure				
Barrier Vehicle for Operation Involving Exposed Personnel	C	C	D	D
Barrier Vehicle for Condition Involving Significant Hazard	E	E	E	E

* The ranking letter indicates the priority assigned to the use of a shadow/barrier vehicle. The use of shadow/barrier vehicles:

A is very highly recommended.
B is highly recommended.
C is recommended.
D is desirable.
E may be justified on the basis of special conditions encountered on an individual project when an evaluation of the circumstances indicates that an impact with a shadow/barrier vehicle is likely to result in less serious damage and/or injury than would impact with a working vehicle or the hazard.

Source (30)

Table 14.9C

RECOMMENDED CLEAR ZONE DISTANCE (FT)
(Construction Work Zones)

Design Speed	Design ADT	Fill Slopes			Cut Slopes		
		6:1 or flatter	5:1 to 4:1	3:1	3:1	4:1 to 5:1	6:1 or flatter
40 mph or less	Under 750	4	4	See Procedure in Section 11.2.2.	4	4	4
	750-1500	5	6		5	5	5
	1500-6000	6	7		6	6	6
	Over 6000	7	8		7	7	7
45-50 mph	Under 750	5	6		4	4	5
	750-1500	7	8		5	6	7
	1500-6000	8	10		6	7	8
	Over 6000	10	12		7	9	10
55 mph	Under 750	6	7		4	5	5
	750-1500	8	10		5	7	8
	1500-6000	10	12		7	8	10
	Over 6000	11	13		8	10	11
60 mph	Under 750	8	10		5	6	7
	750-1500	10	13		6	8	10
	1500-6000	13	16*		7	9	12
	Over 6000	15	18*		10	12	13
65-70 mph	Under 750	9	10		5	7	7
	750-1500	12	14		6	9	10
	1500-6000	14	17*		8	11	13
	Over 6000	15	19*		11	13	14

* The clear zone may be limited to 15′ for practicality and to provide a consistent roadway template.

Note:

1. All distances are measured from the edge of the travel lane.
2. See discussion in Section 11.2.2 (Comment #2) for application of clear zone criteria on non-recoverable fill slopes.
3. See discussion in Section 11.2.3 for application of clear zone criteria across ditch sections.
4. For clear zones, the "Design ADT" will be the total ADT on two-way roadways and directional ADT on one-way roadways (e.g., ramps and one roadway of a divided highway).

Source (8) Revised

Table 14.9D

FLARE RATES FOR PORTABLE CONCRETE MEDIAN BARRIER

DESIGN SPEED	FLARE RATES
40 mph or less	9 to 1
50 mph	11 to 1
60 mph or greater	13 to 1

Source (8)

distance can be reduced to 15 inches, see Figure 14.9C.

14.9.4 Geometric Design

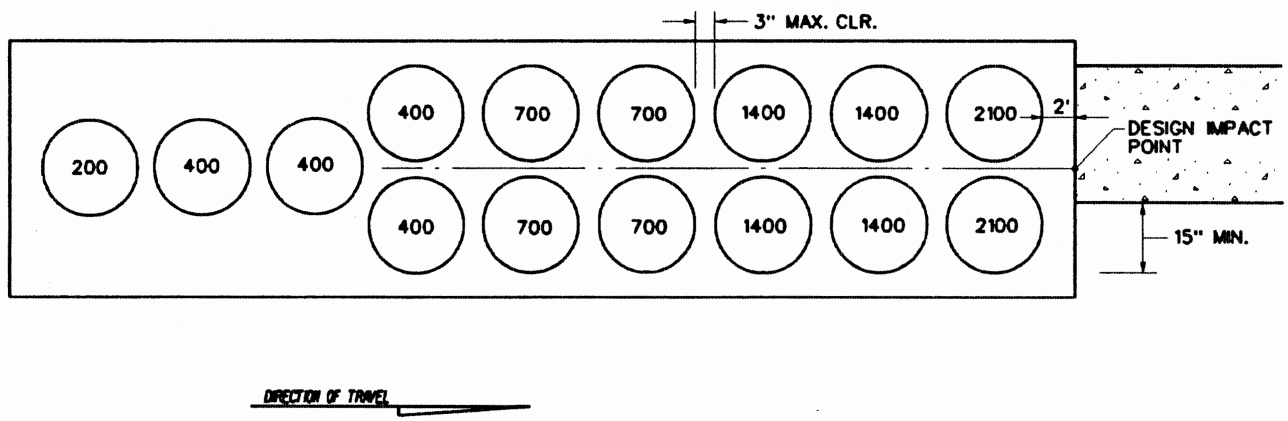
The design criteria presented in the following sections only apply to temporary crossovers on divided highways, the roadway through construction zones and detours specifically constructed for construction projects (e.g., shoo-fly). It does not apply to detours over existing routes.

14.9.4.1 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. Desirably, the design speed through the work zone will be the same as that for the approaching highway and, at a minimum, should not be more than 10 mph below the posted speed limit before the construction zone.

14.9.4.2 Lane/Shoulder Widths

Desirably, there will be no reduction in the cross section width through the construction zone. However, this is rarely practical. For Interstates and other divided highways, an 11-ft minimum lane width should be maintained through the construction zone and, preferably, with a 2-ft or wider right and left shoulder. Crossovers on divided highways are to provide a 12-ft minimum width. For other highways, the lane and shoulder width selection will be determined on a site-by-site basis. The designer should try to minimize any width reductions but, in some cases, it may even be necessary to provide a narrow lane with no shoulders. This is considered acceptable although not desirable.



NOTE: NUMBERS INDICATE WEIGHT OF SAND IN POUNDS IN EACH BARREL.
SEE STANDARD DRAWINGS FOR ADDITIONAL DETAILS

SAND BARREL ARRAY
(Work Zone Installations)

Figure 14.9C

14.9.4.3 Lane Closures/Other Transitions

The designer should ensure that the rate of taper conforms to the MUTCD criteria. These are:

$L = SW \quad (S \geq 45\ \text{MPH})$
$L = WS^2/60 \quad (S \leq 40\ \text{MPH})$

Where:

L = minimum length of transition, ft

S = posted speed limit before construction, mph

W = width of offset, ft

Table 14.9E provides the minimum taper lengths for various taper situations that are presented in construction zones (e.g., lane closures, lane shifts).

14.9.4.4 Horizontal Curvature

The horizontal curvature will depend upon the type of facility under construction and the type of detour under design. Chapter Six discusses the theoretical basis for horizontal curvature. The following applies to horizontal curvature through construction zones:

1. Temporary Crossovers on Divided Highways. Desirably, the maximum degree of curvature will be $D_{max} = 2.5°$. Under restricted conditions, Table 14.9F may be used to determine the appropriate D_{max} for curves based on the design speed of the crossover and a cross slope of .02 (i.e., S = -0.02). A cross slope of .02 should be maintained throughout the crossover.

2. Detours (All Rural Facilities and Urban Facilities Where $V \geq 50$ mph). Table 14.9F presents D_{max} for curves on detours where the typical cross slope of .02 is maintained throughout the detour (i.e., S = -0.02). These criteria are based on side-friction factors for turning roadways and based on AASHTO Method 2 for the distribution of side friction and superelevation.

3. Low-Speed Urban Facilities ($V \leq 45$ mph). Horizontal curvature for detours for these facilities will be based on the criteria presented in Section 6.1.

14.9.4.5. Sight Distance

Changes in the geometric design of the existing highway are often necessary through construction zones (e.g., lane shifts, detours). Therefore, the available sight distance to the approaching motorist is especially important. Unfortunately, the location of many design features often are dictated by construction operations. However, some elements may have an optional location. For example, when lane closures and other transitions are specially designed in the TCP, desirably these should be located so that the approaching driver has decision sight distance available to the closure or transition and, at a minimum, stopping sight distance. Desirably, the TCP should be planned so that stopping sight distance is available throughout the construction zone.

Table 14.9E

TAPER LENGTH CRITERIA FOR WORK ZONES

TYPE OF TAPER	TAPER LENGTH
UPSTREAM TAPERS	
Merging Taper	L Minimum
Shifting Taper	½ L Minimum
Shoulder Taper	⅓ L Minimum
Two-way Traffic Taper (ONE-LANE)	100 feet Maximum
DOWNSTREAM TAPERS (optional)	100 feet per lane

Source (1)

Table 14.9F

MAXIMUM DEGREE OF CURVATURE
(Detours/Crossovers)
(NO SUPERELEVATION)

Design Speed* (mph) V	e	f_{max}	Maximum Degree of Curvature, D**
20	-0.02	.27	53° 30′
25	-0.02	.23	28° 45′
30	-0.02	.20	17° 00′
35	-0.02	.18	11° 00′
40	-0.02	.16	7° 30′
45	-0.02	.15	5° 30′
50	-0.02	.14	4° 00′ R=1432.4
55	-0.02	.13	3° 00′ R=1909.8
60	-0.02	.12	2° 30′ R=2291.83

* Design Speed through the Construction Zone.

** $D = \dfrac{85{,}660\,(e + f)}{V^2}$

14.10 REFERENCES

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Chapter Fifteen

Drainage

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Chapter Fifteen

DRAINAGE

15.1 INTRODUCTION

Drainage considerations ranging from larger streamflow types to small side drain locations at a ramp require assessment by varying levels of personnel to achieve ODOT policy compliance. Drainage may be defined as the prudent consideration of all runoff related impacts upon the various components of the transportation system.

This evaluation process is extremely important in the design of a transportation facility. A significant portion of both construction and maintenance funds are expended for drainage related items. The evaluation processes use technical and non-technical input to accomplish ODOT objectives.

ODOT is developing a Drainage Manual which will contain the majority of technical instruction and applied problem solving for hydrologic and hydraulic analyses. This Manual is based upon the American Association of State Highway and Transportation Officials (AASHTO) *Model Drainage Manual* (1), 1991. The *ODOT Drainage Manual* (2) (AASHTO, with modifications for use in Oklahoma) contains proper design techniques for:

1. hydrology,
2. open channels — natural and manmade,
3. bridge waterways,
4. culverts,
5. storm drainage (surface collection and conduit system),
6. bridge deck drainage,
7. erosion control and sedimentation, and
8. bank protection.

Chapter Fifteen will not add to the technical presentation of the *ODOT Drainage Manual* but will present pertinent information which will complement the information contained therein.

15.2 DRAINAGE

The following design criteria establishes guidelines and procedures for commonly experienced drainage design conditions. Site-specific conditions may require modification of the criteria. This criteria will not replace sound engineering judgement. A glossary of technical terms has been provided in Chapter Twenty.

The drainage system includes all inlets, manholes, sewers, ditches, culverts, bridges or other hydraulic and erosion control appurtenances required to:

1. properly dispose of storm runoff disrupted or generated by the roadway and its associated construction,

2. protect the roadways and slopes from damage by erosion, and

3. maintain clear traffic lanes during the design storm(s) runoff.

15.3 HYDROLOGY

Hydrology is a science which characterizes the occurrence and distribution of surface and ground waters. Highway drainage design is concerned with the surface water hydrology of watersheds. In particular, the peak rate of runoff produced by flood events are of interest in analysis of highway drainage problems. These peak runoff rates are a prerequisite to hydraulic computations. Volume of discharge will also be needed for detention studies and routing analyses.

In general, the design of highway drainage facilities may be described in three steps:

1. Select the design frequency.

2. Compute the peak rate of runoff resulting from the design flood (or the volume if needed).

3. Design the drainage structure to pass this flood peak.

Criteria for the determination of the design flood frequency and methods for computing peak rates of runoff are presented in this section.

15.3.1 ODOT Design Flood or Storm Selection Guidelines

The selection of the design flood (or storm) frequency for use in design will be in general accordance with AASHTO and FHWA design criteria. Consideration must be given to all significant impacts and risks involved.

The ODOT Design Storm Selection Guidelines, as shown in Table 15.3A (for cross and side drain) and Table 15.3B (for storm sewers), are the minimum design criteria recommended based on all the above-mentioned factors. The selection of the design flood (or storm) frequency is the responsibility of the designer and will be approved by the Project Engineer.

15.3.2 ODOT Guidelines for Peak Discharge Design Methods

Determination of the design peak discharge is one of the most difficult phases of drainage design. There are several different methods for determining peak discharge, and it is not unusual to obtain significantly different results from different methods. The designer must recognize that hydrology is not an exact science and that, in the determination of the peak discharge, sound judgment coupled with experience is a most useful tool. The designer must also be familiar with the various methods and their applications and limitations.

The ODOT Guidelines on Peak Discharge Design Methods provide the designer with the method(s) recommended in computing the peak discharges depending on the size of the watershed. See Table 15.3C.

When more than one method is offered, it is the responsibility of the designer to choose the one that yields the most reasonable results by using engineering judgment and data available (surveys, gaging stations, newspapers, etc.).

To find the performance of the drainage structure and to predict the risks associated with floods of frequencies different than the design flood frequency, the designer should also compute discharges resulting from the review (check) floods. See Reference (2) for additional information.

Table 15.3A

ODOT DESIGN STORM SELECTION GUIDELINES FOR CROSS/SIDE DRAIN STRUCTURES

Roadway Classification	Exceedence Probability	Return Period
Freeways (Rural & Urban)	2%	50-year
Rural Principal Arterial System	2%	50-year
Rural Minor Arterial System *	4% - 2%	25 to 50-year
Rural Collector System, Major	4%	25-year
Rural Collector System, Minor	10%	10-year
Rural Local Road System *	20% - 10%	5 to 10-year
Suburban/Urban Principal Arterials	2%	50-year
Suburban/Urban Other Arterials	4%	25-year
Suburban/Urban Collectors	10%	10-year
Suburban/Urban Local Streets *	20% - 10%	5 to 10-year

* At the discretion of the designer, based on risk analysis and DHV.

FOR MINOR ARTERIAL (RURAL),
25 YR ≈ DHV < 3000
50 YR ≈ DHV > 3000

Notes:

1. Federal regulations (executive order) require interstate highways to be provided with protection from the 2% flood event and facilities such as underpasses, depressed roadways, etc., where no overflow relief is available (sump), should be designed for the 2% event. Reference FHPM 6-7-3-2.

2. For bridge design criteria, the designer should contact the Bridge Division.

Table 15.3B

ODOT DESIGN STORM SELECTION GUIDELINES FOR STORM SEWERS

Roadway Classification	Location	Return Period
Freeways & Arterials (Rural, Suburban/Urban)	At grade	10-year
Freeways & Arterials[1] (Rural, Suburban/Urban)	At sag point	50-year[1]
Collectors (Rural, Suburban/Urban)	At grade	10-year
Collectors (Rural, Suburban/Urban)	At sag point	10-year
Local Roads and Streets, AADT > 250 (Rural, Suburban/Urban)	At grade	10-year
Local Roads and Streets, AADT < 250 (Rural, Suburban/Urban)	At grade	5-year
Local Roads and Streets (Rural, Suburban/Urban)	At sag point	10-year

Notes:

1. To lessen the possibility of a pressure flow storm sewer, the designer should design the inlet and outlet conduit system from the true sump (where all runoff must be handled by the storm sewer system) forward on the 2% return frequency (50-year storm). The tailwater elevation or depth of floor in the receiving stream or culvert should also be considered.

2. For bridge design criteria, the designer should contact the Bridge Division.

Table 15.3C

ODOT GUIDELINES ON PEAK DISCHARGE DESIGN METHODS

Watershed Area	Method Recommended
From 0 to 200 acres	Rational Method
From 201 acres to 640 acres (1 SQ. MILE)	Rational Method and SCS Method
From 641 acres to 2000 acres	SCS Method and USGS Regression Method
From 2001 acres to 2500 square miles	USGS Regression Method and Log Pearson III Method
Greater than 2500 square miles	USGS Method "Flood Characteristics of Oklahoma Streams" WRI 52-73

Note: For bridge design criteria, the designer should contact the Bridge Division.

15.3.2.1 Rational Method

The rational formula is expressed in the following form:

$Q = CIA$

Where:

Q = peak discharge, cfs
C = runoff coefficient, dimensionless
I = rainfall intensity, inches/hour
A = drainage area, acres

See Reference (2) for a full discussion on the Rational Method (Standard) and the Modified Rational Method as well as example problems and definitions.

In addition, the following provides a brief overview of the Rational Method:

1. Runoff Coefficient (C). The runoff coefficient characterizes prior precipitation, soil moisture, infiltration, detention, ground slope, ground cover, evaporation, shape of the drainage area and other variables. Average values for various surface types are as follows:

Surface Type	Runoff Coefficient, C
Paved	0.7 - 0.9
Gravel	0.4 - 0.6
Cut or Fill Slopes	0.5 - 0.7
Grassed Areas	0.1 - 0.7
Residential	0.3 - 0.7
Woods	0.1 - 0.3
Cultivated	0.2 - 0.6

For flat slopes and permeable soils, use the lower values. For steep slopes and impermeable soils, use the higher values. See Reference (2) for additional information on coefficients.

Where drainage areas are composed of parts having different runoff characteristics, a weighted coefficient for the total drainage area is computed by dividing the summation of the products of the area of the parts and their coefficients by the total area, as follows:

$$C_W = \frac{C_1A_1 + C_2A_2 + \dots + C_nA_n}{A_{TOT}}$$

2. Time of Concentration (T_c). The time of concentration is the time required for water to flow from the most remote point on the watershed boundary to the proposed structure. The most remote point is defined in terms of travel time, not distance. T_c is defined as the summation of the time for overland flow, T_o, plus the time for channel or gutter flow:

$$T_c = T_o + T_f$$

The time of concentration for overland flow, T_o, is the time required for the water to run from the most remote point on the watershed boundary to the beginning point of the channel. Once the overland flow characteristics (distance, slope, ground cover) have been defined, the time of concentration for the overland flow can be computed by the equation:

$$T_o = \frac{k\,(L_o^{0.37})}{S_o^{0.20}}$$

Where:

T_o = Time of concentration for overland flow, minutes

L_o = Length of overland flow path, ft

S_o = Slope of the overland flow path, ft

k = Dimensionless coefficient, a factor of the retardation of the conveyance of the water through the drainage area (overland) and is depending on the overland ground cover

Values of k for various overland ground covers are:

Surface Type	k
Concrete, Asphalt	0.372
Commercial	0.445
Residential	0.511
Rocky, Bare Soil	0.604
Cultivated	0.775
Woodland, Thin Grass	0.942
Average Pasture	1.040
Tall Grass	1.130

The time of concentration for channel flow, T_f, is the time required for water to flow in the channel from the beginning point of the channel to the proposed structure. Once the channel length and slope have been defined, the time of concentration T_f for channel flow can be computed by the equation:

$$T_f = \frac{k' \ (L_f^{\ 0.77})}{S_f^{\ 0.385}}$$

Where:

T_f = Time of concentration for channel flow, minutes

L_f = Channel length, ft

S_f = Channel slope, ft/ft

k′ = Dimensionless coefficient

Values of k′ for various channel conditions are:

Straight, clean stream	0.00592
Average stream, few obstruction	0.00835
Meandering stream with pools .	0.01020
V-ditch	0.01252

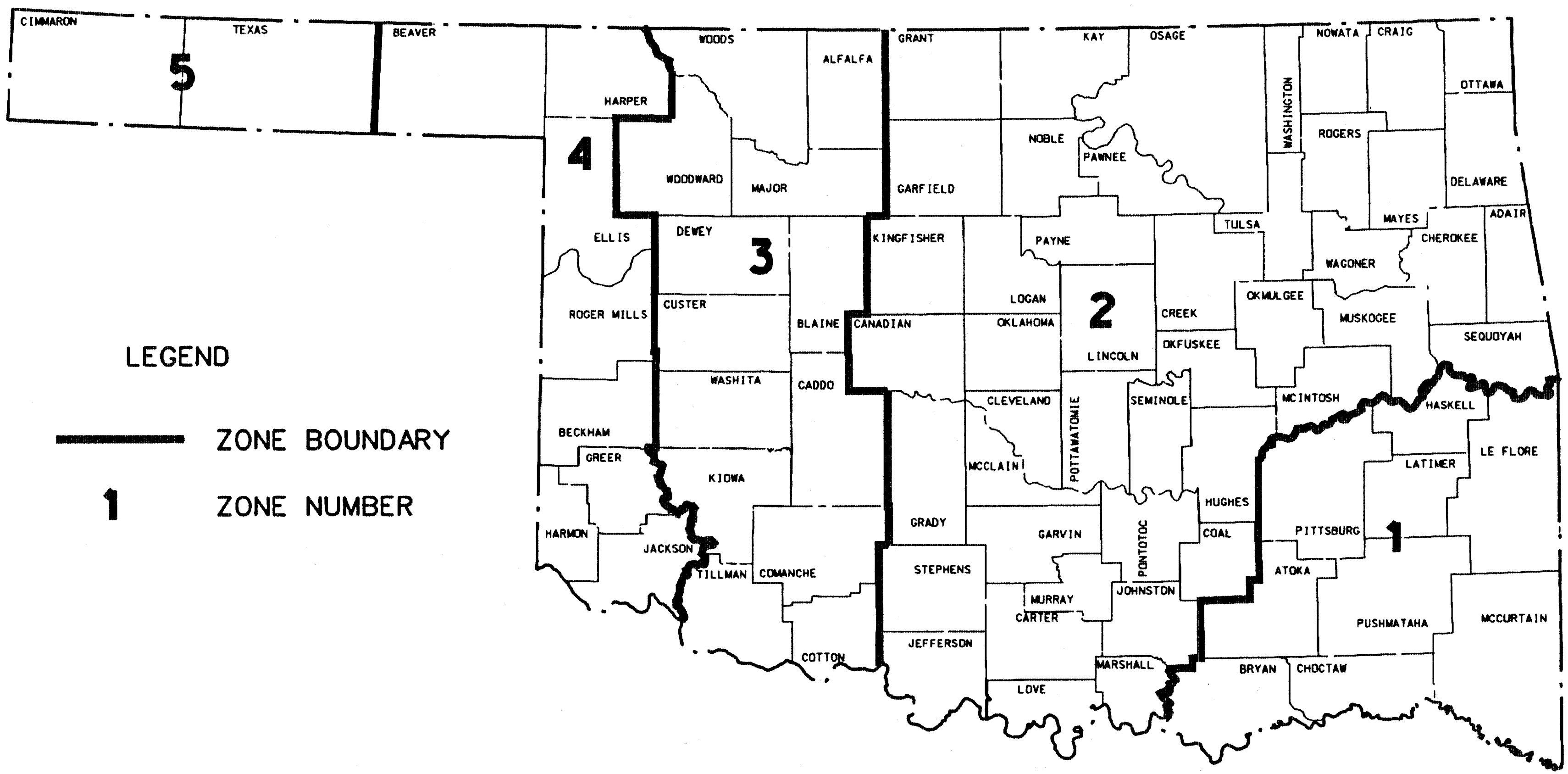
The minimum time of concentration shall be 5 minutes for densely developed, steeply sloped, urban areas (±4%) and 10 minutes for rural areas or well-developed, flat sloped, urban areas.

3. Rainfall Intensity (I). The rainfall intensity is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. As shown in Figure 15.3A, is divided into 5 geographical zones (Zone 1 to Zone 5), based on rainfall precipitation characteristics. For a given time of concentration, T_c, and a given design storm frequency, the rainfall intensity, I, of each zone can be obtained.

   The rainfall intensity - duration - frequency curves for the state of Oklahoma (Figures 15.3B to 15.3F) developed by ODOT are based on the Oklahoma rainfall data given by Technical Paper Number 40 (TP 40) and the National Oceanic and Atmospheric Administration Technical Memorandum NWS HYDRO-35.

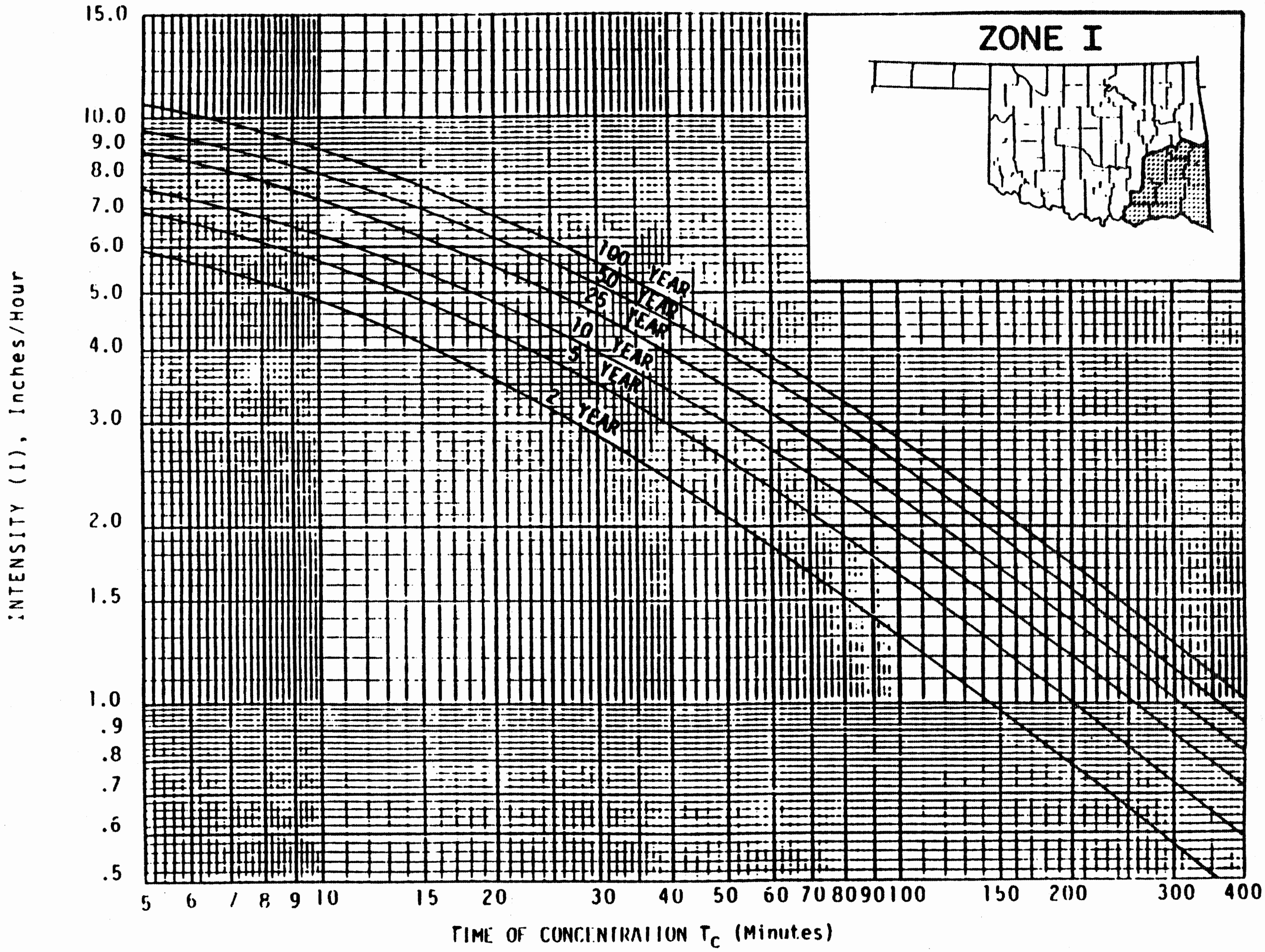
15.3.2.2 SCS Method TR55

The Soil Conservation Service (SCS) method is presented in detail in Section 4 of the U.S. Department of Agriculture *Soil Conservation Service Engineering Handbook* and Reference (1). The SCS computer program TR20 or the U.S. Army Corps of Engineers computer program HEC-1 are acceptable ways of



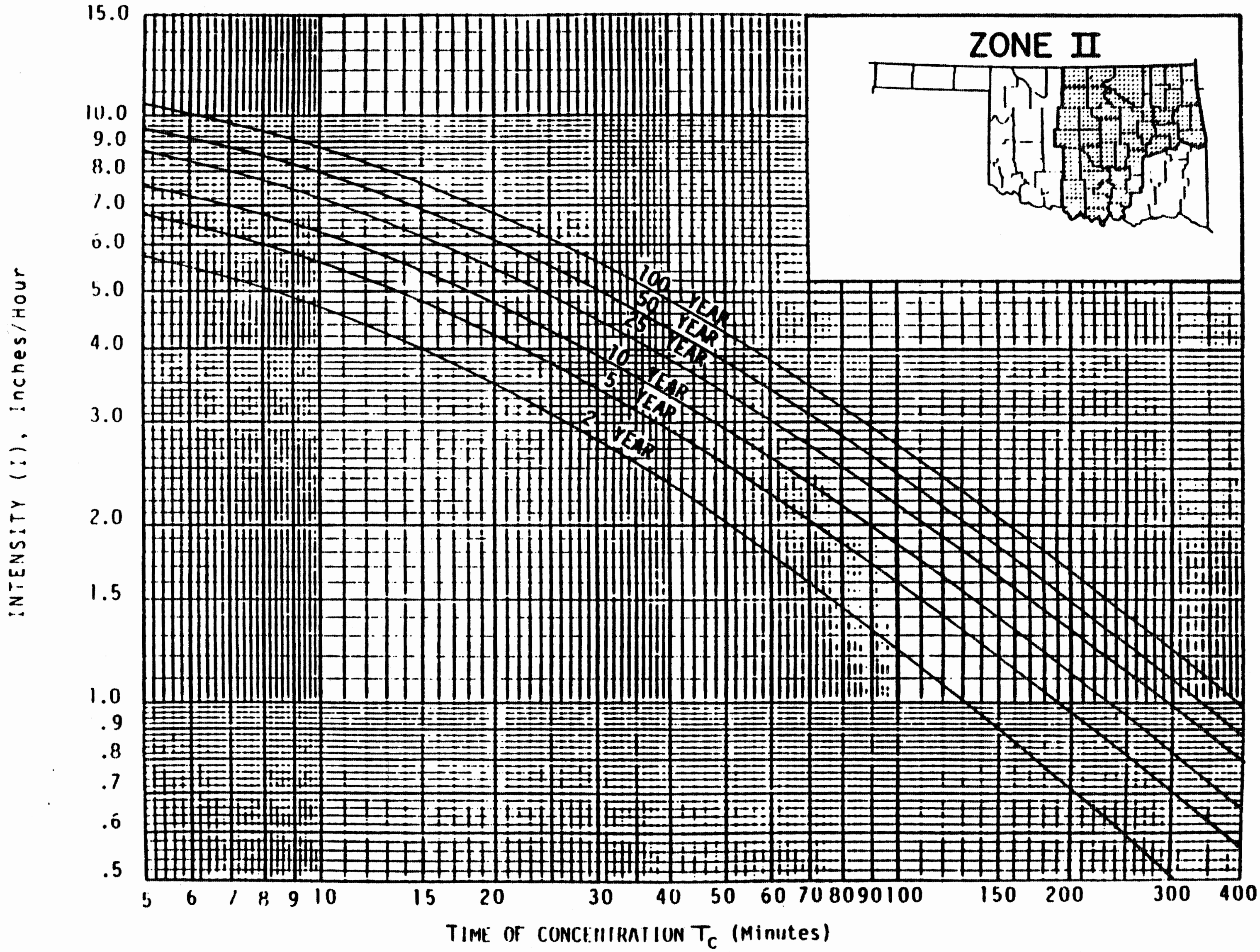
GEOGRAPHICAL ZONES IN OKLAHOMA
(for Rainfall Precipitation)

Figure 15.3A



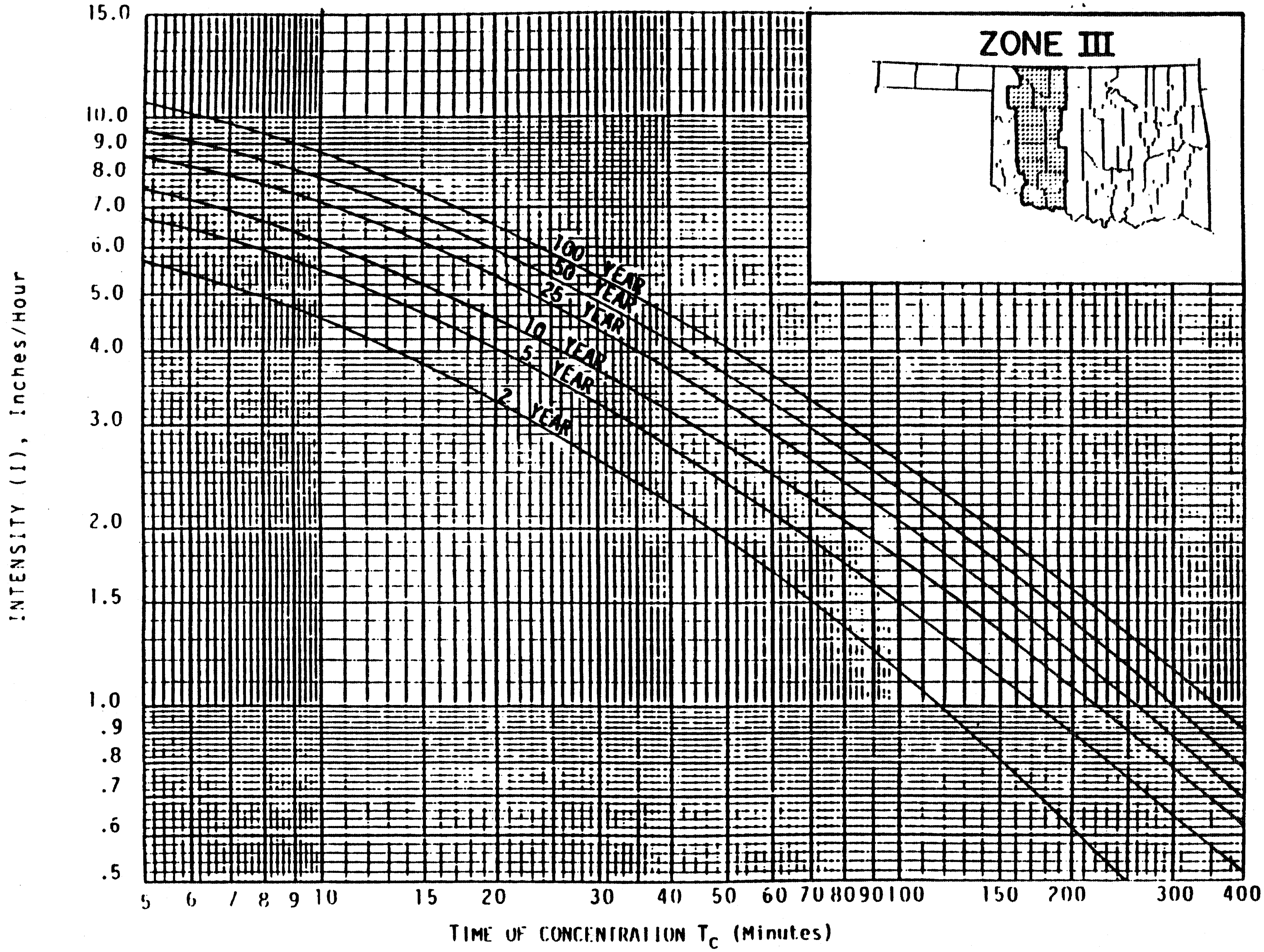
INTENSITY-DURATION-FREQUENCY CURVE
(Zone 1)

Figure 15.3B



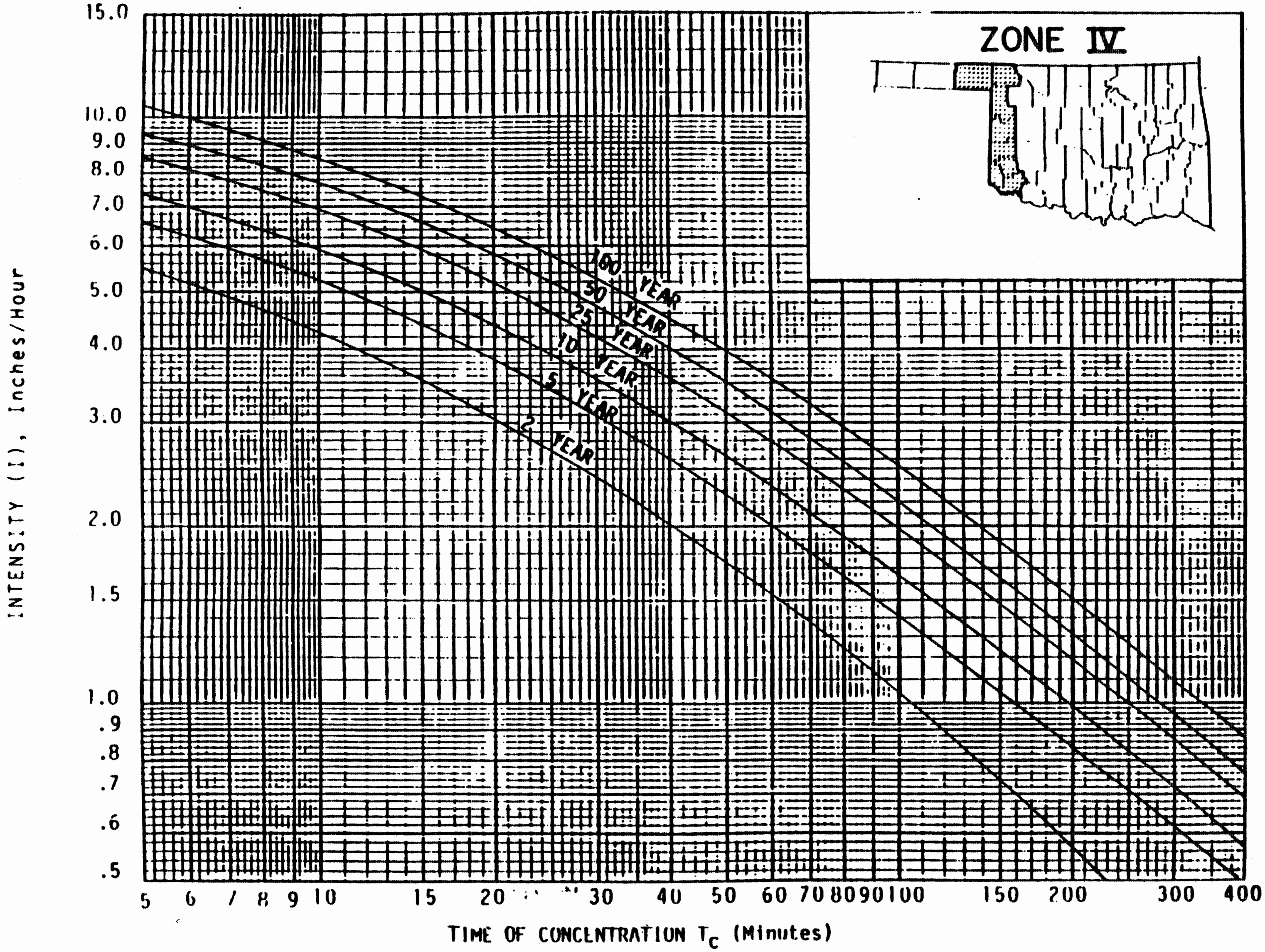
INTENSITY-DURATION-FREQUENCY CURVE
(Zone 2)

Figure 15.3C



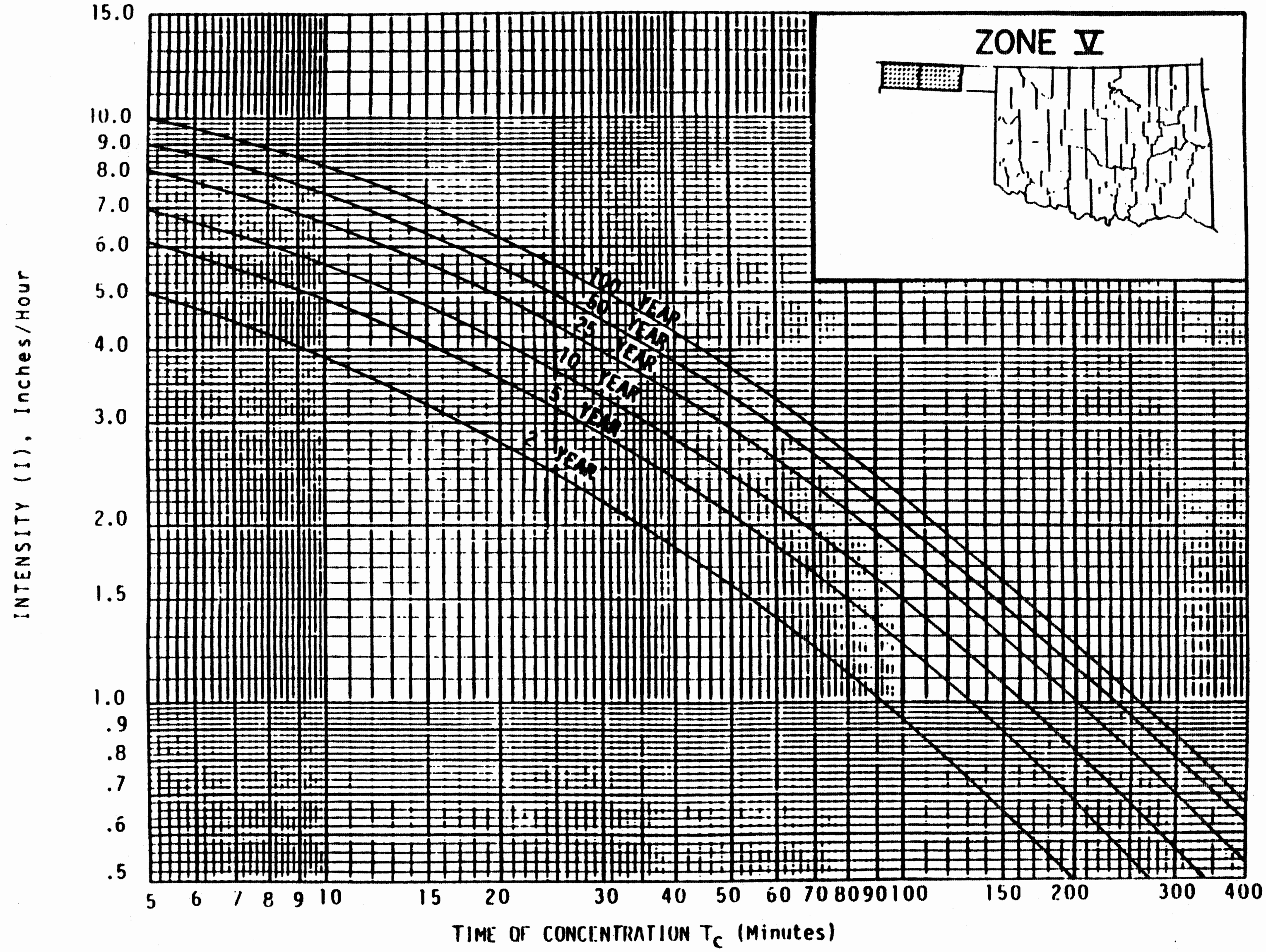
INTENSITY-DURATION-FREQUENCY CURVE
(Zone 3)

Figure 15.3D



INTENSITY-DURATION-FREQUENCY CURVE
(Zone 4)

Figure 15.3E



INTENSITY-DURATION-FREQUENCY CURVE
(Zone 5)

Figure 15.3F

utilizing the SCS methodology. The SCS publication TR55 may be used for areas up to 2,000 acres.

See also Reference (2) for ODOT's acceptable usage of this method.

15.3.2.3 USGS Regression Equations

The United States Geological Survey (USGS) regression equations for ungaged streams can be found in the USGS publication, "*Techniques for Estimating Flood Discharges for Oklahoma Streams*," Water Resources Investigation 77-54, Thomas, Corley (3).

15.3.2.4 Log Pearson III Method

See Reference (2), *ODOT Drainage Manual.*

15.3.2.5 USGS WRI 52-73

For drainage areas greater than 2,500 square miles, the peak discharge of a typical creek or river can be found in the USGS Water Resources Investigation 52-73, *Flood Characteristics of Oklahoma Streams* (22).

15.4 HYDRAULICS

15.4.1 General

Hydraulics of various roadway drainage items are normally more predictive than hydrologic items. This is generally due to the more stable set of design criteria with which designers work. Knowing the discharge rate, or volume, and various physical characteristics of the drainage item, performance values may be calculated which define the suitability of a culvert and/or give the limitations of an existing drainage item.

Research and prototype testing on roadway drainage items also contribute to the reliability of calculations which the designer will be required to develop during plan preparation.

15.4.2 Culverts

A highway embankment constitutes a barrier to the flow of water where the highway crosses water courses. A culvert is a closed conduit that provides a means of carrying the flow of water through the embankment.

15.4.2.1 Culvert Design

Culvert design will follow the methodology presented in References (1), (2) and (7).

1. Materials. Culvert materials include but are not limited to:

    a. reinforced concrete boxes,

    b. reinforced concrete pipes,

    c. corrugated galvanized steel pipes,

    d. corrugated galvanized structural plate pipes,

    e. corrugated aluminum alloy pipes,

    f. bituminous coated or paved invert corrugated metal pipes, and

    g. selected plastic and/or fiberglass product pipes.

    For additional information, the designer should refer to Section 726 of Reference (19).

    Pipes normally classified as corrugated steel are not generally used when:

    h. corrosion will be a problem,
    i. soils have a pH of less than 7.0, or
    j. there is standing water.

2. Tailwater. The designer should fully investigate conditions which will impact culvert outlet control conditions. Tailwater in the receiving stream at the same peak time and frequency as the culvert may cause the culvert to operate in outlet control.

3. Minimum Size.

    a. Pipe culverts 18" equivalent.

    b. Box culverts no less than 3 ft in height.

    c. Pipe culverts and box culvert designs which are less than minimum size require a Level Two design exception.

4. Minimum Cover. Desirable cover should be based upon 1-ft clearance between the top of the conduit and the lowest part of the subgrade or base course which might receive manipulation during the compaction operations or admixture insertion.

Refer to the "fill height tables" in the *ODOT Standard Drawings* and Reference (19) for additional specific clearances.

ODOT also occasionally uses a box designed with a full load-carrying roof, or minimum cover box. In this case, the minimum cover criteria will not apply.

5. Inlet Structures. Inlet structures are appurtenances used for the following purposes:

   a. to improve the hydraulic efficiency of the culvert,

   b. to provide erosion protection and prevent flotation, and

   c. to retain the fill adjacent to the culvert.

   These inlet structures include end sections, headwalls, beveled inlets, side tapered inlets and slope tapered inlets.

15.4.2.2 Allowable Headwater

The allowable headwater elevation should be the most critical of the following:

1. One ft below of the finished elevation of the shoulder line at any adjacent low point.

2. Elevation at which damage to upstream property will be minimized and which will prevent flooding around buildings.

If enough storage is provided at the inlet end of a culvert and right-of-way is available, the designer may investigate peak reduction by storage routing. See *ODOT Drainage Manual* for the methodology. The Project Engineer must be consulted for concurrence.

15.4.2.3 Culvert Outlet Velocities

Because culverts are hydraulically efficient, the velocity of flow through a culvert is normally greater than the velocity of flow in a natural channel of the same slope. Therefore, the potential for scour at all culvert outlets should be investigated. The designer should follow guidelines in Section 15.11 "Erosion Control."

Maximum outlet velocities at the design discharge should not exceed the following limits:

1. boxes (dirt bottom) = 12 ft/sec 3.65 m/s
2. boxes (rock bottom) = 15 ft/sec (PIPE) 4.57 m/s

Minimum outlet velocity at design discharge of 3 ft/sec should be maintained to minimize siltation under low-flow conditions. However, if supercritical stream flow occurs, high velocities at the outlet may be accepted, but the designer should consider control measures to prevent erosion.

15.4.2.4 Culvert Locations

The alignment of a culvert in both plan and profile should ensure efficient hydraulic performance and keep the potential for erosion and sedimentation to a minimum. The AASHTO *Guidelines*, Volume 4, shows alternative culvert alignments for plan and profile that may be considered. Usually, the ideal location for the culvert is in the existing channel, with a slope the same as the existing channel.

15.5 PAVEMENT DRAINAGE DESIGN

For inlet design and pavement spread calculations, use the methodology presented in *Drainage of Highway Pavements*, Hydraulic Engineering Circular #12 (HEC #12) (3). The design flow will be estimated by the Rational Method.

The minimum time of concentration will be five (5) minutes for densely developed, steeply sloped urban areas (≥4%) and ten (10) minutes for rural areas or well-developed, flatter sloped urban areas.

The Manning's roughness coefficient for street and expressway gutters are shown in the *ODOT Drainage Manual*.

15.5.1 Pavement Spread Criteria

The maximum allowable extent of flooding or water spread on the pavement depends on several factors such as:

1. risk of traffic accidents,
2. traffic delays, and
3. nuisance and possible hazards to pedestrian traffic.

allowable water spread

The allowable water spread values for curbed pavement sections are as follows:

4. Freeway and principal arterials with shoulders — No spread permitted on driving lane.
5. Ramps — Shoulder plus half width of a ramp (maintain 12-ft width open).
6. Multi-lane street — One full lane in each direction (outside lane).
7. Two-lane street — Half width of a traffic lane.
8. Medians — Lowest edge of driving lanes.

The concentration of sheet flow across pavement should be avoided. Pavement transitioning into a superelevation condition will require special treatment to minimize sheet flow across the pavement.

15.5.2 Inlet Design

1. Types. Inlets can be divided into four different types:

   a. curb inlet,
   b. grate inlet,
   c. combination inlet, and
   d. slotted inlet.

   Refer to the *ODOT Standard Drawings* and Reference (2). The combination inlet will normally be the design used.

   Grate inlets are usually used in places where:

   e. the gutter longitudinal slope is steeper than 3%,
   f. the traffic does not travel near the curb, and
   g. the clogging of the inlets by debris is not a problem.

2. Interception Capacity. Inlet grates on city streets, cross roads and frontage roads will be bicycle and pedestrian safe.

   The combination inlet consists of both a curb opening and a grate inlet. Because the capacity of a combination inlet is computed neglecting the curb opening, there is no need to decrease the theoretical capacity to account for clogging. It is usually economically efficient to design combination inlets on grade to intercept 80% of the flow, bypassing 20% to the next inlet downstream.

The slotted inlet consists of a pipe cut along the longitudinal axis with a grate of spacer bars to form a slot opening. The slotted inlet will have a 80% clogging efficiency on grade and 50% on sump condition.

3. Location. In general, inlets should be placed at all low points in the gutter grade, at median breaks and crosswalks, and on side streets at intersections where drainage would flow onto the highway pavement (gutter). Inlets should be located well away from curb ramps (wheelchair ramps) except for slotted drains parallel with the curb face.
Where pavement surfaces are warped or in transition between superelevated and normal sections, gutter flow should be picked up before the cross slope of the pavement begins to transition to lessen water flowing across the roadway.

   In a sump, the types and dimensions of inlets are determined by the maximum allowable ponding depth.

   Where significant ponding depth can occur (sump conditions), flanking inlets should be placed on both sides of the inlet at the low point of the sag. These flanking inlets should be located at an elevation of 0.3 ft higher than the low point but in no case at a distance greater than 88 ft on either side of the low point.

15.5.3 Bridge Deck Drainage

To comply with the spread criteria, it may be necessary on long bridges to construct deck drains or scuppers. For a full discussion on this item, see the *ODOT Drainage Manual* and the Bridge Division.

15.6 CONDUIT SYSTEMS

15.6.1 General

The runoff water, after being collected, will be conveyed to a nearby body of water (stream, creek, river, etc.) or cross-drain structure through the storm sewer system (manholes, junction boxes, pipes, boxes).

The flow in a conduit system is called partially full when the normal depth of flow is less than the inside diameter of the conduit (actually at 0.91D). When the normal depth of flow is equal to the inside diameter of the pipe or the hydraulic gradient is above the crown of the pipe, the storm sewer is considered surcharged or under pressure.

15.6.2 Pipe Design

1. Design. The design of the conduit system is as follows:

    a. Desirably, design for gravity flow (80% full) for the chosen design frequency.

    b. Under surcharged (pressure flow) condition for a chosen frequency storm, the calculated hydraulic gradient (including flow energy losses and all physical losses) should not be higher than the opening of the curb inlet or the grate top whichever is lowest.

    For the hydraulic analysis of the sewer system, see HEC #12 (8) and the *ODOT Drainage Manual*.

    The following are good practices in the design of storm sewers and should be followed as practical:

    c. Use standard commercial pipe sizes as shown on the *ODOT Standard Drawings*. Do not substitute shapes with equivalent areas without Project Engineer concurrence.

    d. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slopes.

    e. At changes in pipe size, match the top of inside openings (inside top of pipe soffit).

2. Conduit Types. The permissible storm sewer types are:

    a. Reinforced concrete box.
    b. Reinforced concrete pipe.

    For pipe sizes in excess of 54" (or equivalent area), check the use of reinforced concrete box.

    c. Approved Thermoplastic products.
    d. Approved steel products.

    The use of steel pipe (ductile iron, corrugated metal, etc., with bituminous coating) may be approved in areas where soil corrosion is not a major problem (pH > 7). Use and location of steel and thermoplastic conditions must be approved by the Project Engineer.

3. Manning's Friction Factor. The Manning's coefficients are:

Concrete pipe	0.012
Reinforced concrete box	0.015
Corrugated metal pipe .... (½" corrugations)	0.024
*Smooth-flow flexible pipe	0.012

* Proprietary products of various material combinations which present a smooth hydraulically efficient inside surface.

4. Velocity. The minimum velocity in storm sewers should be 3 ft per second. Maximum velocity should desirably not be greater than 20 ft per second.

5. Pipe Sizes. For main line or long laterals, ODOT policy is to use 18" minimum pipe size.

6. Minimum cover. One-ft clearance between the top of the pipe and the lowest part of the subgrade, or top of select grading. Use 15 inches where admixtures to top of subgrade are anticipated.

7. Utilities Clearance. Clearance with other underground sewer lines and utilities: A minimum of 1 ft should be maintained where practical. Additional clearance for trenching or for code compliance may be required. See the *ODOT Standard Drawings* and the *ODOT Utilities Manual* for additional guidance.

8. Pipe Location. Storm pipe line should be located outside of pavement to facilitate future repairs. If not possible due to conflicts, the next choice is under the curb.

9. Manholes and Junction Boxes. Manholes are provided on all conduit systems to permit access for inspection and cleanout and also when:

   a. there is a change in direction of conduit,

   b. there is a change in slope of conduit,

   c. there is a change in size of conduit, or

   d. if there is a junction of two or more pipes.

   The recommended maximum distance between manholes should be:

   e. for pipes up to 36-inch diameter, use 300 ft;

   f. for pipes larger than 36-inch up to 60-inch diameter, use 600 ft; and

   g. for pipes larger than 60-inch diameter, use 1000 ft.

   Any size smaller will require a Level Two exception.

   Head losses due to bends, junctions and manholes will be included in the design.

15.6.3 Pipe Strength Design Criteria

The *ODOT Standard Specifications* (19) and *Standard Drawings* have the following acceptable classes of reinforced concrete pipe — Classes II, III, IV, IV½, and V. ODOT generally uses Class III for RCP installations, except in certain situations such as:

1. high fills,

2. pipes under railroads,

3. jacked conduits, and

4. pipe exposed to extraordinary live loads during construction.

Metal pipe gauge varies with pipe size, shape and loading. See the *ODOT Standard Drawings*, Fill Height Tables, the *ODOT Standard Specifications for Highway Construction*, and manufacturer recommendations for additional information on pipe strength, loading, bedding classes and installation requirements.

15.7 ROADSIDE DITCHES AND INTERCEPTOR DITCHES

15.7.1 General

Roadside ditches are channels adjacent to the roadway used to intercept runoff and ground water occurring within the right-of-way and to carry this flow to drainage structures or to natural waterways. They are also used to intercept flow from small drainage areas adjacent to the right-of-way.

Road ditches should be grassed channels except where lining is warranted. A minimum desirable slope of 0.3% should be used for grassed channels and 0.1% for concrete paved channels.

Interceptor ditches are located on the natural ground near the top edge of cut slopes or along the edge of the right-of-way to intercept runoff from a hillside before it reaches the back slope.

An interceptor ditch, wherever feasible, should be constructed by forming a dike rather than by excavation, thereby leaving the natural cover.

Intercepting ditches should be built well back from the top of the cut slope and, generally, on a flat grade until the water can be emptied into a natural water course or brought into the road ditch in a back slope flume or pipe. In potential slide areas, storm water should be removed as rapidly as practical and the ditch paved with concrete if the natural soil is permeable.

15.7.2 Ditch Design

Flow of water in a grassed channel may be analyzed by the Manning formula. However, the solution to the problem is complicated because a single *n* value cannot be used to describe the retardance in such channels. For different *n* values and the methods to arrive at grassed waterway *n* values, see References (1), (2), (5) and (6).

Design of grassed (erodible) channels is based on the criterion of maximum permissible velocity. If the calculated velocity is greater than the maximum permissible velocity, then the channel section will erode and a larger channel, or a channel lining, will be required. Channel design should include a freeboard of a least 3 inches from the depth of flow to the top of lining and 6 inches to the top of the channel embankment.

For paved trapezoidal ditches, the minimum bottom widths are as shown on *ODOT Standard Drawings* (DC-1). Vee ditches are discouraged except in areas of extremely confining right-of-way.

Side slopes and bottom widths are determined in accordance with soil characteristics and highway design requirements. See Chapters Eight and Seventeen.

15.8 MEDIAN DRAINAGE OR ROADSIDE DRAINAGE

15.8.1 General

The basic purpose of a median is to separate opposing lanes of traffic. The width, grade and shape of a median is determined primarily by safety considerations. A wide, shallow, depressed median is usually selected as best fulfilling the median purpose.

A provision to drain the median with inlets or culverts must be included in the median design. The following contains procedures and criteria for the design of median drainage.

15.8.2 Median Drainage Design

Median cross slopes should desirably be 6:1 or flatter to reduce the hazard to vehicles driving off the road. See Chapter Eleven. Drainage structures, including paved channels located in the median, should be safe for run-off-the-road vehicles.

Median inlets are generally drop structures covered by grates or pipes. The inlets and covers should be designed to withstand the impact of vehicles and of the maintenance equipment without causing damage to the vehicle striking them. See the *ODOT Standard Drawings* for guidance.

The capacity of the median inlets can be increased by depressing them slightly, and dikes or ditch blocks across the median below the inlet will ensure complete interception. The dikes usually should have slopes of 10:1 or flatter for safety.

Conventional culverts may be used to drain wide, deep medians where the culvert end would not constitute a roadside hazard.

The intercepting capacity of a grate will be equal to 50% of the computed theoretical capacity to account for the possibility of clogging.

As a margin of safety, a freeboard of at least twelve inches should be provided from the design depth of the top of the ditch block or to the shoulder line of the roadway, whichever is lower. See Chapter Eight.

15.9 BRIDGE HYDRAULICS

For the hydraulic design of stream crossings and bridge locations, see Reference (2) and the ODOT Bridge Division, Hydraulics Branch.

15.10 CHANNELS

15.10.1 General

This section will apply to flood channels which convey runoff from off-site areas to main water courses, rivers or appropriate outlets.

15.10.2 Channel Design

1. Channel Cross Sections. Channels will normally have side slopes no steeper than 2:1 (H:V) for concrete-lined channels and 4:1 (H:V) for aggregate-lined and unlined channels. The Project Engineer's concurrence is required for steeper channel side slopes.

   The bottom width of channels should be at least eight ft with a 2% cross slope to one side. The cross slope is required for concentration of low flows and transportation of sediment. Appropriate transitions will be designed at inlets and outlets of culverts and outfalls.

2. Concrete Lining. Channel lining, where needed, may be a continuously reinforced concrete design or reinforced using wire mesh. The following will apply:

   a. Joints. Only construction joints will be used, except at channel lining/ concrete structure junctions where expansion joints are required.

   b. Weepholes. Weepholes will be provided when the channel exceeds 100 ft in length. On less than 100-ft long channels, granular backfill will vent the pressure.

   c. Lining thickness. Bottom slab thicknesses will be:

Velocity (fps)	Slab Thickness (inches)
Less than 10	5
10 to 15	6
15 to 20	7
More than 20	8

   Minimum slab thickness of 6 inches is required if channel is intended to accommodate maintenance vehicles.

   Side slope lining thicknesses (for slopes 1.5:1 or flatter) will be:

Velocity (fps)	Thickness (inches)
Less than 15	5
15 to 20	5½
More than 20	6

   For side slopes steeper than 1.5:1, channel linings will be designed as retaining walls.

   e. Cutoff Walls. Cutoff walls are generally not required to prevent progressive failure in reinforced concrete channels. However, there is some concern for the stability of lining slope walls at transitions where the cross section shape changes or at locations where channel slopes change. To prevent local buckling at these locations, cutoff walls rigidly attached to the paving should be installed to stiffen the linings.

   Cutoffs will also be required at the start and end of channels where they change to other types of lining and at existing structures where the new linings cannot realistically be made continuous with existing lining. Cutoff walls may be used to stabilize channel slopes. See the *ODOT Standard Drawings*.

3. Minimum Slope. Channels will have minimum slopes of 0.1% for concrete-lined channels and 0.2% for aggregate-lined or unlined channels. The Project Engineer's approval is required for channels with a flatter slope.

4. Maximum Velocity. For grass-lined and unlined channels, criteria for the maximum allowable velocity will be based on FHWA HDS #3. Flow velocity in concrete-lined channels will be restricted by Froude Number limitations.

   Bank protection such as riprap or gabions will be appropriately designed at inlet and outlet transitions which connect unlined channels to concrete structures.

5. Critical and Supercritical Flows. Water depths close to critical depth will create waves along the channel and should be avoided. Froude Numbers should not be within the range of 0.86 to 1.15. High-velocity supercritical flow will cause hydraulic jumps in the channel and should be avoided. Froude Numbers should not exceed 2.0.

$F_r = \frac{V}{\sqrt{g D_H}}$

6. Channel Depths and Freeboard. Where practical, the design water surface elevation will be kept below the level of natural ground. A 1-ft freeboard should be added to calculated flow depths to determine minimum channel depths. For flows near critical or supercritical with Froude Numbers equal to or greater than 0.86, the following equation will be used for freeboard, if the value obtained from the equation is greater than 1 ft:

$$F = 1/5\ (y + v^2/2g)$$

   Where:

   F = freeboard, ft
   y = flow depth, ft
   v = velocity, ft/sec
   g = gravity, 32.2 ft/sec^2

   For levied channels where the water surface elevation is higher than natural ground, an additional 1 ft to the above freeboard is recommended.

7. Drainage Outlets to Major Watercourses. The drainage area of a flood channel may be much smaller than that of the main watercourse into which the flood channel discharges. Peak times of floods at the junction from two different sources normally would be quite different. Peak of the flood channel may arrive at the junction when the water level of the main watercourse is low. An energy dissipator may be required to eliminate energy from the flood channel at the outlet. The outlet will be designed to the 100-year peak flow of the channel concurrent with the 10-year peak flow in the main watercourse. It will also be designed to the 10-year peak flow of the channel concurrent with the 100-year peak flow in the main watercourse. For bank protection measures at the outlet and nearby channel, water levels of the 100-year peak flow, in either the main watercourse or flood channel (not concurrent peaks), should be considered. The designer should consult with the Bridge Division, Hydraulics Branch, and the Roadway Drainage Engineer, Rural Design Division.

8. Energy Dissipators. Drop structures, chute spillways, stilling basins and other energy dissipators will be designed in

accordance with FHWA HEC #14 *Hydraulic Design of Energy Dissipators for Culverts and Channels* and the *ODOT Drainage Manual*.

9. Maintenance. On both sides of drainage channels, a 20-ft strip, sloping towards the channel, should desirably be provided. Right-of-way limitations which do not meet this criteria will be brought to the attention of the Project Engineer. At drainage structures, a 10-ft wide maintenance access ramp may be incorporated into the channel design where practical.

15.11 EROSION CONTROL

15.11.1 General

Providing for the temporary and permanent control of erosion is a necessary part of the complete design of any highway construction project. This subsection is concerned with methods and criteria for the control of both sheet and gully erosion in roadside ditches, culvert outlets, channels, bridges, both during construction (Temporary Erosion Control) and after construction (Permanent Erosion Control). See the *ODOT Drainage Manual*. Consultation with the Rural Design Division, Roadside Development Branch, personnel is recommended for technical input. See Section 17.14.

15.11.2 Ditch Protection Design

Control of ditch erosion is given careful consideration during design to reduce maintenance costs and to improve appearance. All ditches are analyzed to determine the necessary erosion control measures. Ditch erosion is controlled by widening ditches and flattening ditch grades, or by the application of proper ditch protection such as netting, sodding and concrete lining.

1. Design of Permanent Protection. After the design flow rate has been computed and the channel slope has been established, the ditch section (usually a standard section) is analyzed as a grassed channel. This analysis is implemented by procedures outlined in Section 15.7 "Roadside Ditches and Interceptor Ditches." If this analysis shows that the maximum velocity for a grassed channel is exceeded, then one of the following may be used:

    a. The ditch may be modified by widening the ditch section or flattening the ditch slope, if these actions are feasible; or

    b. The ditch may be lined. It should be noted that if the ditch is lined, consideration should be given to reducing the channel dimensions. The capacity of a paved ditch, for example, can be as much as ten times the capacity of a grassed waterway of the same dimensions.

2. Design of Temporary Protection. Temporary erosion protection should be provided for grassed channels while the grass is being established. Jute mesh or excelsior mat are usual forms of temporary erosion protection. The designer can propose temporary protection measures in accordance with standard ODOT practices.

15.11.3 Culvert Outlet Erosion Control

Because culverts are hydraulically efficient, the velocity of flow through a culvert is generally greater than the velocity of flow in a natural channel of the same slope. When this high velocity flow exits the culvert, a scour problem may develop. Scour holes at culvert outlets develop because of the need to dissipate excess energy. These scour holes may undermine the culvert headwall and endanger the structure or damage the embankment. Therefore, the potential for scour at all culvert outlets should be investigated.

Data in FHWA HEC #14 may be used as a guide to aid the designer in determining when scour protection is needed at culvert outlets. The stability of the channel at the culvert outlet is a function of the culvert outlet velocity and the average stone size of the channel bed.

Generally, culvert outlet protection is needed when the culvert outlet velocity is greater than 8 fps. If it is determined that culvert outlet protection is needed and the calculated outlet velocity is between 8 and 20 fps, then a lining should be considered which both protects and slows the velocity. For outlet velocities greater than 20 fps, an impact type energy-dissipating headwall should be used.

15.11.4 Slope Erosion Control at Top of Slope Ditch

Interception levees and ditches may be required at the top of earth back slopes at all locations where the natural ground slopes toward the back slope. See Chapter Eight.

Interception ditches receive the same erosion control analysis and treatment as other ditches using the same controls, with particular attention for outlets.

Control of sheet erosion on embankment and cut slopes is discussed in the *ODOT Drainage Manual*. Contact the Rural Design Division, Roadside Development Branch, for additional information.

15.11.5 Channel Protection (Riprap)

For riprap design, follow the guidelines in HEC #15 "Design of Stable Channels with Flexible Linings," U.S. Department of Transportation, Federal Highway Administration, and the *ODOT Drainage Manual*.

15.11.6 Bridge Embankment Protection

The designer should contact the Bridge Division and refer to the *ODOT Drainage Manual*.

15.11.7 Temporary Erosion Control and Abatement of Erosion and Water Pollution

This section consists of the measures required to control erosion on the project and in areas outside of the right-of-way where the work is accomplished in conjunction with the project. The objective is to prevent pollution of water, the detrimental effects to public or private property adjacent to the project right-of-way and damage to work on the project. These measures will consist of temporary erosion control features which include bale barriers, sediment basins, sediment filters, siltation screens, diversion dikes and slope drains.

Bale barriers are used as erosion checks to trap sediment and assist in maintaining low ditch velocities. Suggested spacings for erosion checks are as follows:

Ditch Slope	Spacing (ft)
0.1%	±300
0.2%	±200
0.3%	±100
>0.6%	±50

Sediment basins are located at pipe outlets and are used for removal of large volumes of sediment and turbidity prior to runoff discharge into live streams. Sediment basins will have a capacity for storage of 1.0 acre/inch sediment from its drainage area under construction less the estimated sediment removal of other erosion control features within the drainage area. No storage will be below the spillway elevation.

Sediment filters are used to retain material in front of inlets. Siltation screens are fences used to protect areas outside of the right-of-way from erosion material from fill embankments.

Diversion dikes are used to divert runoff from areas than can erode.

Slope drains are used to concentrate runoff down a slope in a protected area preventing wide spread slope erosion. They are located at low points of the top of the slope, or as required for proper drainage. See the *ODOT Drainage Manual* and Rural Design Division, Roadside Development Branch.

15.12 DETENTION BASINS

15.12.1 General

Detention basins may be provided for attenuation of peak discharges by storing runoff from on-site or off-site drainage. Outflow discharges from the basin will not cause peak discharges for the range of storm frequencies greater than peak discharges with no project. Design criteria will be as covered in the *ODOT Drainage Manual.*

15.12.2 Basin Design

1. Hydrographs. Use the SCS Method, Type II 24-hour rainfall as shown in the *ODOT Drainage Manual* or HEC #1, and other methods as may be approved by the Project Engineer.

2. Routing. Maximum detention storage will be determined through the basin routing by using the inflow hydrograph and outflow rating curve. This routing can be accomplished using the COE HEC #1 computer program Modified Pulse Methodology.

3. Outflow Structure. The outflow structure must be placed to ensure complete basin drainage.

4. Emergency Spillways. Every detention facility should have an emergency spillway which is designed to allow overflow of runoff when the outlet is blocked. A broadcrested weir is normally used for overflow purposes because it is not easily blocked.

5. Layout. Side slopes should not be steeper than 3:1 (H:V). The maximum design water level will not exceed the level of natural ground. Low flow channels or underdrains are required on the bottom of the basin leading from inlet to outlet.

6. Freeboard. Use a minimum of 1-ft of freeboard between the design flow level and the top of the basin.

7. Maintenance. Minimizing future maintenance and operational costs are important elements. Ease of removal of silt and debris should be considered in the design of the facility.

8. Erosion Protection. Slopes will be adequately protected from erosion by adequate lining such as grass, riprap, etc.

9. Fencing. To negate any possibility of attractive nuisance liability, any detention basin with permanent pools and/or with extremely slow drain-down should be analyzed for the design of a security fence system. The fence should include climb-retardant devices and have lockable gates of adequate width to pass the largest maintenance vehicle needed or have drop-down sections which can be easily removed and rebuilt.

15.13 PUMP STATIONS

Pump stations are usually designed to drain those portions of a depressed roadway that cannot be connected directly to a gravity storm system. When a pump station may be required, contact the Rural Design Division, Roadway Drainage Engineer for assistance.

15.14 POLICY STATEMENTS

Applied policy at federal, state and local government levels are contained in various publications available to the designer. Over time, the designer will gather quite an extensive publications library and should strive to keep relevant information current. The discussion in this section will be to guide the designer to sources of the various policy statements.

15.14.1 Federal Policy

Federal policy which affects, or is affected by, the work done by the ODOT drainage designers, or by the consulting engineer's drainage designer, is cited fully in the *ODOT Drainage Manual* (2), Chapter Three, Policy.

Chapter Two of the *ODOT Drainage Manual*, Legal Aspects, has a full discussion of legal aspects of drainage practice. Chapter Four, Documentation, of Reference (2) presents the concept support and actual guidance for documentation which is vital to the designer's record keeping process.

15.14.2 State Policy

Statement of the general types of State policy is contained in Chapter Three, Policy, of Reference (2).

The application policy statement (State) is contained in the specific chapters of Reference (2) which also have the related design criteria fully explained.

15.14.3 Local Policy or Ordinances

The designer may adhere to local policy contained in the jurisdictional agency headquarters (County Courthouse, City Hall building, Water District Headquarters, etc.) under the following guidelines:

1. drainage policy or ordinance has been developed under the professional auspices of an engineer trained in drainage-related subdiscipline; and

2. strict adherence to the intent and letter of the policy or ordinance will result in no additional public fund expenditure (State and Federal funds) as a result of the compliance.

In the case of consultant employees, the ODOT project engineer will decide on the level of compliance.

If the requirements of #1 above are met, and #2 above would result in additional cost, a compromise may be in order.

15.15 SUMMARY

The designer has a good deal of latitude in arriving at a solution to specific problems presented by transportation facility road design. The latitude of engineering judgment inherently invokes the necessity of using good state of practice approaches and being consistent in application of similar solutions.

All members of the public must be treated objectively, and those individuals and/or groups affected by the designer's work should be fully heard. If the taxpayer can be accommodated with minimal cost, then their actual or perceived losses may be softened.

The most beneficial tool of the designer may be documentation. Documentation will be a record of the process, answers and decisions reached in solving a particular problem. It should reflect the actual and assumed future watershed characteristics which the designer used in his solution. ODOT may need this data, especially in the future absence of the original designer, to explain, extend or defend the original solution.

15.16 REFERENCES

1. *Model Drainage Manual*, American Association of State Highway and Transportation Officials, 1991.

2. *ODOT Drainage Manual*, Oklahoma Department of Transportation, 1992.

3. *Techniques for Estimating Flood Discharges for Oklahoma Streams*, Water Resources Investigation 77-54, U.S. Geological Survey, Water Resources Division, June 1977.

4. U.S. Department of Transportation, FHWA, *Hydraulics of Bridge Waterway*, Hydraulic Design Series HDS No. 1.

5. U.S. Department of Transportation, FHWA, *Design Charts for Open-Channel Flow*, Hydraulic Design Series HDS No. 3.

6. U.S. Department of Transportation, FHWA, *Design of Roadside Drainage Channels*, Hydraulic Design Series HDS No. 4.

7. U.S. Department of Transportation, FHWA, *Hydraulic Design of Highway Culverts*, Hydraulic Design Series HDS No. 5.

8. U.S. Department of Transportation, FHWA, *Drainage of Highway Pavements*, Hydraulic Engineering Circular HEC-12.

9. U.S. Department of Transportation, FHWA, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular HEC-14.

10. U.S. Department of Transportation, FHWA, *Design of Stable Channels with Flexible Linings*, Hydraulic Engineering Circular HEC-15.

11. U.S. Department of Transportation, FHWA, *Design of Urban Highway Drainage — The State of the Art*.

12. State of Oklahoma, Department of Transportation, *Drainage Design Manual* Preliminary Edition, 1988.

13. City of Tulsa, Oklahoma, *Design Criteria Manual*.

14. U.S. Army Corps of Engineers, Hydraulic Engineering Center, *HEC-1 Flood Hydrograph Package*, Davis, California.

15. U.S. Army Corps of Engineers, *HEC-2 Water-Surface Profiles Generalized Computer Program*, Davis, California.

16. U.S. Department of Agriculture, Soil Conversation Service, National Engineering Handbook, Section 4, "Hydrology."

17. U.S. Department of Agriculture, Technical Release TR-20, *Computer Program for Project Formulation, Hydrology*.

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19. *Standard Specifications for Highway Construction* and Supplemental Specifications thereto, Oklahoma Department of Transportation, current edition.

20. *Utilities Manual*, Oklahoma Department of Transportation, Right-of-Way Division, current edition.

21. *Highway Drainage Guidelines*, Volumes I through VIII, American Association of State Highway and Transportation Officials, 1987.

22. *Flood Characteristics of Oklahoma Streams*, United States Geological Survey, Water Resources Investigation 52-73 (USGS), V. B. Sauer.

23. Oklahoma Department of Transportation, *Standard Drawings*, ODOT.

Chapter Sixteen

Pavement Design

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Chapter Sixteen

PAVEMENT DESIGN

16.1 INTRODUCTION

Pavements represent the largest expenditure of funds for the Oklahoma Department of Transportation in the construction and maintenance of highways and streets. The traveling public is more influenced by the quality of the pavement than by any other element on the system. Many other items are important to the road user such as signing and striping, safety features and properly engineered vertical and horizontal alignments. However, the most lasting impression the typical road user retains is the quality of the ride on the road. All other roadway elements may become insignificant to the driver if a pavement structure is severely faulted or deeply rutted.

Pavement structures are systems of inter-connected materials which, when properly designed, constructed and maintained, deliver a durable, high-quality ride for the traveling public. Good design and construction processes result in the lowest annual maintenance costs for any pavement system.

Chapter Sixteen presents a narrative discussion on the two types of pavement structures — flexible pavements and rigid pavements. The three primary areas of pavement engineering include the:

1. design methodology;
2. construction materials, processes and control; and
3. routine, preventive and rehabilitative maintenance.

When these are properly implemented and meet current ODOT policies, either of the two types of pavement structure will function properly.

Chapter Sixteen also presents a brief introduction into the role of the designer, ODOT Pavement Design Engineer, and his interface with the ODOT Pavement Design Committee.

16.2 PAVEMENT DESIGN PROCEDURE

The current procedure for developing a "pavement design" is for the ODOT Pavement Design Engineer to design the pavement or to furnish guidance to the designer to design the pavement. The ODOT Pavement Design Engineer or the designer is furnished pertinent data, including:

1. the proposed project scope relative to the pavement criteria, including the scoping report if available;

2. design traffic data;

3. preliminary plans with grades and, for some special designs, cross sections; and

4. soils report.

After completion of the design by the ODOT Pavement Design Engineer or the designer, the ODOT Pavement Design Engineer prepares a pavement recommendation for submission to the Pavement Design Committee, that includes:

5. projected letting date;

6. project description;

7. rigid and flexible pavement design alternatives with estimated cost comparisons;

8. information on pavement type, availability of materials, traffic data, soils and environment;

9. recommended pavement design alternative; and

10. design parameters.

The package is presented to the ODOT Pavement Design Committee for their review. Based upon their expertise, the design options may be approved as presented or the Committee may request another design alternative. The ODOT Pavement Design Committee consists of the:

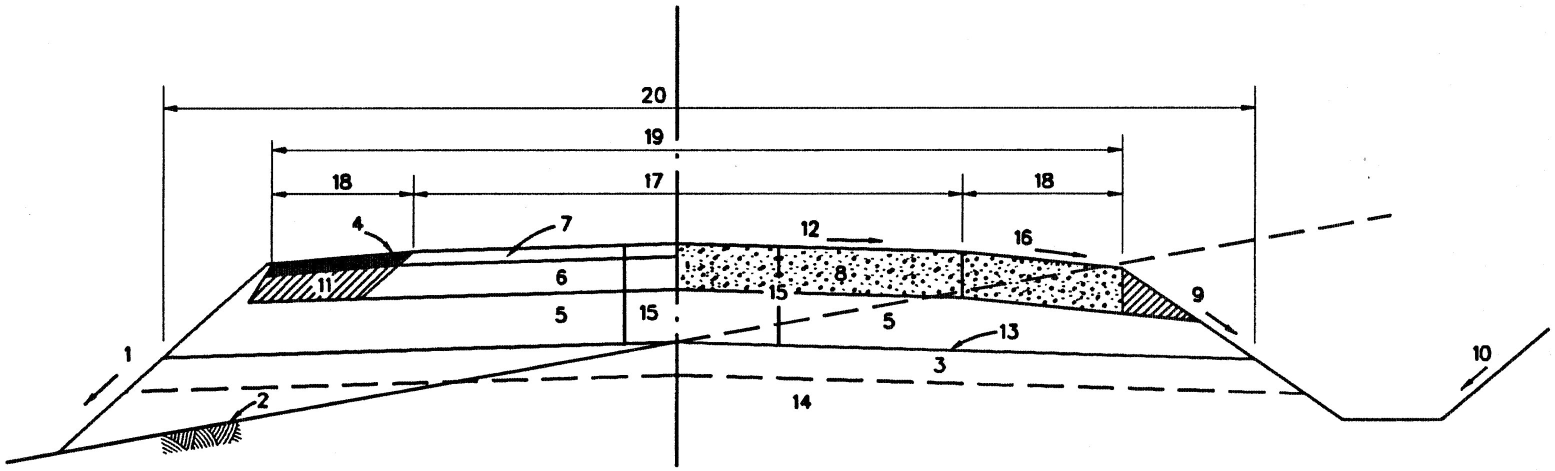
1. Chief Engineer,
2. Assistant Director — Design,
3. Assistant Director — Operation,
4. Construction Engineer,
5. Materials Engineer,
6. Rural Design Engineer,
7. Urban Design Engineer,
8. Research and Development Engineer, and
9. Pavement Design Engineer.

In addition to the Pavement Design Committee, the Field Division Engineer also reviews and concurs in the proposal. Upon approval, FHWA is furnished the completed design recommendation.

16.3 PAVEMENT TYPES

There are two types of new pavement structure systems — flexible and rigid.

Figure 16.3A presents a definition of pavement structure terms used in Chapter Sixteen. For further definition of the terms, see Reference (2).



FLEXIBLE PAVEMENT SECTION

RIGID PAVEMENT SECTION

Source (2) Revised

STRUCTURAL DESIGN TERMS

1 - FILL SLOPE	11 - SHOULDER BASE
2 - ORIGINAL GROUND	12 - CROWN SLOPE
3 - SELECTED MATERIAL OR PREPARED ROADBED	13 - SUBGRADE
4 - SHOULDER SURFACING	14 - ROADBED SOIL
5 - SUBBASE	15 - PAVEMENT STRUCTURE
6 - BASE COURSE	16 - SHOULDER SLOPE
7 - SURFACE COURSE	17- TRAVEL LANES
8 - PAVEMENT SLAB	18 - SHOULDER
9 - FORE SLOPE	19 - ROADWAY
10 - BACK SLOPE	20 - ROADBED

Note: It is always desirable to have the ditch bottom lower than the top of subgrade or the bottom of the modified layer.

RIGID OR FLEXIBLE PAVEMENT STRUCTURE

Figure 16.3A

16.4 PAVEMENT DESIGN METHODS

16.4.1 General

This Section presents an overview of the two methods currently used for pavement design. These are the methodology presented in the American Association of State Highway and Transportation Officials (AASHTO) *Guide for Design of Pavement Structures* (2), hereinafter referred to as the *AASHTO Pavement Design Guide*; and the ODOT Oklahoma Subgrade Index (OSI) method (1), developed in-house.

16.4.2 The AASHTO Guide for the Design of Pavement Structures

The AASHTO *Pavement Design Guide* (2) represents the evolution of much nationwide research from the original AASHO pavement tests completed in the mid-1950's. Although the data input is much more extensive and calculations more complicated, available computer software makes calculations relatively simple and fast.

ODOT has a resilient modulus testing machine and furnishes that data as part of the Materials Division soils report for selected projects. Some data, however, are not the result of tests from ODOT labs, but rather values estimated from a range of values suggested by the *Pavement Design Guide* (2). There are seven data requirements common to designing either flexible or rigid pavements by the AASHTO method.

For detailed design procedures, the designer is referred to the AASHTO *Pavement Design Guide* (2).

16.4.3 The Oklahoma Subgrade Index (OSI) Method

The Oklahoma Subgrade Index (OSI) Method (1), developed by the Department in the early 1960's, is an empirical method for flexible pavement design. The OSI number of an individual soil is calculated from a formula that requires the liquid limit, plasticity index, and the percent of fines passing the #200 sieve. An OSI number is chosen that is representative of the soils that are found in the top 2 ft of the final subgrade within a logical extent of the project.

The OSI rigid pavement design policy is as follows:

1. Minor collectors should have 9 inches of dowel-jointed Portland Cement Concrete (PCC).

2. Major collectors should have 9 inches of continuously reinforced concrete pavement (CRCP) or 10 inches of dowel-jointed PCC.

3. High-type facilities (e.g., freeways, principal arterials) always have 10 inches of CRCP.

4. Every rigid design is placed on a 4-inch, non-erodible base. Shoulders should be plain PCC pavement tied to the travel lane. Plastic soils with the potential to swell and shrink (PI>25) should be stabilized or undercut and replaced.

16.4.3.1 Selection of Wheel Load Criteria for Design

Wheel loads for various functional classifications are shown in Table 16.4A.

Table 16.4A

AREA	FUNCTIONAL CLASSIFICATION	DESIGN ADT RANGE (vpd)	MINIMUM DESIGN WHEEL LOAD (lbs.) (1)
RURAL	Freeways	ALL	15,000
	Principal Arterials	over 5000	15,000
		0 to 5000	12,000
	Other Arterials	over 2500	12,000
		0 to 2500	9,000
	Collectors (2)	over 1200	9,000
		0 to 1200	7,000
	Local Roads (2)	ALL	7,000
SUBURBAN or URBAN	Freeways	ALL	15,000
	Principal Arterials	ALL	12,000
	Other Arterials	ALL	9,000
	Collectors (2)	ALL	9,000
	Local Streets (2)	ALL	7,000

Notes:

1. For facilities with heavy truck traffic ($T_3 \geq 25\%$), use a design wheel load of 15,000 pounds.

2. For pavement design for county road projects administered by the Local Government Division, see Reference (6).

16.4.3.2 Determination of Equivalent Base Thickness

The equivalent base thickness (EBT) is the necessary paving thickness of a stabilized aggregate base course material to support a given wheel load. The unadjusted equivalent base thickness is determined from the Oklahoma Subgrade Index Number (OSI) of the soil (see Figure 16.4A) and the wheel load. OSI and wheel loads are combined in Figure 16.4B. Adjustments for shoulders (Table 16.4B), traffic, and climate (Figure 16.4C) are made. When this adjustment has a positive value, it is added to the unadjusted EBT, and the result is the required equivalent base thickness.

The adjustments are made by using the nomograph in Figure 16.4D and the accompanying Table 16.4C.

The Planning Division supplies the Design Division with traffic data which is used to compute the traffic factor. The traffic factor is the product of:

1. The design average daily traffic (Design ADT). It is determined by averaging the present ADT and the design year ADT.

2. The percent of heavy commercial truck traffic.

3. The number of overloaded axles per one hundred heavy commercial trucks.

4. The lane factor. This is an adjustment used to estimate the proportion of traffic in the design lane. For two-lane roads, the lane factor is 1.0, four-lane is 0.8 and for six or more lanes 0.6.

The equation is as follows:

Design ADT x % Heavy Commercial Truck Traffic x Overloaded Axles/100 x lane factor (1.0, 0.8 or 0.6) = The traffic factor.

In addition to the factors considered above, the potential vertical rise (PVR) of the soil must be considered. Soils with high PVR values should, where possible, be eliminated from the upper portion of the subgrade. This may be accomplished by subgrade modification or, in some cases, by select grading. If they cannot be eliminated from the upper portion of the subgrade, sufficient material must be placed on top of them (to act as a restraining force) to prevent excessive movement of the soil (see Figure 16.4E). Typically, lime is estimated at 5% (by weight) and fly ash at 18% (by weight) of the modified subgrade. Subgrade soil should be estimated at 120 lbs per cubic foot. The actual quantity of lime/fly ash shall be determined in the construction phase.

See Table 16.4D for equivalent base thickness values for selected pavement materials.

16.4.3.3 Pavement Thickness Design Steps

1. Locate project to find functional classification of highway.

2. Use Design Year ADT to find design wheel load in Table 16.4A.

3. Use soil report and Table 16.4A to determine OSI with design wheel load. Use EBT nomograph (Figure 16.4B) to determine unadjusted EBT.

4. Locate the shoulder design in shoulder factor table (Table 16.4B) to establish the shoulder factor.

5. Use State map (Figure 16.4C) to locate climate factor for project location.

6. Calculate traffic factor (design ADT x T_3% x overload % x lane factor).

7. Use nomograph (Figure 16.4D) connecting shoulder factor to traffic factor; holding turning point, connect to climate factor. Read STC value on column "E"; it never exceeds 3.00. STC is the resultant factor from the nomograph on Figure 16.4D. It is the resultant of the shoulder, traffic and climate factors.

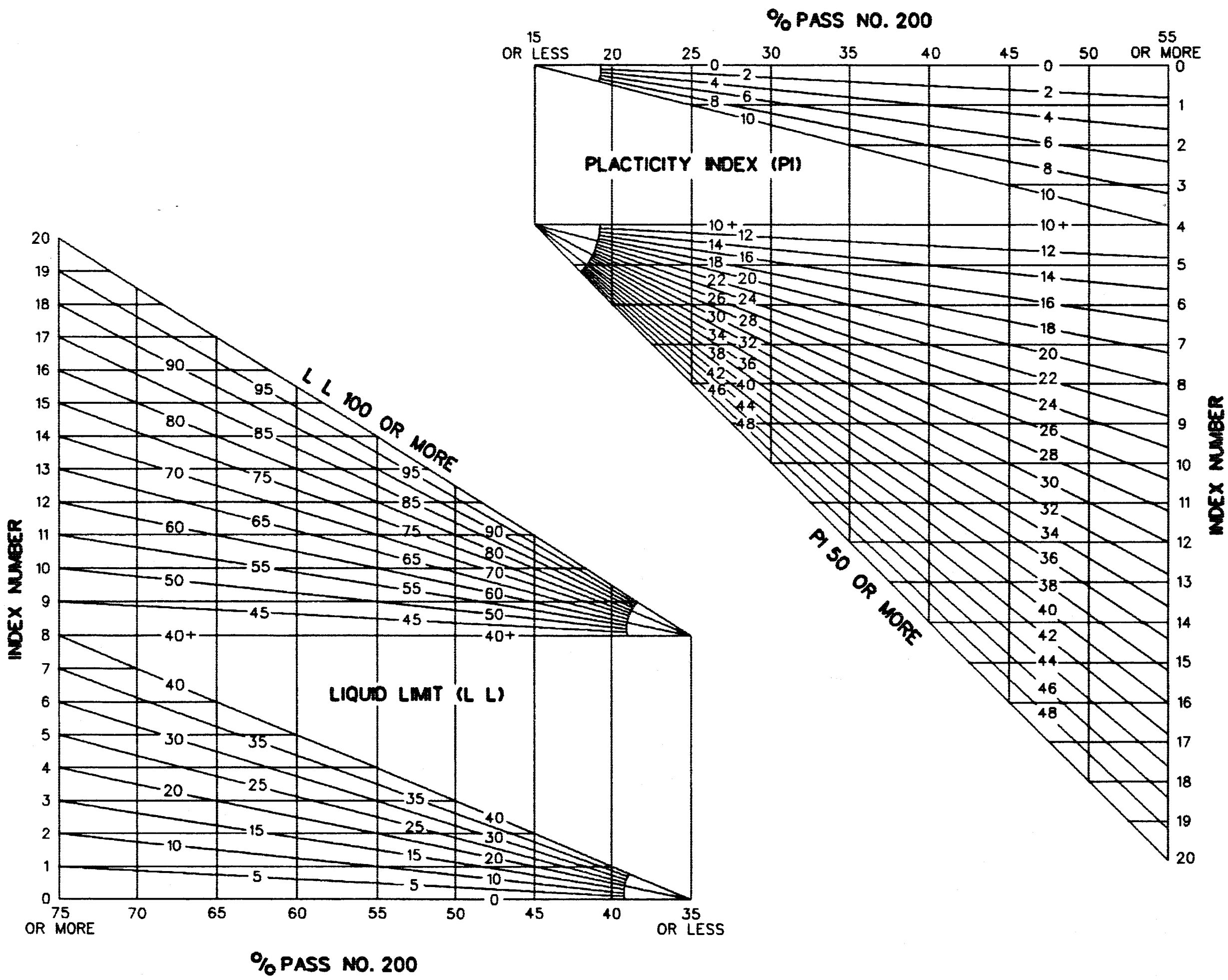
8. Use STC Table of EBT adjustment factors (Table 16.4C) by identifying shoulder factor column and finding nearest STC factor. Move horizontally along the row to the left-most column to find EBT adjustment.

9. Add EBT adjustment to unadjusted EBT for the design EBT.

10. The thickness of pavement layers are proposed to meet the required design EBT and meet overburden requirements (Figure 16.4E).

11. Local availability, cost of materials, and construction constraints dictate the type and thickness of pavement layers.

12. Compare to minimum design thicknesses. (Table 16.4E).



To Use This Chart

1. Determine the percent of the soil passing the #200 sieve, and the L.L. and P.I. of the soil.

2. On the L.L. chart, find the % passing #200 along the bottom of the chart and move vertically up to the L.L. (sloping) line.

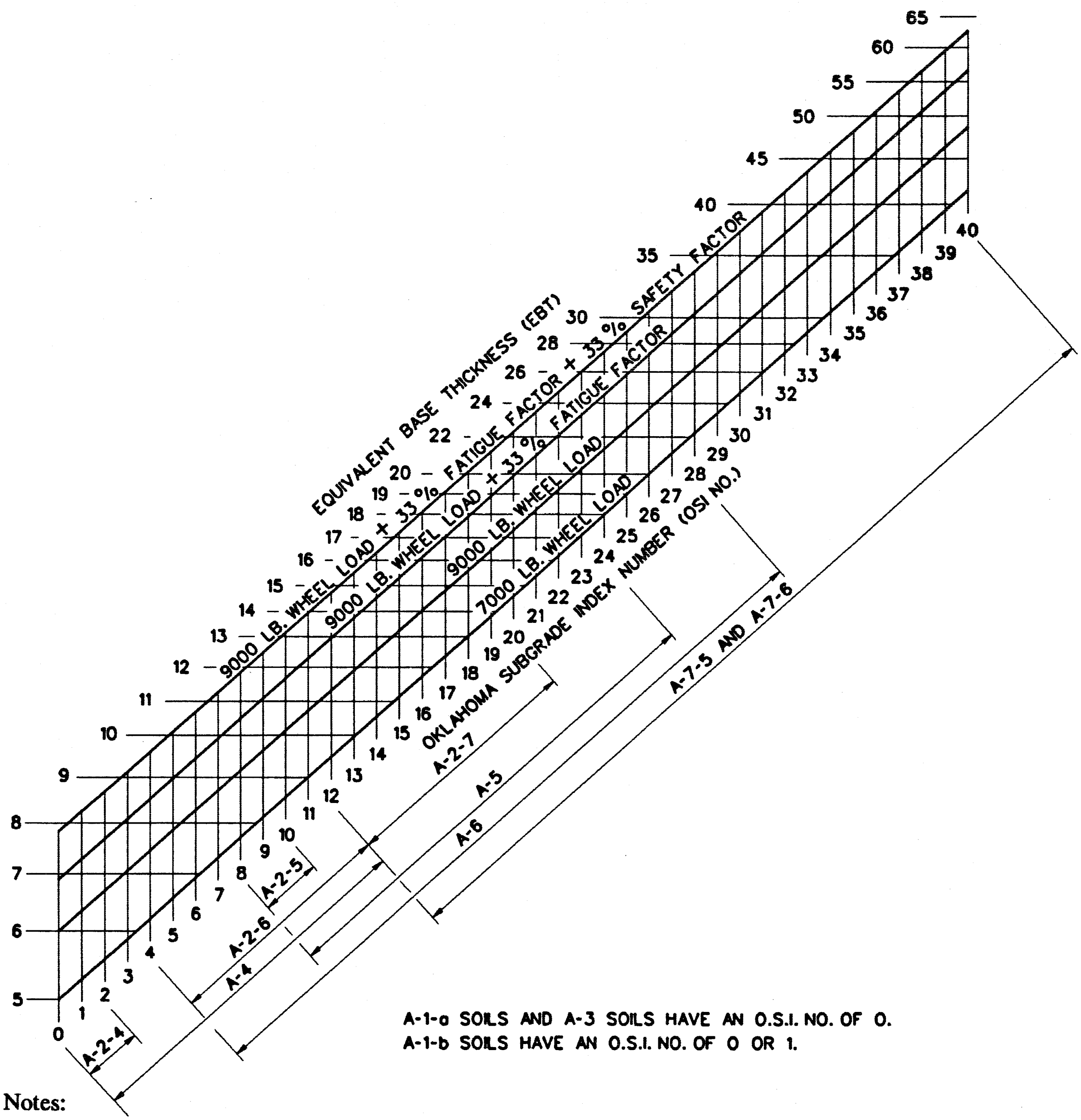
3. From the intersection of these lines, move horizontally to the left to determine the index number.

4. Follow a similar procedure (reading down and right) and determine the index number from the P.I. chart.

5. The sum of the index numbers determined in Step #'s 3 & 4 is the Oklahoma Subgrade Index (0.S.I.) Number.

CHART FOR DETERMINING OSI NUMBER

Figure 16.4A



Notes:

1. Determine the O.S.I. number of the soil from the O.S.I. number chart (Figure 16.4A).

2. Find the O.S.I. number along the bottom of this chart and follow the vertical line up to the appropriate wheel load line.

3. From the intersection of these lines, move horizontally to the left to determine the required equivalent base thickness.

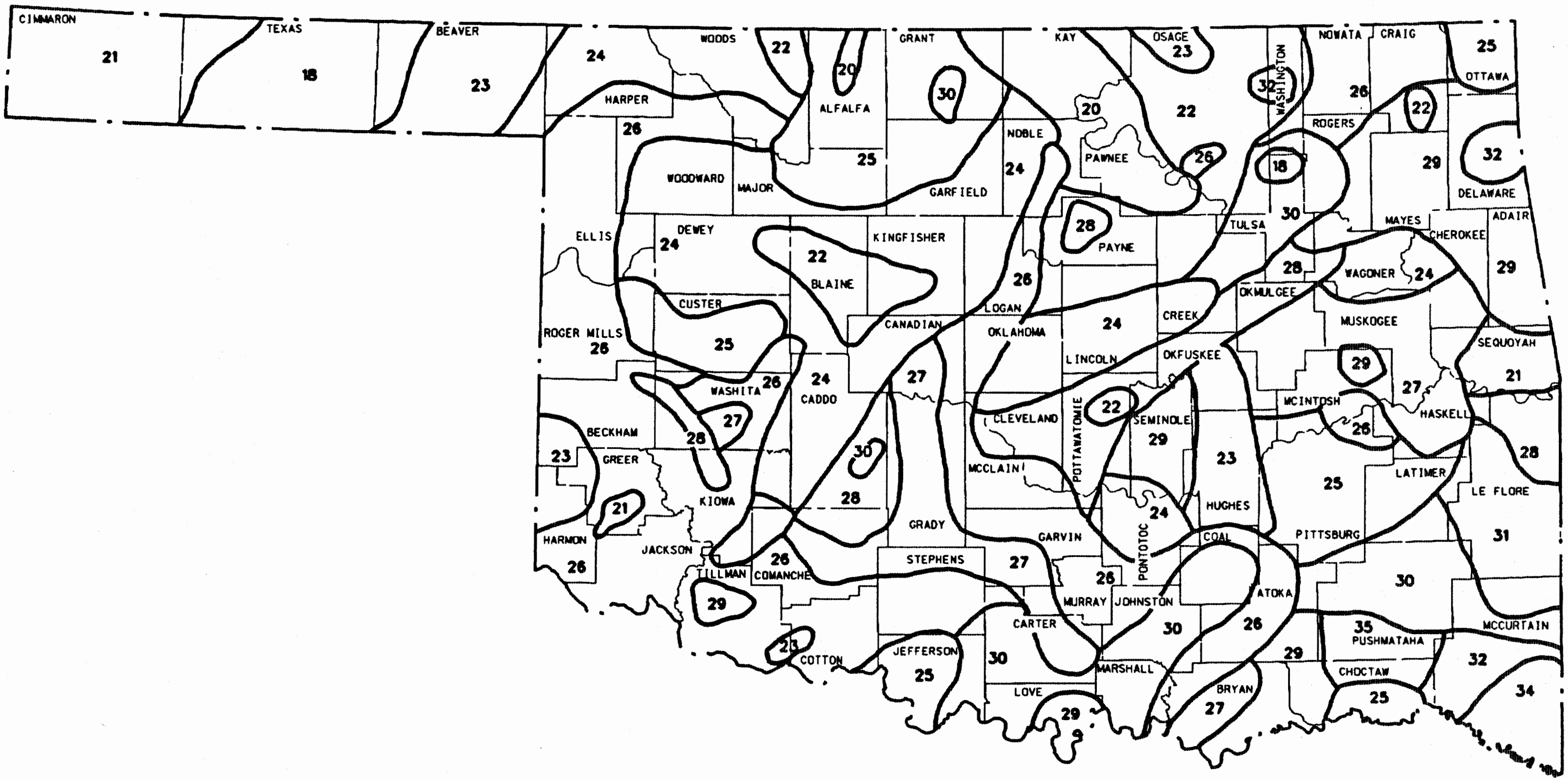
NOMOGRAPH FOR DETERMINING EQUIVALENT BASE THICKNESS (EBT)
(Unadjusted)

Figure 16.4B

Table 16.4B

TABLE FOR DETERMINING SHOULDER FACTOR

SHOULDER DATA			SHOULDER FACTOR
BASE MATERIAL	SURFACING TYPE	WIDTH (ft)	
Suitable soil	None	1-2	0
Suitable soil	4" gravel	1-2	1
Soil Asphalt	None or single bit.	1-2	2
Stabilized aggregate	None	1-2	2
Stabilized aggregate	Single bit.	1-2	3
Suitable soil	None	3-5	5
Suitable soil	4" gravel	3-5	7
Suitable soil	None	6+	10
Stabilized aggregate	None	3-5	13
Suitable soil	4" gravel	6+	14
Soil asphalt	None or single bit.	3-5	14
Stabilized aggregate	Single bit.	3-5	15
Stabilized aggregate	None	6+	18
Soil asphalt	None or single bit.	6+	19
Stabilized aggregate	Single bit.	6+	20
PCC curb and gutter			20



MAP FOR DETERMINING CLIMATE FACTOR
(Wet/Dry Cycles)

Figure 16.4C

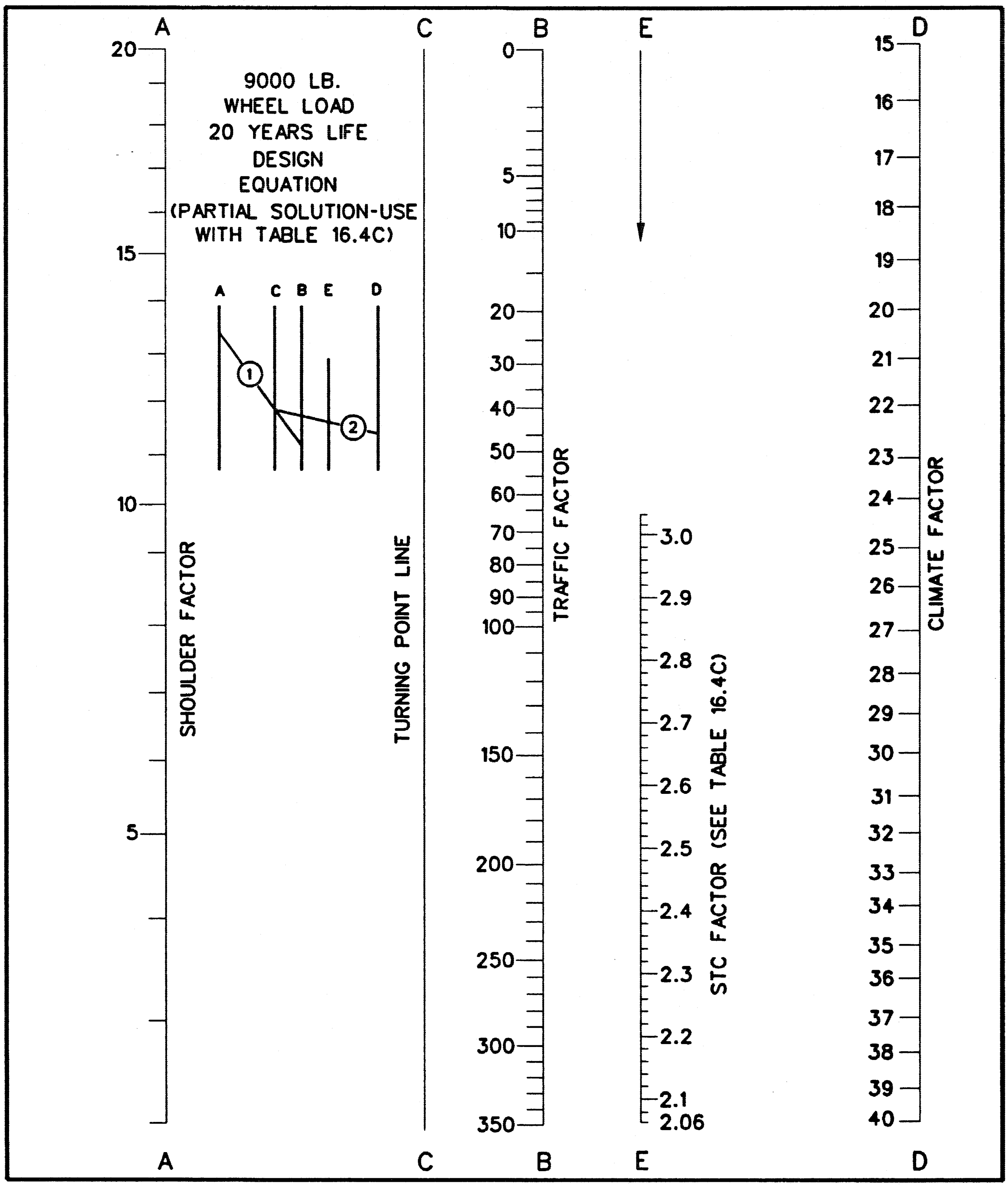
Table 16.4C

STC TABLE FOR DETERMINING EBT ADJUSTMENT FACTOR

EBT ADJUSTMENT FACTOR (1)	SHOULDER FACTOR (from Table 16.4B)									
	2	3	4	5	6	7	8	9	10	
+1	2.948	2.950	2.952	2.954	2.956	2.958	2.961	2.962	2.964	
+2	2.898	2.902	2.906	2.909	2.913	2.917	2.921	2.925	2.929	
+3	2.844	2.850	2.856	2.862	2.868	2.874	2.880	2.886	2.892	
+4	2.788	2.796	2.803	2.811	2.819	2.827	2.835	2.843	2.850	
+5	2.727	2.737	2.747	2.757	2.766	2.776	2.786	2.796	2.806	STC
+6	2.662	2.674	2.686	2.697	2.709	2.721	2.733	2.744	2.756	Factor
+7	2.592	2.606	2.619	2.633	2.647	2.660	2.674	2.688	2.702	(from
+8	2.516	2.531	2.547	2.563	2.578	2.594	2.610	2.626	2.641	Fig. 16.4D)
+9	2.433	2.450	2.468	2.486	2.503	2.521	2.539	2.556	2.574	
+10	2.341	2.361	2.380	2.400	2.420	2.439	2.459	2.478	2.498	
+11	2.240	2.261	2.283	2.305	2.366	2.348	2.369	2.391	2.412	
+12	2.126	2.150	2.173	2.197	2.220	2.244	2.267	2.291	2.314	
+13	1.996	2.022	2.047	2.073	2.098	2.124	2.149	2.175	2.200	
+14				1.928	1.956	1.983	2.011	2.038	2.066	

EBT ADJUSTMENT FACTOR (1)	SHOULDER FACTOR (from Table 16.4B)									
	11	12	13	14	15	16	17	18	19	20	
+1	2.966	2.967	2.969	2.971	2.973	2.975	2.977	2.979	2.981	2.983	
+2	2.933	2.937	2.941	2.945	2.949	2.953	2.957	2.960	2.964	2.968	
+3	2.897	2.903	2.909	2.915	2.921	2.927	2.933	2.939	2.944	2.950	
+4	2.858	2.866	2.874	2.882	2.890	2.898	2.905	2.913	2.921	2.929	
+5	2.815	2.825	2.835	2.845	2.855	2.864	2.874	2.884	2.894	2.904	STC
+6	2.768	2.780	2.791	2.803	2.815	2.827	2.838	2.850	2.862	2.874	Factor
+7	2.715	2.729	2.743	2.757	2.770	2.784	2.798	2.811	2.825	2.839	(from
+8	2.657	2.673	2.688	2.704	2.720	2.735	2.751	2.767	2.782	2.798	Fig.
+9	2.591	2.609	2.627	2.644	2.662	2.680	2.697	2.715	2.733	2.750	16.4D)
+10	2.518	2.537	2.557	2.576	2.596	2.616	2.635	2.655	2.674	2.694	
+11	2.434	2.455	2.477	2.499	2.520	2.542	2.563	2.585	2.606	2.628	
+12	2.338	2.361	2.385	2.408	2.432	2.455	2.479	2.502	2.526	2.549	
+13	2.226	2.251	2.277	2.302	2.328	2.353	2.379	2.404	2.430	2.455	
+14	2.093	2.121	2.148	2.175	2.203	2.230	2.258	2.285	2.313	2.340	
+15	1.932	1.962	1.991	2.021	2.050	2.079	2.109	2.138	2.168	2.197	
+16						1.888	1.919	1.950	1.982	2.013	

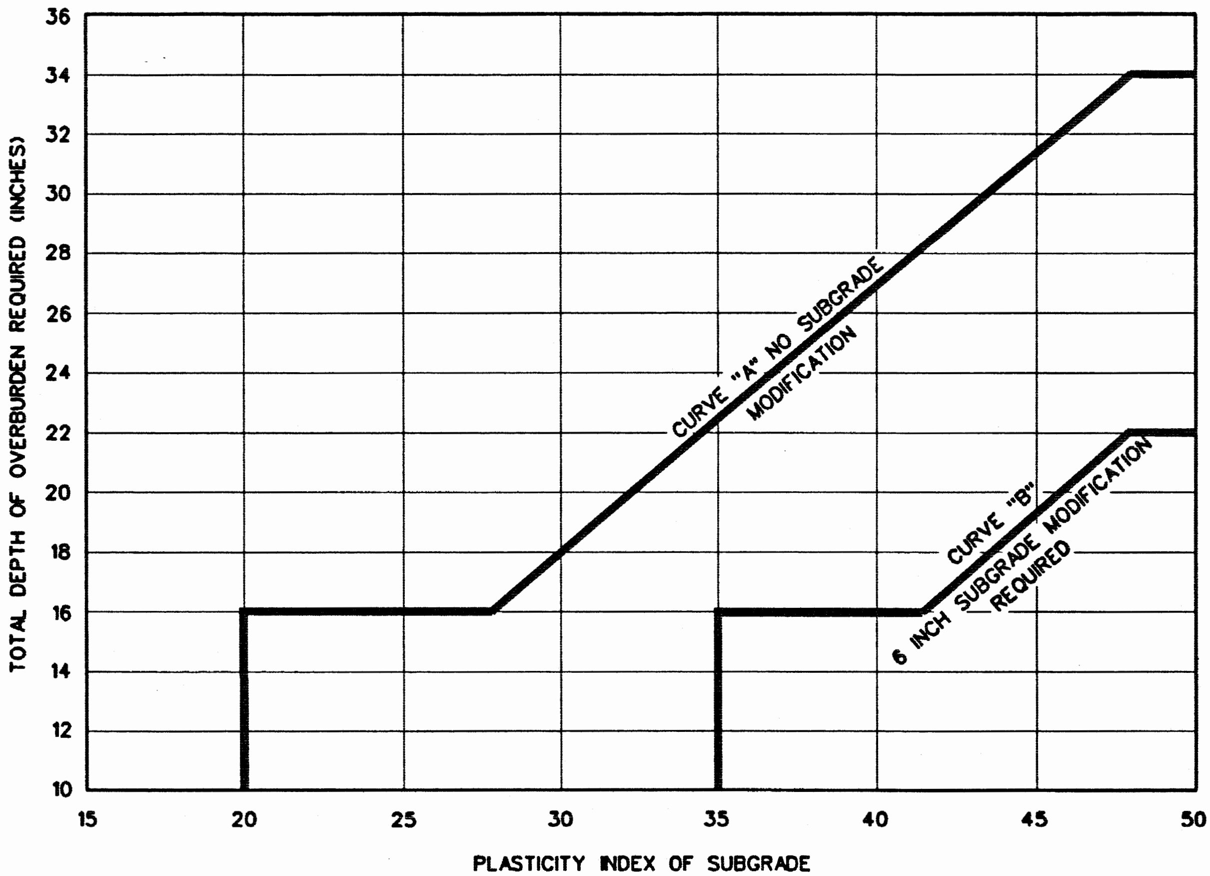
(1) To be added to the unadjusted EBT as derived in Section 16.4.3.2 for Design EBT.



Notes:
1. See Table 16.4C.
2. STC Factor is the product of the shoulder, traffic and climate factors.

NOMOGRAPH FOR DETERMINATION OF STC FACTOR

Figure 16.4D



Notes:

1. From PI of subgrade, determine total overburden required. Subtract total paving thickness from total overburden to determine thickness of additional material required to prevent damage.

2. CURVE "A" - Depth of low volume change material needed without subgrade modification.

3. CURVE "B" - Depth of low volume change material if the top 6 inches of the subgrade is modified.

OVERBURDEN CHART

Figure 16.4E

Table 16.4D

EQUIVALENT EBT VALUES

Thickness & Material	EBT Value
1" Select Borrow	½ inch
1" Fly Ash Modified/Lime Treated Subgrade	¾ inch
1" Aggregate Base	1 inch
1" Open-Graded Bituminous Base	1 inch
Reinforcing Fabric	1 inch
1" Hot-Mix, Hot-Laid Asphaltic Concrete (AC)	1½ inch

Table 16.4E

MINIMUM DESIGN THICKNESS
(Flexible Pavements)

TYPE OF FACILITY	MINIMUM DESIGNS
Interstate/High-Volume Primary	Surface: 4½" of A.C. Equivalent Base Course: 8"
Other Primary	Equivalent Base Course: 8"
Secondary/Local	Equivalent Base Course: 6"

16.4.3.4 Widening and Resurfacing

Where an existing pavement is to be widened and resurfaced, the thickness design for flexible pavement is based on the same factors as the OSI method contained herein.

In trenched sections, a minimum of six inches of the subgrade should be modified with lime or fly ash for soils with a plasticity index of 25 or more.

16.4.4 Other Design Considerations

Special soil conditions or soil-moisture conditions, either known to exist or found during the soil surveys or during construction, are generally minimized by one or more of the following methods:

1. use of drainable base,
2. subgrade modification,
3. increased thickness of base and/or subbase courses,
4. undercutting and backfilling of unstable areas, and/or
5. use of pipe underdrains.

When special situations arise, the engineers involved in design and construction of the project should meet and discuss the problems to determine a feasible solution.

16.5 EXISTING PAVEMENTS

The designer is presented with a different set of problems in seeking solutions for pavements already in service. He must inspect and evaluate the pavement to quantify the following:

1. condition assessment (serviceability rating),
2. rate of general deterioration,
3. rate of specific deterioration,
4. suitability of the pavement structure to continue to serve as designed,
5. suitability of the pavement structure to serve as a base course, and/or
6. unsuitability as a pavement structure (must be removed and rebuilt).

Answers to these questions will not be a direct output from the initial assessment. However, a rating scheme (preferably a part of a pavement management system) will over time yield data which can then be used to determine specific action plans for a site or a project rehabilitation.

16.5.1 Testing Methods

The pavement condition analysis will normally use visual observation and physical testing to gather data. An assessment form, augmented with photographs, which allow team members to give closely correlated opinions is recommended.

Physical testing falls into one of two categories:

1. destructive testing, or
2. non-destructive testing.

More discussion on the various tests is presented in the sections on asphaltic concrete (AC) surface distresses and Portland Cement Concrete (PCC) surface distresses. The correct classification of pavement type is important. The analysis must determine any previous maintenance and/or rehabilitation on the existing pavement to correctly classify the pavement structure. For example, a PCC pavement may have been routinely overlaid with a new AC wearing surface. Even if some or all the PCC pavement has been repaired, the designer may encounter the existing pavement at a point where the serviceability index is approaching terminal.

Both the PCC underlying pavement and AC overlay may be distressed. Classification of this combination pavement will be critical to the proper decision-making process being used for rehabilitation.

Existing pavements may be:

3. Portland Cement Concrete (PCC) pavement,
4. asphaltic concrete (AC) pavement, or
5. a combination.

Field testing methods include:

6. Benkleman Beam deflections;
7. falling weight deflectometer (FWD);
8. rut depth measurement;
9. coring pavement;
10. measurement of slab faulting;
11. slab section removal, including:
    a. logical slab section (between joints), or
    b. slab slice (in or over distress);
12. soil sample for subgrade analysis;
13. patch destruction for analysis;
14. removal of large extent of slab overlay to:

   a. check for underslab cracks,
   b. check for joint repairs, and/or
   c. check for overlay bond and quality of AC overlay; and

15. shoulder material (and subgrade) analysis.

With methods requiring removal of cores, slabs or overlays, the quality of repairs to the pavement are usually made, dependent upon the timing and estimated type of rehabilitation anticipated for the pavement under assessment.

Requests for field and/or laboratory analyses are usually made as a result of the scoping process, where the scoping team has had an opportunity to make pertinent input. A strategy for gathering samples and for the analysis process is also developed at this time.

The surface condition survey should have been completed or should be performed in the same time frame. The Materials Division crew should be guided to suspected hidden conditions or likely locations which may reveal either the extent or definite distress types or causes.

16.5.2 Distress Types

When making a surface condition survey, the designer may encounter recurrent distress conditions. Guidance on the classification of distress types may be found in Reference (4).

16.5.2.1 Asphaltic Concrete Surfaces

Distress types include:

1. crack types:

   a. alligator (fatigue) cracks,
   b. block cracks,
   c. edge cracking,
   d. longitudinal cracking,
   e. reflection cracks at joints, and
   f. transverse cracking;

2. patching and potholes:

   a. patch with patch deterioration, and
   b. potholes;

3. surface deformation:

   a. rutting,
   b. shoving, and
   c. corrugations;

4. surface distress or defects:

   a. bleeding,
   b. polished aggregates,
   c. raveling and weathering,
   d. delamination (separation of layers);

5. miscellaneous distresses:

   a. lane-to-shoulder drop-off,
   b. lane-to-shoulder separation, and
   c. water bleeding and pumping.

16.5.2.2 Portland Cement Concrete Surfaces

Distress types include:

1. jointed PCC surfaces:

   a. cracking:
      1) corner breaks,
      2) durability "D" cracking,
      3) longitudinal cracking, and
      4) transverse cracking;

   b. joint distress or deficiencies:

1) seal damage,
2) spalling at joints,
3) load-transfer device failures;

c. surface defects:
1) map cracking and scaling,
2) polished aggregates, and
3) popouts;

d. miscellaneous distresses:

1) blow-ups,

2) faulting at transverse joints (or cracks),

3) lane-to-shoulder dropoff,

4) lane-to-shoulder separation,

5) patch and patch deterioration,

6) water (and fines) pumping; and

2. continuously reinforced PCC surfaces:

a. cracking:
1) durability "D" cracking,
2) longitudinal cracking, and
3) abnormal transverse cracking;

b. surface defects:
1) map cracking and scaling,
2) polished aggregate, and
3) popouts;

c. miscellaneous distresses:

1) blowups,
2) construction joint deterioration,
3) lane-to-shoulder dropoff,
4) lane-to-shoulder separation,
5) terminal end joint distress,
6) patch deterioration,
7) punch outs,
8) spalling,
9) pumping, and
10) bridge end distress.

When the distress types have been cataloged and tied to a lane location map, the actual sampling may be finished. With observation, photo logging, test results and engineering experience, a strategy for rehabilitation can be developed.

16.5.3 Rehabilitation Methods

The term rehabilitation requires clarification by listing some methods and types which will be eligible for all types of pavement.

16.5.3.1 Rehabilitation Methods for Asphaltic Concrete Pavements

Rehabilitation methods include:

1. crack sealing,
2. full-depth patching or repair,
3. partial-depth patching or repair,
4. subdrainage,
5. cold milling,
6. stress-absorbing membranes,
7. asphaltic concrete overlay,
8. open-graded surface friction course,
9. fog sealing (shoulders),
10. slurry sealing/microsurfacing (shoulders and pavement), and
11. chip and seal.

16.5.3.2 Rehabilitation Methods for PCC

Rehabilitation methods include:

1. crack rehabilitation:

   a. reservoir preparation, and
   b. cleaning and sealing;

2. joint sealing;

3. full-depth patching or repair;

4. partial-depth patching/repair;

5. undersealing;

6. pressure-relief joints;

7. subdrainage (edge drains - horizontal drains);

8. grinding and/or milling;

9. concrete grooving;

10. overlays — PCC - bonded;

11. overlays — PCC - unbonded;

12. asphaltic concrete (AC) overlay;

13. cracking and seating plain PCC;

14. breaking and seating (reinforced PCC);

15. rubblizing; and

16. in-lay (reconstruct lanes inside existing shoulders).

16.6 OVERLAYS

Overlays to restore desirable wet weather friction characteristics, to re-establish a quality ride, and/or to increase the structural adequacy of a pavement are often a prudent solution. Repair and rehabilitation of the distress items prior to the overlay process is recommended when cost effective.

The evaluation process to establish the cost-effective ranking of a specific rehabilitation method is a prime ingredient in the success of an overlay. A slurry seal to rejuvenate the surface and reseal fatigue cracks in an asphaltic pavement may be less cost-effective (based upon a life-cycle cost analysis) than a stress membrane plus overlay.

Overlays generally should not increase the total thickness of AC wearing course layers to more than three inches. Two inches of AC wearing course is generally the maximum on routes with higher truck traffic counts, such as the Interstate system or high-volume expressways. Excessively thick AC wearing courses may tend to shove and rut.

16.7 PAVEMENT MANAGEMENT SYSTEMS (PMS)

The total record keeping, visual assessment, diagnosis engineering and prioritizing process are part of a Pavement Management System (PMS). Even with a wealth of data, the process must depend upon sound engineering judgment developed over years of experience. Over time, many rehabilitative methods will prove quite cost effective, and others will prove to be quite costly with little longevity. These experiences should be factored into any decision tree which ODOT uses to keep its roads repaired and maintained to a high level of reliability.

It is outside the scope of this chapter to delineate components of a PMS, but the need to document design, routine and rehabilitative maintenance and reconstruction projects for all roads is strongly recommended.

16.8 LIFE-CYCLE COSTING (LCC)

16.8.1 General

Life-Cycle Costing (LCC) is simply a method to determine, with economic assessment strategies, the true cost of a particular item, system, facility or parts of them over a chosen design life. The concept is not a new one, but the expertise needed and the decisions upon which to base an LCC system, require objective analysis and a dedication to continual review of the results.

Basically, the LCC system will allow an owner (ODOT) to choose a desirable "life" of a project, assign dollar values to various components of the project, expand those current dollars values to include inflation and other subjective impacts and arrive at an annual cost of a pavement system in today's dollars.

In this way, the two pavement systems, rigid and flexible, may be objectively evaluated, even though the initial capital cost and various maintenance requirements are known to be vastly different when taken over a finite span of years.

16.8.2 Input Parameters

Various data items may be included in the cost assessment. Some are vital and self-explanatory where others might be difficult to identify. The input parameters might include, but not be limited to:

1. life-cycle (total life or time to specific action);
2. initial capital cost;
3. known required preventive maintenance;
4. known required remedial maintenance;
5. anticipated (future development) impacts;
6. discount rate (interest rate minus inflation rate);
7. other inputs - initial construction and maintenance:
   a. detour requirements,
   b. increased road user costs,
   c. impacts on adjoining property,
   d. public input,
   e. property damage and injury accidents, and
   f. loss of life.

Life-cycle costing is fundamentally intended to determine the true expenses associated with design alternatives over their design life. The LCC system developed over time and refined could be used to determine cost-benefit values for items within the original study. Documentation, objectivity and sensitivity assessment are key components of LCC systems. See Reference (5) for additional discussion.

16.9 REFERENCES

1. Oklahoma Subgrade Index Method for Pavement Design, ODOT (many publications, various dates).

2. *Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials (AASHTO), Washington, DC, 1986.

3. *Soils in Construction*, 3rd Edition, W. L. Schroeder, pub. by John Wiley & Sons, New York, NY, 1987.

4. *Distress Identification Manual for the Long-Term Pavement Performance Studies*, SHRP, 1991.

5. *Life-Cycle Costing for Design Professionals*, Kirk & Dell'isola, McGraw-Hill Book Company, New York, NY, 1981.

6. *State of Oklahoma County Roads Design Guidelines Manual*, Oklahoma Department of Transportation and Association of County Commissioners of Oklahoma, June 1991.

Chapter Seventeen

Special Design Elements

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Chapter Seventeen

SPECIAL DESIGN ELEMENTS

17.1 PARKING

17.1.1 On-Street Parking

The designer should evaluate the following when providing on-street parking along 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 and may produce undesirable traffic operations or increase the accident potential.

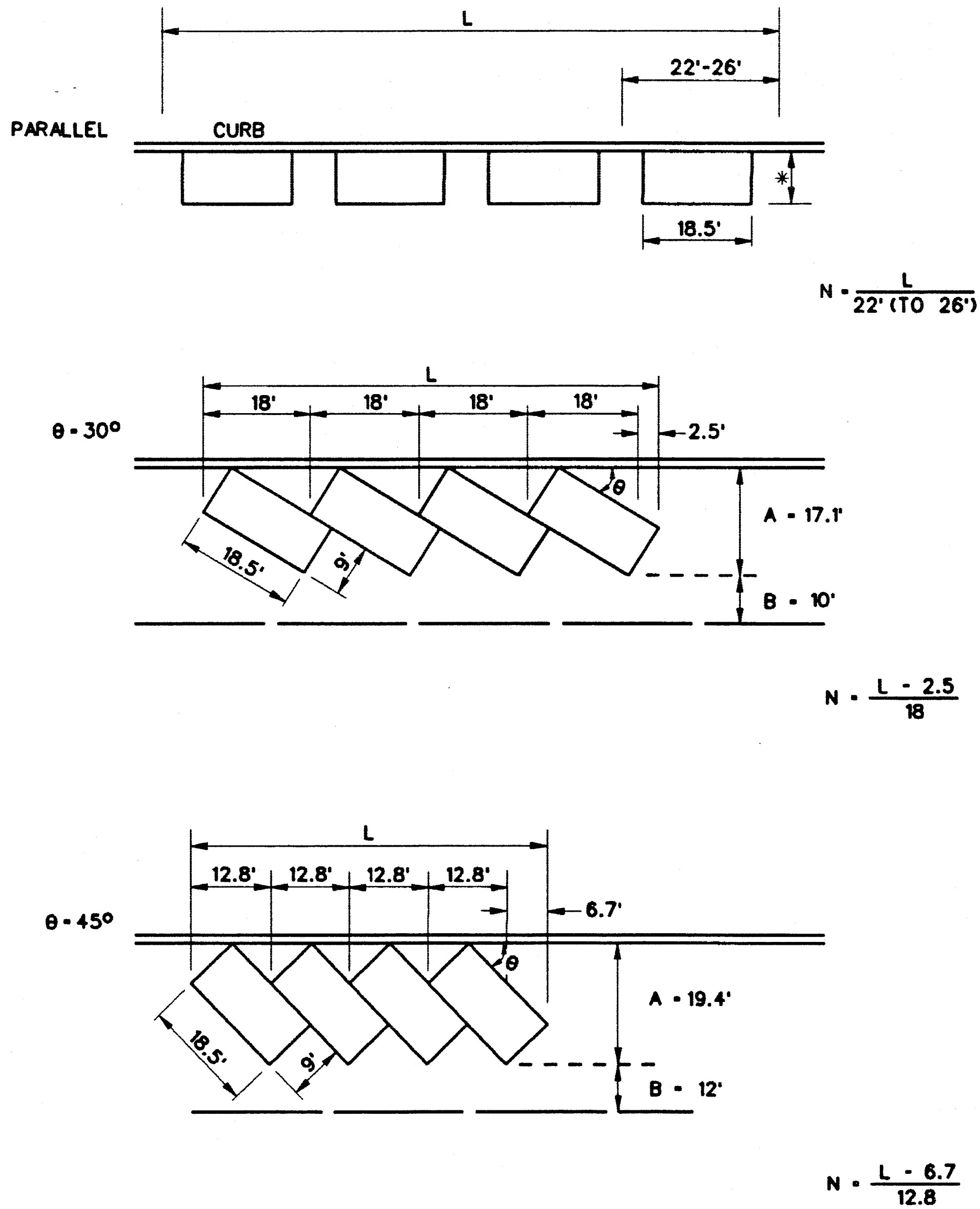
2. Configuration. The two basic types of on-street parking are parallel and angle parking. These are illustrated in Figure 17.1A. Parallel parking is the preferred arrangement when street space is limited and traffic capacity is a major factor. Angle parking provides more spaces per linear foot than parallel parking, but 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 travel lane 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.

   When selecting the parking configuration, the designer should evaluate the operational consequences of the selection. In particular, the designer should consider the backing maneuver required by angle parking. As indicated in Figure 17.1A, the parked car will require a certain distance "B" to back out of its stall. Whether or not this is a reasonably safe maneuver will depend upon the number of lanes in each direction, lane widths, operating speeds, traffic volumes during peak hours, parking demand and turnover rate of parked vehicles.

3. Stall Dimensions. Figure 17.1A 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.

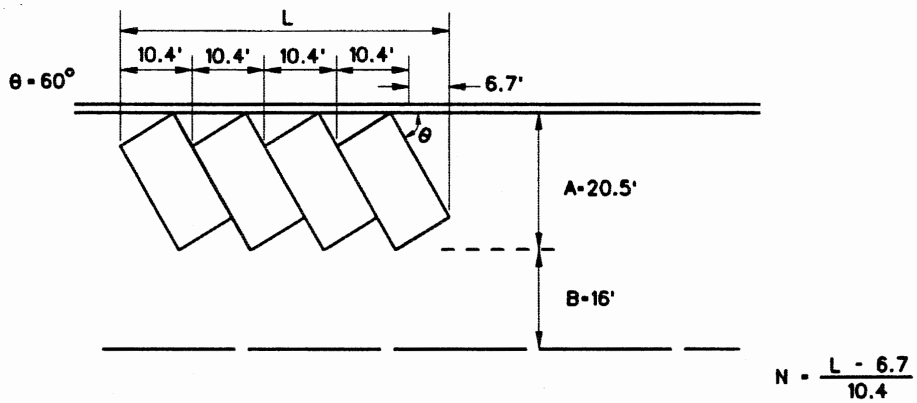
   The Tables in Chapter Twelve and Thirteen provide parking lane widths for parallel parking. For angle parking, desirably, the parking lane width will be a combination of "A" and "B" as shown in Figure 17.1A exclusive of the through travel lane. However, in restricted areas a portion of the "B" dimension may be required for the through travel lane, thereby reducing the actual parking lane widths.

   Section 17.4 provides information on the parking stall dimensions for handicapped parking spaces.



CURB PARKING CONFIGURATIONS

Figure 17.1A



Key:
- L = given curb length with parking spaces
- 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.

* See Tables in Chapters Twelve and Thirteen for parking lane widths.

Source (3)

CURB PARKING CONFIGURATIONS
(Continued)

Figure 17.1A

4. Cross Slope. The cross slope of the parking lane will typically be the same as that of the adjacent travel lane.

5. Location. Parking spaces should be located as follows:

    a. Parking should be prohibited within 20 ft of any crosswalk.

    b. Parking desirably should not be allowed within 20-30 ft of the beginning of the curb radius at intersections.

    c. Parking should be prohibited within 10-15 ft of the beginning of the curb radius at mid-block driveway entrances.

    d. Parking should be prohibited within 50 ft of the nearest rail of a railroad/highway crossing.

    e. Parking should be prohibited from areas designated by local traffic and enforcement regulations (e.g., near fire hydrants, loading zones, bus stops). See local ordinances for additional information on parking restrictions.

    f. Parking should be prohibited near bus stops (see Section 17.2).

17.1.2 Off-Street Parking

A proposed highway project may incorporate some form of off-street parking. Typical applications may include:

1. providing off-street parking to replace on-street parking which will be removed as part of a proposed project;
2. the construction of a park-and-ride lot for commuters; or
3. the construction of a new rest area or improvement to an existing rest area.

The following presents criteria specifically for off-street parking lots. Section 17.5 discusses rest areas.

17.1.2.1 Location (Park-and-Ride Lots)

During the planning stages for transportation facilities, the Planning Division may determine the general location for a park-and-ride lots. However, in general, the designer will be responsible for determining the location of these facilities. The Field Division Offices may provide location recommendations based on actual field conditions, or current usage where commuters are already parking.

Park-and-ride lots should be located at strategic points where transfers can conveniently be made from auto to carpooling or transit modes. Considerations that will affect the location of the parking facility are:

1. Accessibility. The lot should be convenient to residential areas, bus and rail transit routes, and the major highways used by commuters.

2. Congestion. The location should precede any points of congestion on the major commuting highway to maximize its benefits.

3. Connections. There should be sufficient capacity on connections between the lot and the major commuting highway.

4. Design. The site location must be compatible with the design and construction of the lot. The designer

must consider property costs, terrain, drainage, subgrade soil conditions, and available space in relation to the required lot size, visibility and access.

5. Land Use. The location of the lot should be consistent with the present and future adjacent land use. Visual and other impacts on surrounding areas should be considered. Where necessary, site sizing and design should allow for buffer landscaping to minimize the visual impact.

17.1.2.2 Design Elements

The following elements should be considered during the design of a parking facility:

1. Entrances and Exits. Entrances and exits should be located to have the least disruption to existing traffic (e.g., away from intersections) and still provide the maximum storage space. Combined entrances and exits should preferably be as close to mid-block as practical. Where entrances and exits are separated, the entrance should be on the "upstream" side of the traffic flow nearest the lot and the exit on the "downstream" side.

   All entrances and exits should be designed as commercial driveways in accordance to the design criteria presented in Chapter Nine. The typical design vehicle will be a BUS or SU vehicle.

2. Accessibility for Handicapped Individuals. Section 17.4 discusses the accessibility criteria, which also apply to off-street parking lots.

3. Parking Stall Dimensions. Parking dimensions vary with the angle at which the stall is arranged relative to the aisle. Figure 17.1B provides the design dimensions for 9-ft x 18.5-ft parking stalls based on one-way circulation and angle parking. Typical stall widths (measured perpendicular to the vehicle when parked) range from 8.5 to 9.5 ft. In attendant parking facilities, attendants can park standard-size cars in spaces as narrow as 8.0 ft. However, the minimum stall size recommendation is 8.5 ft for self-parking of long-term duration. For higher-turnover self-parking, a stall width of 9.0 ft is recommended. Stall widths at supermarkets and other similar parking facilities, where large packages are prevalent, should desirably be 9.5 or even 10 ft in width.

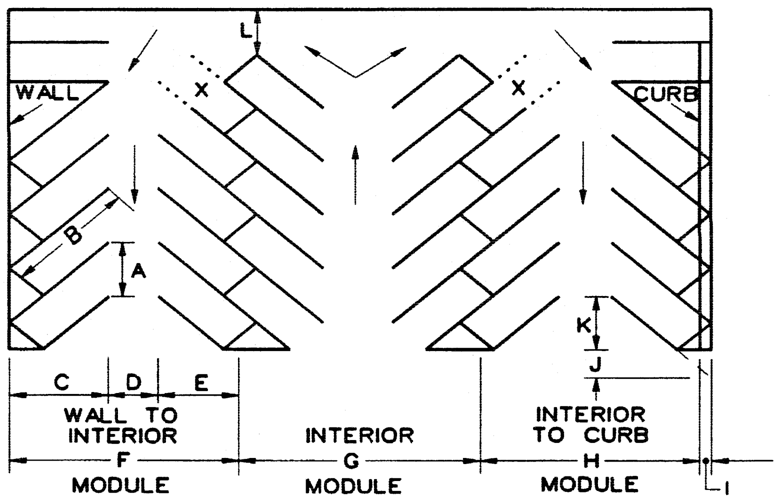
4. Sidewalk Dimensions. All sidewalks should be at least 4-ft wide. In loading areas, the width should be at least 12 ft. The accessibility criteria for the handicapped must be met (see Section 17.4).

5. Gradients. To provide proper drainage, the minimum gradient on the parking lot should be 0.5%. As a maximum, the gradient should not exceed 7%.

   Desirably, the lot should be designed directing the runoff into existing drainage systems. If water impoundment cannot be avoided along pedestrian routes, bicycle routes and standing areas, drop inlets and underground drainage should be provided. In parking areas, drainage should be designed to avoid standing water. Chapter Fifteen provides additional information for the proper hydraulic design of drainage elements.

6. Lighting. Desirably, the lot should be lighted for pedestrian safety and lot security. Chapter Fourteen provides information on the design of lighting.

7. Shelters. Pedestrian shelters are desirable where loading areas for buses and trains



Parking Layout Dimension (in ft) for 9 ft x 18.5 ft Stalls at Various Lengths

Dimension	On Diagram	Angle 45°	60°	75°	90°
Stall width, parallel to aisle	A	12.7	10.4	9.3	9.0
Stall length of line	B	27.5	23.7	20.9	18.5
Stall depth to wall	C	19.5	20.5	20.0	18.5
Aisle width between stall lines	D	12.0	16.0	23.0	26.0
Stall depth, interior	E	16.5	18.5	19.0	18.5
Module, wall to interior	F	48.0	55.0	62.0	63.0
Module, interior	G	45.0	53.0	61.0	63.0
Module, interior to curb face	H	46.0	52.5	59.5	60.5
Bumper overhang (typical)	I	2.0	2.3	2.5	2.5
Offset	J	6.4	2.6	0.6	0.0
Setback	K	13.1	9.3	4.8	0.0
Cross aisle, one-way	L	14.0	14.0	14.0	14.0
Cross aisle, two-way	—	24.0	24.0	24.0	24.0

1. See Section 17.4 for criteria on the number and dimensions of parking spaces for handicapped individuals.

2. If a special section is designated for subcompact vehicles, these stalls can be 8' x 15' for a 90° angle.

3. Stalls should be wider for commercial parking.

4. The designer should consider bumper overhang when placing lighting, railing,etc. Therefore, these appurtenances should be placed beyond dimension "I" in the figure.

Source (2)

PARKING STALL DIMENSIONS

Figure 17.1B

are provided. Their inclusion will be determined on a case-by-case basis. The shelter should provide 3 to 5 sq ft of covered area per person. As a minimum, the shelter should provide lighting, benches and trash receptacles. Other amenities that should be considered are routing information signs and a telephone.

8. Fencing. The need for fencing around a parking lot will be determined on a case-by-case basis.

9. Landscaping. In some locations, landscaping may be provided to minimize the visual impact of the parking lot by providing a buffer zone around the perimeter of the lot or to improve the aesthetics of the lot itself. Desirably, space will be provided for a 10- to 20-ft buffer zone around the lot to accommodate vegetation screens. Also, traffic islands and parking lot separators provide suitable locations for shrubs and trees. Landscaping should include low maintenance vegetation which does not cause visibility or security problems. For information on appropriate vegetation selections, the designer should contact the Roadside Development Branch.

17.2 BUS STOPS AND BUS TURNOUTS

17.2.1 Location

17.2.1.1 Bus Stops

If local bus routes are located on an urban or suburban highway, the designer should consider the impact on normal traffic operations. The stop-and-go pattern of local buses will disrupt traffic flow, but certain measures can minimize this disruption. The location of bus stops is particularly important. These are determined not only by convenience to patrons but also by the design and operational characteristics of the highway and the roadside environment. If the bus must make a left-turn, for example, a bus stop should not be located in the block preceding the left turn. Common bus stop locations are shown in Figure 17.2A.

Some considerations in selecting an appropriate bus-stop location are listed below:

1. Far-Side Stops. The far side of at-grade intersections is generally superior to near-side or mid-block bus stops. Far-side stops produce less impediment to through and right-turning traffic; they do not interfere as much with corner sight distance; and they lend themselves better to bus turnouts.

2. Mid-Block Stops. Mid-block bus stops may be advantageous where the distance between intersections is large or where there is a fairly heavy and continuous transit demand throughout the block. They may be desirable if there is a high bus stop demand located mid-block. Mid-block bus stops may also be considered when right turns at an intersection are high (250 in peak hour) and far-side stops are not practical.

3. Near-Side Stops. Near-side stops allow easier vehicle re-entry into the traffic stream where curb parking is allowed, and they can increase street capacity. At intersections where there is a high volume of right-turning vehicles, near-side stops can result in traffic conflicts and should be avoided. However, near-side stops must be used where the bus will make a right turn at the intersection.

17.2.1.2 Bus Turnouts

Interference between buses and other traffic can be reduced significantly by providing bus turnouts. Turnouts help remove stopped buses from the through lanes and provide a well-defined user area for bus stops. Turnouts should be considered under the following conditions:

1. The street provides arterial service with high traffic speeds and volumes and high-volume bus patronage.

2. Right-of-way width is sufficient to prevent adverse impact on sidewalk pedestrian movements.

3. Where curb parking is permitted, but is prohibited during peak hours.

4. During peak-hour traffic, there are at least 500 vehicles per hour in the curb lane.

5. Bus volumes do not justify an exclusive bus lane, but there are at least 100 buses per day and at least 10 to 15 buses during the peak hour.

6. At locations where specially equipped buses are used to load and unload handicapped individuals.

17.2.2 Design

17.2.2.1 Bus Stops

Figure 17.2A provides the recommended distances for the prohibition of on-street parking near bus stops. An additional 45 ft of length should be provided for each additional bus expected to stop simultaneously at any given bus stop area. This allows for the length of the extra bus (40 ft) plus 5 ft between buses.

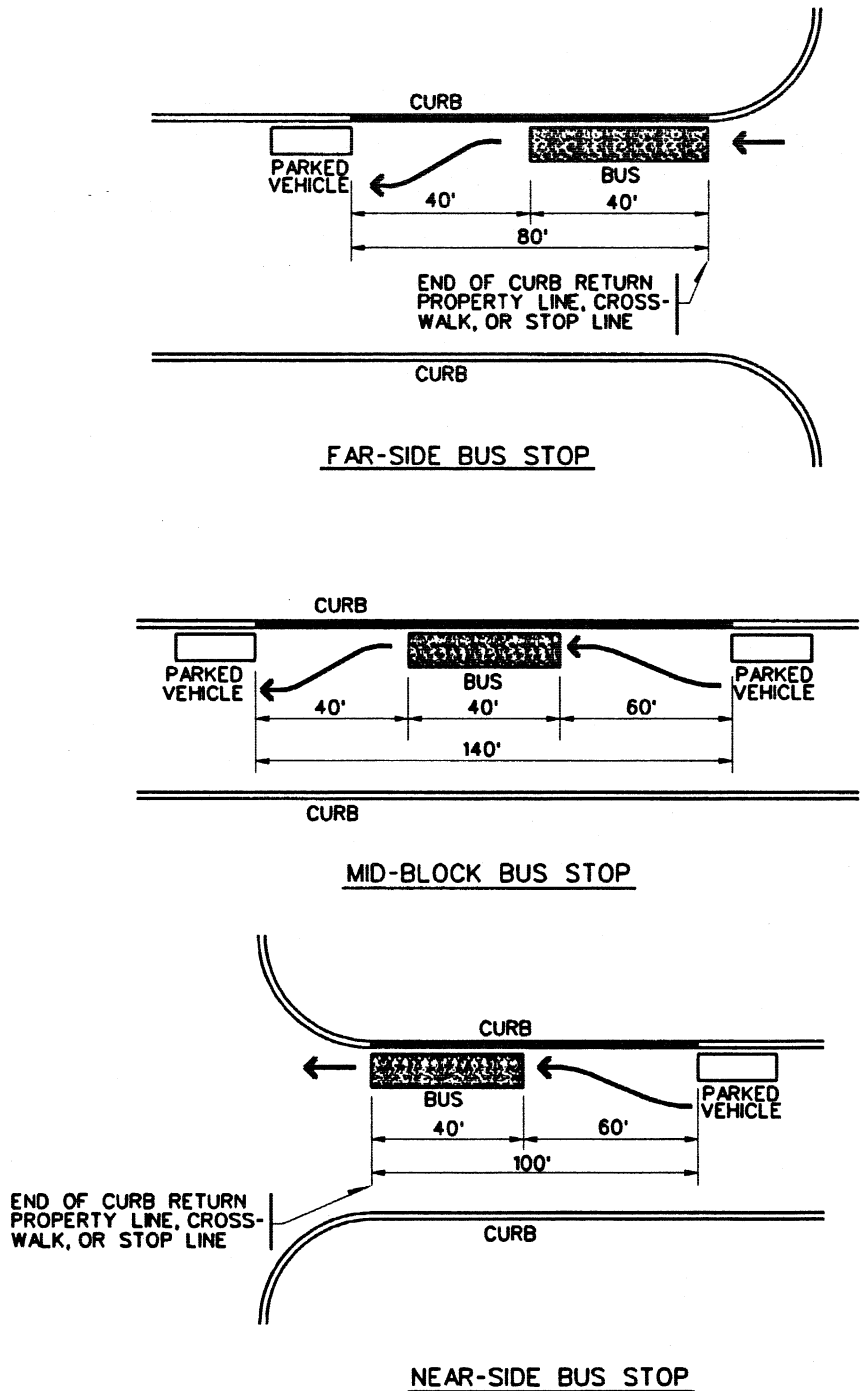
17.2.2.2 Bus Turnouts

The following design criteria will apply:

1. The desirable width is 12 ft; the minimum width is 10 ft.

2. The full-width area of the turnout should be at least 50-ft long. For a two-bus turnout, add 45 ft.

3. Figure 17.2B illustrates the design details for bus turnouts. In the transition areas, an entering taper no steeper than 5:1 and a re-entry taper no steeper than 3:1 should be provided. As an alternative, short horizontal curves (100-ft radius) may be used on the entry end and 50- to 100-ft curves on the re-entry end. When a turnout is located at a far-side or near-side location, the cross street area can be assumed to fulfill the need for the exit or entry area, whichever applies.



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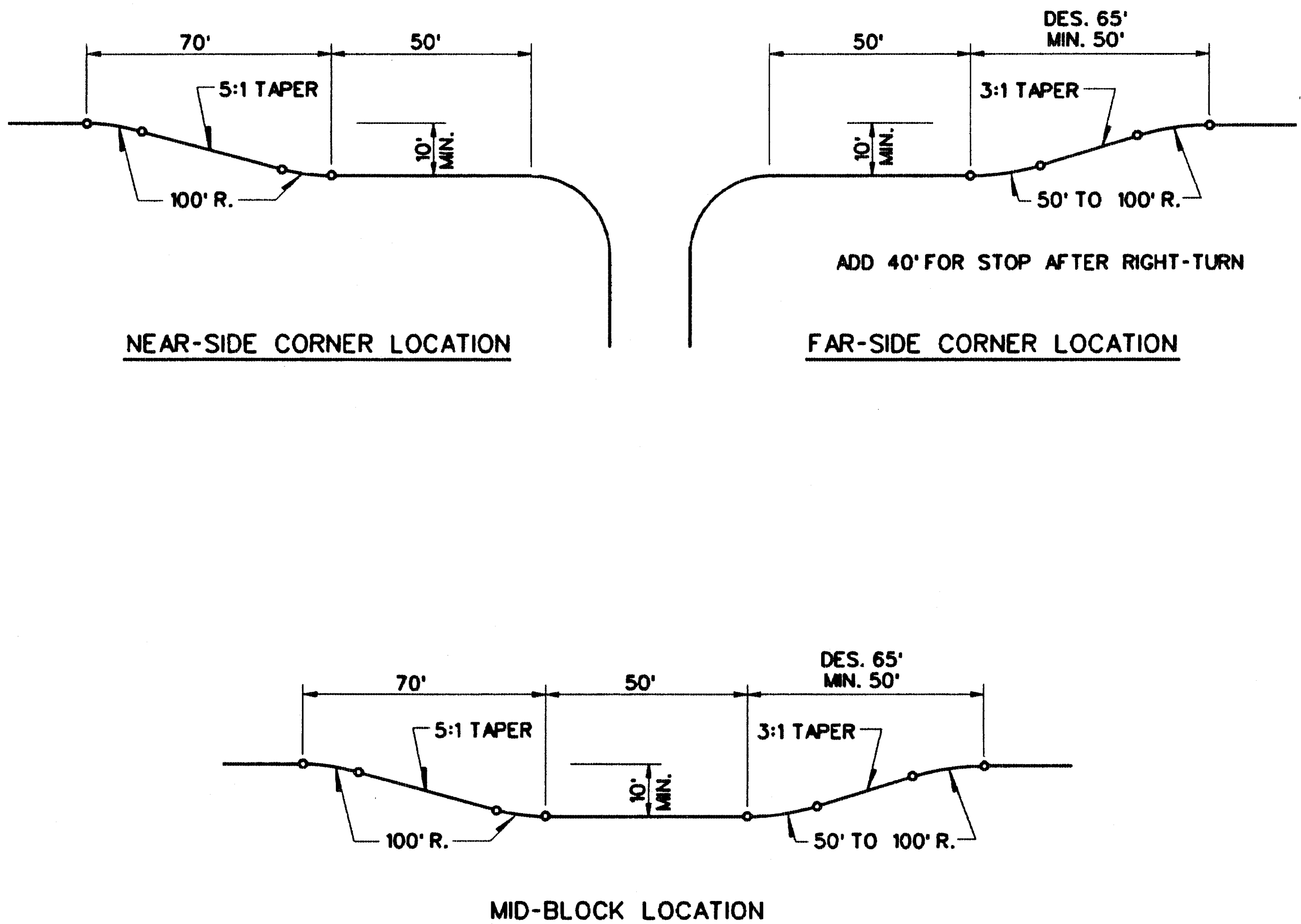
ON-STREET BUS STOPS

Figure 17.2A



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BUS TURNOUT DESIGNS

Figure 17.2B

17.3 BIKEWAYS

The majority of bicycling takes place on public roads with no dedicated space for bicyclists. Bicyclists can be expected to ride on almost all roadways. Sometimes they use sidewalks as joint bicycle and pedestrian facilities, unless such usage is prohibited by local ordinance. This section primarily provides information on the development of new facilities to enhance and encourage safe bicycle travel.

17.3.1 Bikeway Classifications

The following bikeway definitions will apply:

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 are to 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 such facility is specifically designated as a bikeway.

3. 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 assume different forms, as conditions warrant. They may be two-direction, multilane facilities or, where the path would parallel a roadway with limited right-of-way, a single lane on both sides of the road.

4. 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. Bicycle lanes may also assume varying forms but are typically included in one of the following categories:

   a. bicycle lane between parking lane and travel lane, or

   b. bicycle lane between roadway edge and travel lane, where parking is prohibited.

17.3.2 Warrants

Each type of facility has its own merits and disadvantages. Care must be exercised in choosing the appropriate type of facility for a given site. The following discussion and warrants are offered as a guide in making decisions regarding bikeway type. The use of definite, numerical limits for warrants should be avoided, and placing excess emphasis on any single warrant should be avoided. Each route is unique and must be evaluated individually.

17.3.2.1 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 warrants may be used to 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 volume;

6. absence of suitable alternative routes;

7. demonstration that the facility would serve a definite purpose; and

8. reasonable indication that the bicycle path would be the safest and most economical method of providing a bicycle facility.

17.3.2.2 Bicycle Lanes

The occupation of a portion of a roadway by a bicycle lane implies a reasonable degree of safety for the cyclist. Conditions must be generally less severe than those which warrant a bicycle path. The use of a bicycle lane is normally restricted to bicycles, but exceptions may be made. 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 a fraction of the expense associated with bicycle paths. Another advantage of bicycle lanes is the relatively minor land requirements. They can be installed in many areas where the construction of paths would be next to impossible. In practice, bicycle lanes, although not ideal, may be the most practical means of developing bikeways.

The following warrants may be used to 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 on the surroundings;

6. demonstration that the facility would serve a definite purpose; and

7. indication that the bicycle lane would be the safest and only feasible method of providing a bicycle facility.

17.3.2.3 Shared Roadway

Mixing bicycles and motor vehicles should generally be avoided. There are instances, however, where this is a 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 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 warrants may be used to 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. indication that the shared roadway would be the safest and only feasible method of providing a bicycle facility; and

8. a higher grade facility not warranted.

17.3.3 Bikeway Design Elements

17.3.3.1 Widths and Clearances

The widths and clearances required for the proper and safe operation of a bikeway are important. The following lists the various width requirements based on the type of bikeway facility:

1. Shared Roadways. There usually is no additional width provided with a shared roadway facility. Desirably, a smooth paved shoulder will be present.

2. Bicycle Path. A desirable paved width of 10 ft should be provided on two-directional bicycle paths. An 8-ft bicycle path may be used if the bicycle volume is expected to be low and where the pedestrian use of the facility is expected to be minimal. Where there is expected to be a significant number of pedestrians (e.g., joggers) or where bicyclist will be likely to ride two abreast, it is desirable to increase the width to 12 ft.

   If it is determined that the bicycle path will be one directional, then the minimum paved width may be 5 ft.

   To provide lateral clearance from trees, poles, walls, fences, guardrails or other hazards, a minimum 2-ft graded area should be maintained adjacent to both sides of the pavement area.

3. Bicycle Lane. The width of a bicycle lane depends on whether curbing is used and/or parking is allowed.

   If a parking lane is present, a 5-ft width should be provided between the parking lane and vehicular travel lane. Where parking is permitted but a parking lane is not provided, the combination lane, intended for both motor vehicle parking and bicycle use, should be a minimum of 12-ft wide.

   If parking is not allowed, the minimum bicycle lane width is 4 ft. If curbing is present, a minimum width of 5 ft should be provided between the curb face and the travel lane.

17.3.3.2 Design Speed

The speed of a bicyclist is dependent upon the type of bicycle and equipment, slope, surface conditions, air resistance, wind velocity and the physical condition of the cyclist. Bicycles have the capability of traveling at very high speeds, but this is not the normal case. A cyclist's average speed is in the vicinity of 10 to 11 mph, with a normal traveling range between 7 and 15 mph.

For design purposes, an overall minimum design speed of 20 mph is established for paved surfaces. On unpaved surfaces the minimum design speed is 15 mph. Greater design speeds should be used where conditions, primarily governed by slope, indicate a need. A design speed of 20 mph should be used for grades between +3% and -3%. For grades steeper than 3%, the design speed should be 30 mph or higher if the slope is very long. For climbing grades greater than

3%, a minimum of 15 or 20 mph should be sufficient.

17.3.3.3 Grade

Grades on bicycle paths should be kept to a minimum, especially on long inclines. Grades greater than 5% are undesirable because the ascents are difficult for many bicyclists to climb and the descents cause some bicyclists to exceed the speeds at which they are competent. Where terrain dictates, grades over 5% and less than 500-ft long are acceptable when a higher design speed is used and additional bicycle path width is provided. Grades steeper than 3% may not be practical for bicycle paths with crushed stone surfaces.

This discussion is generally applicable only to bicycle paths. The slopes for other types of facilities are determined by the roadway of which they are part.

17.3.3.4 Sight Distance

The safe operation of any bicycle facility requires that the design of the facility provide for adequate stopping sight distance. The sight distance required is determined by the design speed and gradient. Sight distance design values may be calculated much the same as for motor vehicles.

Stopping sight distance should be applied to horizontal and vertical curves. Figure 17.3A may be used to determine the appropriate value. This graph was computed for paved bikeways. A smaller coefficient of friction is used for unpaved bikeways, resulting in longer sight distance requirements. The height of eye is 4.5 ft and the height of object is assumed to be 0 ft. This is to recognize that hazards to bicycle travel are at pavement level.

In general, sight distance considerations are necessary only in the case of bicycle paths. Because shared roadways and bicycle lanes occupy a portion of a facility designed for motor vehicles, sight distances are normally more than adequate. There are exceptions, however, and any questionable situations should be inspected when locating such a facility.

17.3.3.5 Horizontal Curves and Superelevation

Simple, circular curves are adequate for bicycle facilities. The comfort and safety of a horizontal change of direction is determined, to a large extent, by the size of the radius of curvature. A very sharp curve may result in an upset or loss of control. A wide curve presents no riding difficulties.

Radius of curvature should be directly proportional to the anticipated operating speed. It is therefore chosen as a function of the design speed of a facility.

Superelevation is inversely proportional to the minimum radius of curvature and should be employed in the construction of bicycle paths. The minimum value should be 0.02 ft per ft to ensure adequate drainage. This value will be adequate for most conditions and will simplify construction. Superelevation should not exceed 0.05 ft per ft.

The point-mass equation for horizontal curves on highways also applies to bikeways:

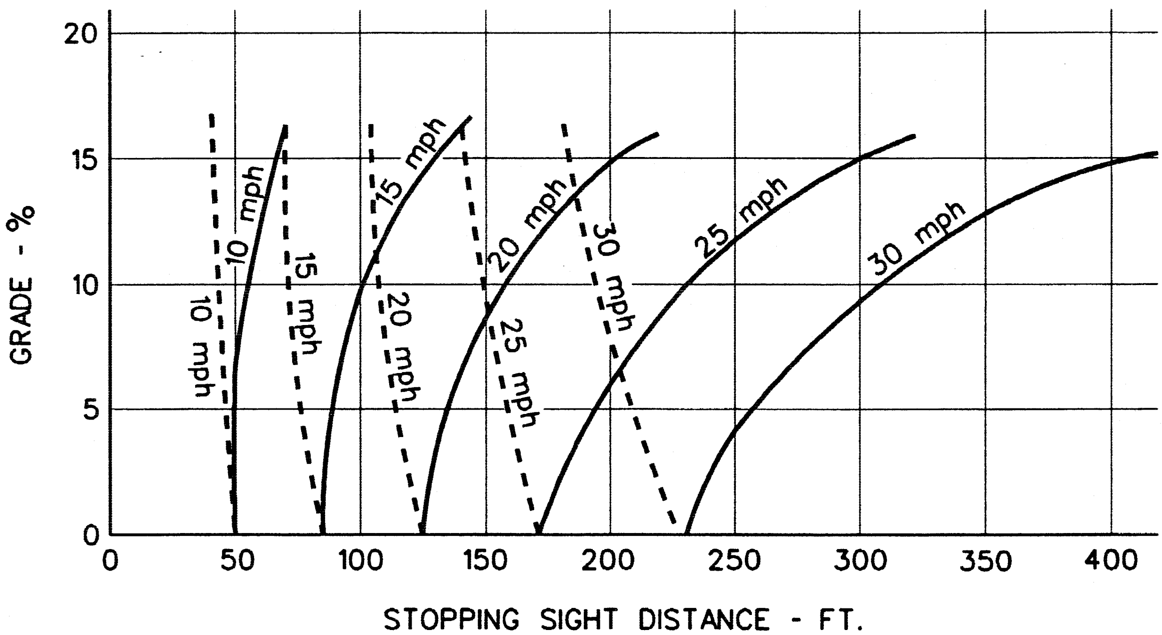
$$R = \frac{V^2}{15(e+f)}$$

where: R = minimum radius, ft

V = design speed, mph

e = superelevation rate

f = coefficient of friction



$$S = \frac{V^2}{30(f \pm G)} + 3.67V$$

DESCEND (−G) ———
ASCEND (+G) - - - - -

WHERE: S = STOPPING SIGHT DISTANCE, FT.
V = VELOCITY, MPH
f = COEFFICIENT OF FRICTION (USE 0.25)
G = GRADE FT.\FT. (RISE/RUN)

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STOPPING SIGHT DISTANCES

Figure 17.3A

Table 17.3A
MINIMUM CURVE RADII FOR BIKEWAYS

Design Speed (mph)	e	f	R (ft)
15	.02	.30	50
20	.02	.27	95
25	.02	.25	155
30	.02	.22	250

Table 17.3A provides criteria for minimum radii for various design speeds. The table assumes a superelevation rate of 0.02; if a higher rate is used, the designer should use the equation directly. The friction values have been extrapolated from those used in highway design.

This discussion of horizontal curves is not applicable to bicycle lanes or shared roadways. Their geometry is determined by the roadway which is designed for motor vehicles. Highway curves will be more than adequate for bicycle use.

17.3.3.6 Vertical Curves

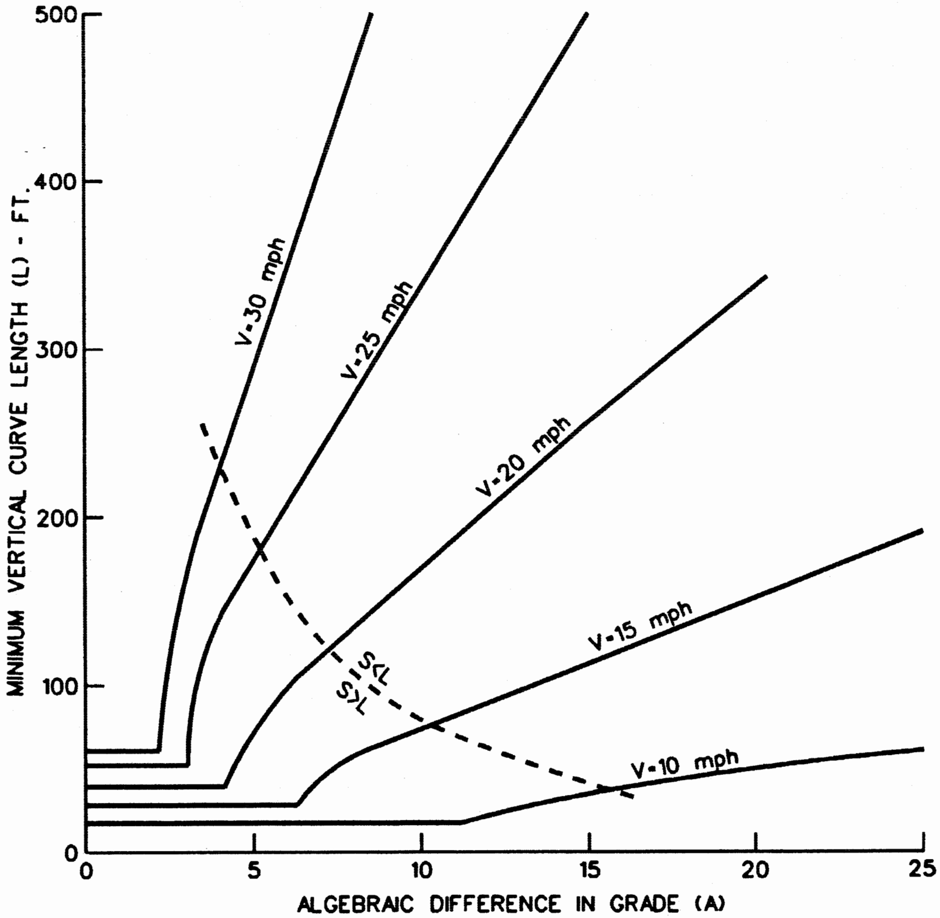
Like sight distance, vertical curve lengths on bikeways are computed much the same as for motor vehicles. It is determined largely by bicycle speed and grade difference. On a two-way facility, the design speed for the descending grade is used.

The appropriate vertical curve length may be selected by using either of the formulas or the graph in Figure 17.3B. Design speed for motor vehicles is much higher than for bicycles. Therefore, the figure is not applicable to bicycle lanes and shared roadways.

17.3.3.7 Surface and Structural Section

The surface of a bikeway of any type must be smooth, hard and durable. A smooth surface is required for the safety and comfort of the cyclist. Rough surfaces can result in a lack of control and, due to the poor ride quality of modern bicycles, result in a very bumpy, uncomfortable ride. Durability of a bikeway surface is important because it will prolong the life of the facility and reduce maintenance costs and effort.

The primary criteria governing a bikeway structural section are its own stability and the ability to support anticipated wheel loads. This is determined primarily by the



$$L = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A} \quad \text{When } S > L$$

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \quad \text{When } S < L$$

$L_{MIN} = 2V$

S	=	Stopping Site Distance (Ft.)
A	=	Algebraic Difference in Grade
h_1	=	Eye Height of Bicyclist (4.5 Feet)
h_2	=	Height of Object (0 Feet)
L	=	Minimum Vertical Curve Length (Ft.)

Source (5)

SIGHT DISTANCES FOR CREST VERTICAL CURVES

Figure 17.3B

maintenance equipment and other motorized vehicles that must use or cross the facility, rather than by the bicycles themselves.

Bicycle lanes and shared roadways use existing pavements intended for motor vehicle use, which normally will satisfy the surface and structural criteria. Should widening and/or resurfacing be required, acceptable material would automatically be used. It is highly recommended that the surface of bicycle lanes be of equal or better quality than the adjacent travel lanes to encourage the cyclist to use the bicycle lanes. Wherever a shoulder is widened, the entire shoulder should be resurfaced to avoid seams or irregular surfaces.

17.3.3.8 Drainage

Adequate drainage should be provided for all types of bicycle facilities.

To ensure proper runoff, all bicycle paths must be cross-sloped. The slope may be to one side or crowned, as conditions dictate, and should not exceed 0.02 ft per ft on straight sections. To avoid problems with icing and flow of excess water, bicycle paths should be crowned wherever practical. Paths placed on a hillside, where a significant amount of runoff is expected, must be provided with a drainage ditch on the up-slope side. Other related drainage structures, such as catch basins and underdrains, should be installed where appropriate. Ditches should also be provided in other areas where the soil has poor drainage qualities. In extreme cases, subdrainage may be warranted.

Bicycle paths crossing waterways should be designed with culverts or combinations of culverts and sags in the profile which will provide for the passage of storm flow without hazard to private property or highways. The design frequency should be based on the particular conditions at the site.

Existing roadway drainage systems will normally be adequate to satisfy the drainage requirements of bicycle lanes and shared roadways. Any questionable situations, however, must be investigated and, if necessary, corrected.

The primary problem with roadway drainage systems is the hazard presented by metal catch basin grates. Any of these within the bikeway should be equipped with grates or with some other configuration which will not entrap a narrow bicycle tire. All grates must be placed and maintained at grade in order to ensure a smooth ride. See ODOT *Drainage Manual* for a discussion on bicycle-safe grates.

17.3.3.9 Signing and Marking

Adequate signing and marking are essential on bicycle paths, especially to alert bicyclists to potential hazards and to convey regulatory messages to both bicyclists and motorists at highway intersections. In addition, guide signing, such as to indicate directions, destinations, distances, route numbers and names of crossing streets, should be used in the same manner as they are used on highways. In general, uniform application of traffic control devices, as described in the MUTCD, will tend to encourage proper bicyclist behavior.

17.3.3.10 Intersection and Crossing Treatment

A well-designed intersection is essential to safety at all points where a bikeway crosses a roadway or other transportation facility. This is especially true if one or both facilities carry a large volume of traffic.

A grade-separated crossing is far safer than an at-grade crossing. These may take one of two forms -- the overpass or underpass. The latter has the advantage of the downgrade being first, allowing the cyclist to gain momentum which facilitates upgrade pedaling. There is also less vertical distance to be traveled. The overpass has the advantages of being less expensive and less of a security problem in high crime areas. Either type of grade-separated crossing is prohibitively expensive and is justified only in the most severe conditions.

At-grade crossings and intersections should be provided with some sort of channelization, especially if there is a large amount of turning traffic. This will tend to restrict the movement of both cyclists and motorists to areas which are designated for them. Each crossing and intersection is a unique situation and should be treated as such.

Figure 17.3C provides possible treatments for a bicycle lane approaching an intersection.

17.3.3.11 Capacity

Bicycle capacity has several aspects, including:

1. the impacts of bicycles on the capacity of a highway which also serves as a shared roadway or bicycle lane;

2. the impacts of bicycles on intersection capacity; and

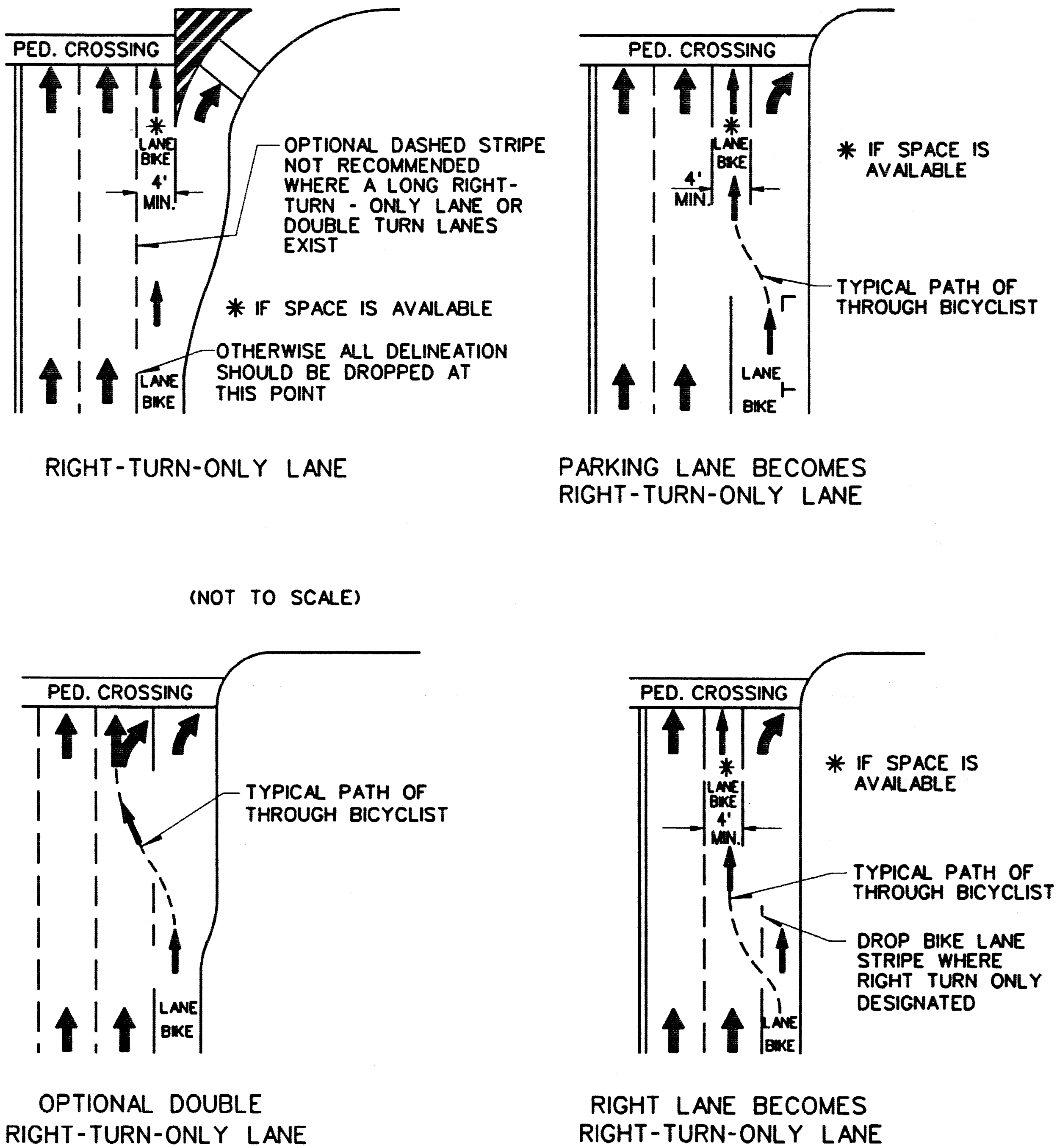
3. the capacity of bike paths.

The *Highway Capacity Manual* provides criteria for each of the above.

17.3.3.12 Railroad Crossings

Railroad-bikeway grade crossings should ideally be at right angles to the rails. The greater the crossing deviates from this ideal crossing angle, the greater the potential for a bicyclist's front wheel to be trapped in the flangeway causing loss of steering control. It is also important that the roadway approach be at the same elevation as the rails.

Consideration should be given to the materials of the crossing surface and to the flangeway depth and width. If the crossing angle is less than approximately 45 degrees, consideration should be given to widening the outside lane, shoulder or bicycle lane to allow bicyclists adequate room to cross the tracks at a right angle. Where this is not practical, commercially available compressible flangeway fillers can enhance bicyclist safety. In some cases, abandoned tracks can be removed. Warning signs and pavement markings should be installed in accordance with the MUTCD.



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BICYCLE LANES APPROACHING MOTOR VEHICLE RIGHT-TURN-ONLY LANES

Figure 17.3C

17.4 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 is based on information presented in the *Uniform Federal Accessibility Standards* (UFAS). Designers are required to meet the criteria presented in the following sections. When other agencies or local codes require standards which exceed the UFAS, then the stricter criteria may be required. This will be determined on a case-by-case basis.

17.4.1 Parking

The following criteria apply for handicapped parking spaces:

1. Minimum Number. Table 17.4A provides the criteria for the minimum number of accessible spaces. A typical handicapped stall layout is shown in Figure 17.4A.

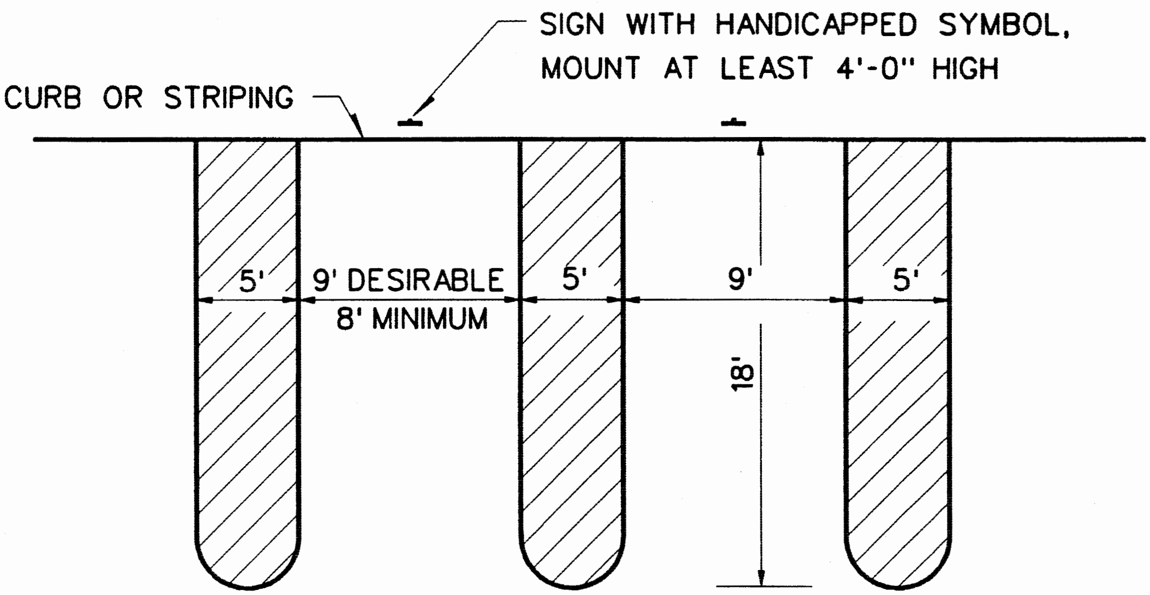
2. Location. Parking spaces for disabled individuals and accessible passenger loading zones that serve a particular building shall be the spaces or zones closest to the nearest accessible entrance on an accessible route. In separate parking structures or lots that do not serve a particular building, parking spaces for disabled individuals shall be located on the shortest possible circulation route to an accessible pedestrian entrance of the parking facility.

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 (see Section 2D-46 of the MUTCD). The sign shall not be obscured by a vehicle parked in the space.

4. Striping. ODOT has not adopted specific criteria relative to the placement of handicapped markings. The designer should use the applicable pavement marking criteria presented in Chapter Fourteen.

5. Dimensions. The parking spaces designated for the handicapped shall be at a minimum 8-ft wide and desirably 9-ft wide with an additional 5-ft minimum access aisle, or the space should be parallel to a sidewalk on a public highway (See Figure 17.4A). 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. Parking spaces and access aisles shall be level with surface slopes not exceeding 50:1 (2%) in all directions. Any parking garage or terminal should have a 9′-6″ vertical clearance at its entrance and along the route to at least two parking spaces which have a 9′-6″ vertical clearance.

6. Passenger Loading Zones. Passenger loading zones shall provide an access aisle at least 60-inches wide and 20-ft long 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 ramp complying with Section 17.4.6 shall be provided. Vehicular standing spaces and access aisles shall be essentially level. Surface slopes shall not exceed 50:1 (2%) in all directions.



Source (2)

HANDICAPPED PARKING STALL DIMENSIONS

Figure 17.4A

Table 17.4A
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

Source (8)

Notes:

a. *If one or more passenger loading zones are provided, then at least one passenger loading zone shall comply with Section 17.4.1, Comment No. 6.*

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.*

17.4.2 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 a building, structure or area, including the site on which such building, structure or area is located, wherein specific services are provided or activities performed. Interior accessible routes include corridors, floors, ramps, elevators, lifts and clear floor space at fixtures. Exterior accessible routes include parking access aisles, curb ramps, walks, 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, and accessible passenger loading zones, and public

streets or sidewalks to the accessible building entrance they serve.

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, weigh stations, park-and-ride lots, etc. The accessible route criteria do not apply to, for example, sidewalks adjacent to an urban street.

17.4.3 Sidewalks

Section 8.1 presents the Department's warrants and design criteria for sidewalks. In addition, all sidewalks on an accessible route must comply with the UFAS criteria. These include:

1. Width. The minimum clear width shall be 36 inches, except at doors (minimum 32 inches).

2. Passing Space. If the sidewalk has less than 60 inches clear width, then passing spaces at least 60 inches by 60 inches shall be located at reasonable intervals not to exceed 200 ft. A T-intersection of two walks is an acceptable passing place.

3. Surface. All sidewalk surfaces shall be slip resistant. The longitudinal gradient shall be flush and free of abrupt changes.

   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 ½" 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 50:1 (2%). If the longitudinal gradient exceeds 20:1 (5%), the sidewalk must meet the accessibility criteria for ramps. (See Section 17.4.5).

17.4.4 Stairs

Stairs shall not be part of an accessible route because they cannot be safely negotiated by individuals in wheelchairs. Where stairs are used, however, they should be designed to be accessible by other handicapped individuals. Therefore, the design of stairs must comply with the UFAS criteria (Reference 8, Section 4.9). This includes, for example, the provision of handrails. The designer may also reference FHWA-IP-84-6 *Guidelines for Making Pedestrian Crossing Structures Accessible* for additional design information on stairs.

17.4.5 Ramps

Any part of an accessible route with a slope greater than 20:1 (5%) shall be considered a ramp and shall conform to the UFAS criteria. This includes the provision of handrails. The designer may also reference FHWA-IP-84-6 *Guidelines for Making Pedestrian Crossing Structures Accessible* for additional design information on ramps.

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. The maximum slope of a ramp in new

construction shall be 12:1 (8%). The maximum rise for any run shall be 30 inches. Curb ramps and ramps to be constructed on existing sites or in existing buildings or facilities may have slopes and rises as shown in Table 17.4B if space limitations prohibit the use of a 12:1 slope or less.

Table 17.4B

ALLOWABLE RAMP DIMENSIONS FOR CONSTRUCTION (Existing Sites, Buildings and Facilities)

Slope	Maximum Rise	Maximum Run
Steeper than 10:1 but no steeper than 8:1	3"	2'
Steeper than 12:1 but no steeper than 10:1	6"	5'

Note: A slope steeper than 8:1 (12.5%) is not allowed.

Source (8)

2. Width. The minimum clear width of a ramp shall be 36 inches.

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 inches clear.

   c. If ramps change direction at landings, the minimum landing size shall be 60 inches by 60 inches.

4. Handrails. If a ramp run has a rise greater than 6 inches or a horizontal projection greater than 72 inches, then it shall have handrails on both sides. Handrails are not required on wheelchair ramps. Handrails 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.

   b. If handrails are not continuous, they shall extend at least 12 inches 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 1½".

   d. Gripping surfaces shall be continuous.

   e. Top of handrail gripping surfaces shall be mounted between 30 inches and 34 inches 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.

   h. See Section 4.26 of the UFAS for additional criteria on handrails.

5. Cross Slope and Surfaces. The cross slope of ramp surfaces shall be no greater than 50:1 (2%). Ramp surfaces shall comply with the criteria for "Surface" for sidewalks (Section 17.4.3).

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 inches high.

7. Outdoor Conditions. Outdoor ramps and their approaches shall be designed so that water will not accumulate on walking surfaces.

17.4.6 Curb Ramps (Wheelchair Ramps)

Curb ramps and other provisions for the handicapped are required on all Federal-aid projects involving the provision of curbs or sidewalks at all pedestrian crosswalks. For the purpose of this section, a pedestrian crosswalk is defined as the portion of a street distinctly indicated as a crossing for pedestrians by lines or other markings on the surface. All curb ramps on an accessible route, see Section 17.4.2, shall meet the UFAS criteria. For all other locations, curb ramps may be constructed in accordance with the applicable criteria.

17.4.6.1 Location

In determining the need for a curb ramp, the designer should consider the following:

1. If at least one curb will be disturbed by construction at an existing intersection, then curb ramps will be constructed at all crosswalks which extend from a paved sidewalk in that intersection.

2. Throughout a new or reconstruction project at each intersection with pedestrian crosswalks, curb ramps will be provided on all corners. At T-intersections, the designer must ensure that curb ramps are located on the side opposite the minor intersecting road.

3. There should be full continuity of use throughout; i.e., opposing ramps must always be provided even if outside project limits.

4. Curb ramps shall be positioned so as not to cause a safety hazard for blind pedestrians.

5. Curb ramps shall be located or protected to prevent their obstruction by parked vehicles.

6. A curb ramp on an accessible route shall be wholly contained within the painted markings, excluding any flared sides. For diagonal ramps only, there shall be at least 48 inches between the gutter line and the corner of the two intersecting crosswalks and at least 2 ft of full-height curb within the crosswalk. See Figure 17.4B for an illustration of these criteria.

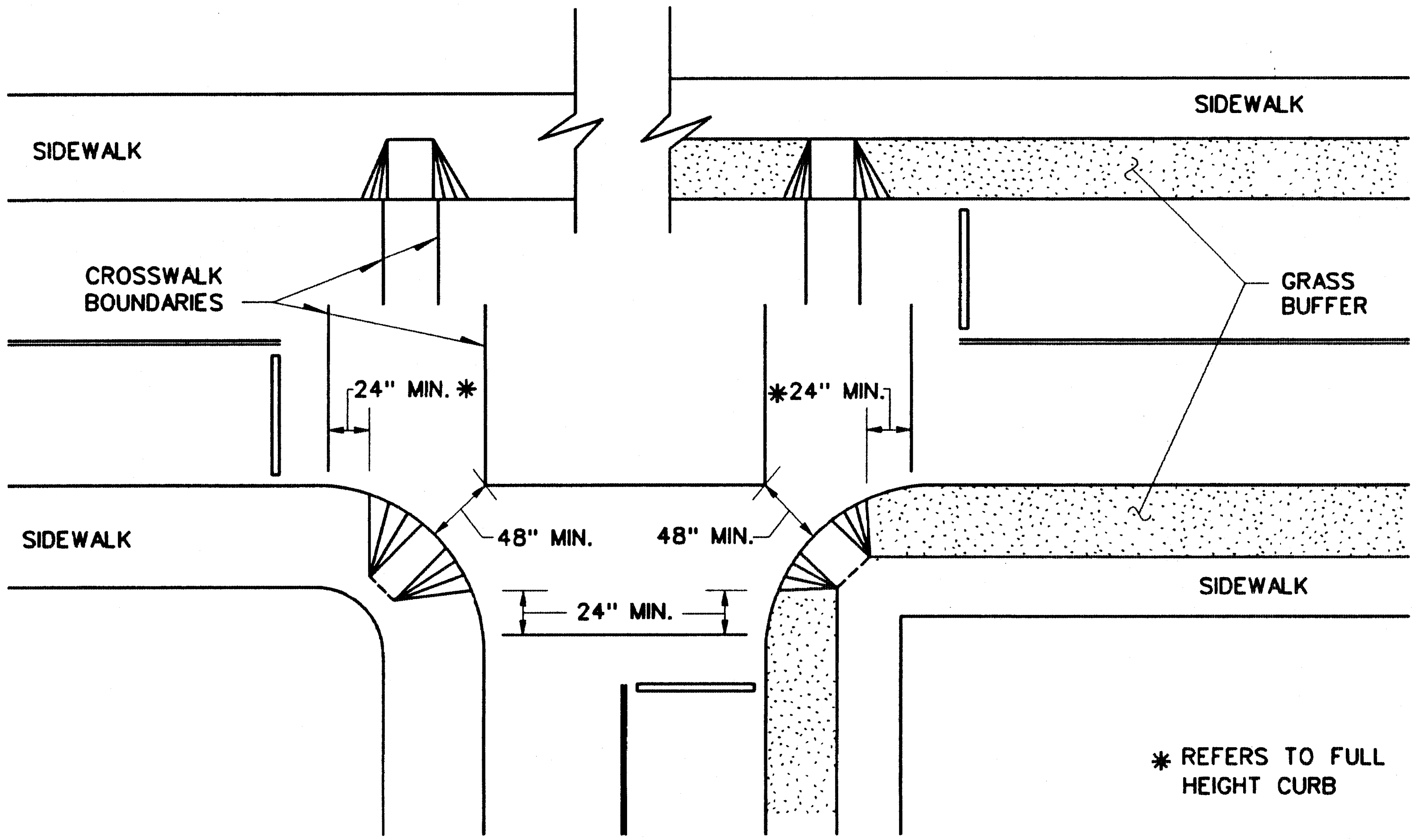
7. Curb ramps shall be provided along all accessible routes.

8. The function of the curb ramp must not be compromised by other highway features (e.g., guardrail, catch basins, utility poles, signs).

9. If a pedestrian crosswalk and curb ramp are present, a pedestrian-actuated traffic signal will be provided. In addition, the pedestrian pushbutton should be located so it can be reached by wheelchair-bound individuals. If necessary, a sidewalk should be provided between the curb ramp and the pedestrian pushbutton.

10. In general, curb ramps are required at all curbed intersections. However, a Level Two exception may be granted for the following locations:

    a. Locations outside the urban boundary of a city with a population of greater



NOTE: SEE ODOT STANDARD DRAWINGS FOR DESIGN DETAILS ON CURB RAMPS

Source (8)

CURB RAMPS AT MARKED CROSSINGS

Figure 17.4B

than 5000 and in rural areas unless there is clearly a pedestrian demand.

b. Locations where there are valid reasons to restrict or prohibit pedestrian access.

c. Locations where there are no pedestrian and there is little likelihood of pedestrians in the future.

The decision to eliminate curb ramps at an intersection will be determined by a mutual agreement between ODOT and the FHWA at the plan-in-hand review meeting.

17.4.6.2 Types

Details for the construction of curb ramps are illustrated in Figure 17.4C and in the ODOT Standard Drawings. The designer needs to ensure that the curb ramp type is appropriate for the site and location. The following provides information for each type of curb ramp used by ODOT:

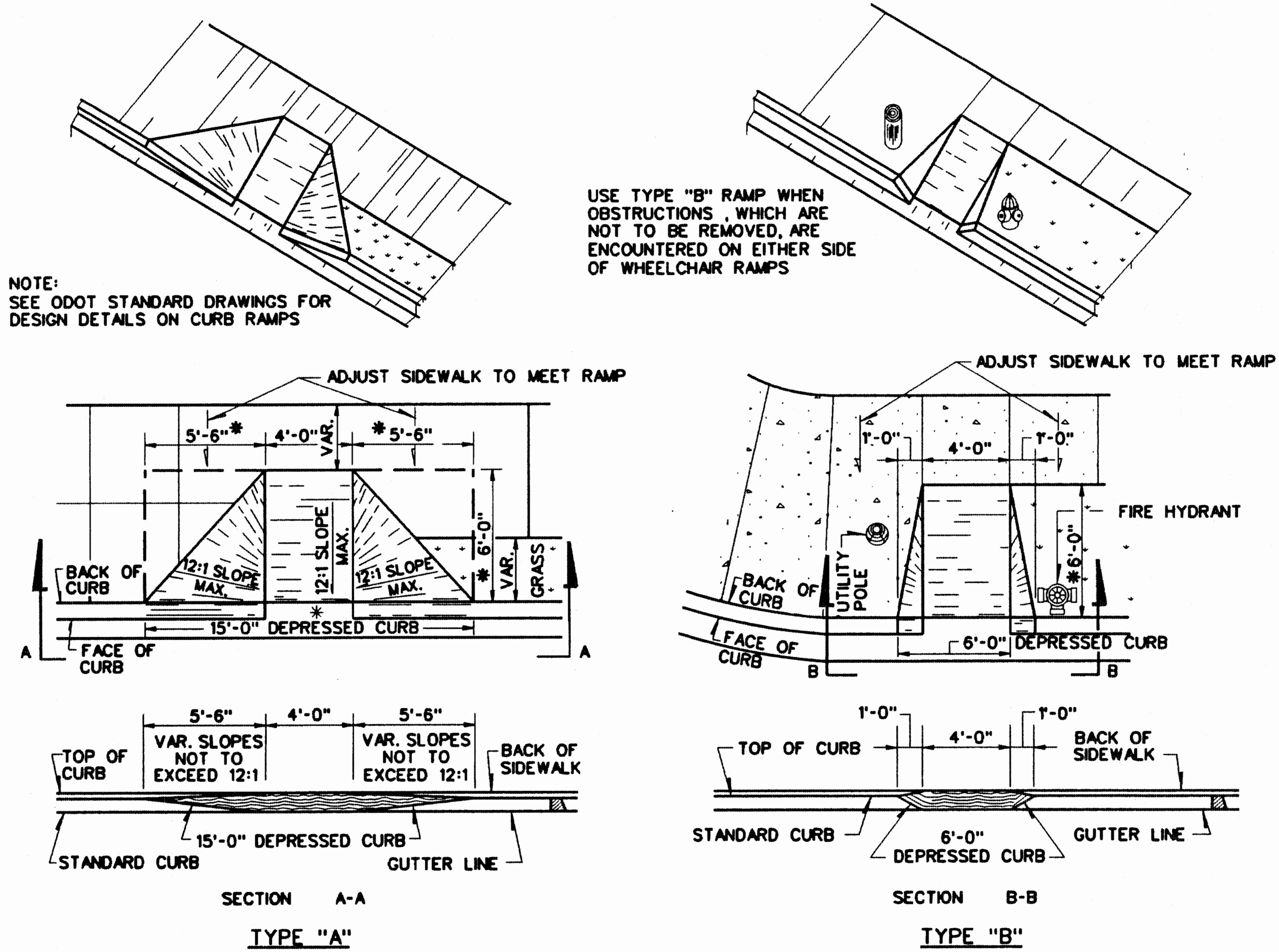
1. Type A. The flare type curb ramp is commonly used at intersections with large turning radii or at mid-block locations. This ramp type can be used with or without a grass border strip. When used at corners, the bottom of the ramp shall have a 4-ft minimum clear space within the marked crossing as shown in Figure 17.4B. In addition, the ramp should also have a 2-ft segment of straight curb located on each side within the markings.

2. Type B. This curb ramp can only be used when there are obstructions that would restrict the use of Type A or D with the full slopes (e.g., utility poles, fire hydrants, signal poles) or at the end of a sidewalk. If the curb ramp is not at the end of a sidewalk, there must be a 4-ft landing space at the top of the ramp to allow handicapped individuals a maneuvering area. If used in a sidewalk which is next to the curb, special care must be given to alert the crossing pedestrian, especially the blind, of the dropoff.

3. Type C. This curb ramp is used in narrow raised medians that are 12 ft or less. This ramp is a 4-ft wide level strip through the raised median which is at the same grade as the roadway. Special consideration must be given to drainage to ensure that water is not allowed to stand or collect in this area. For medians greater than 12 ft, a Type B curb ramp is normally used at each curb.

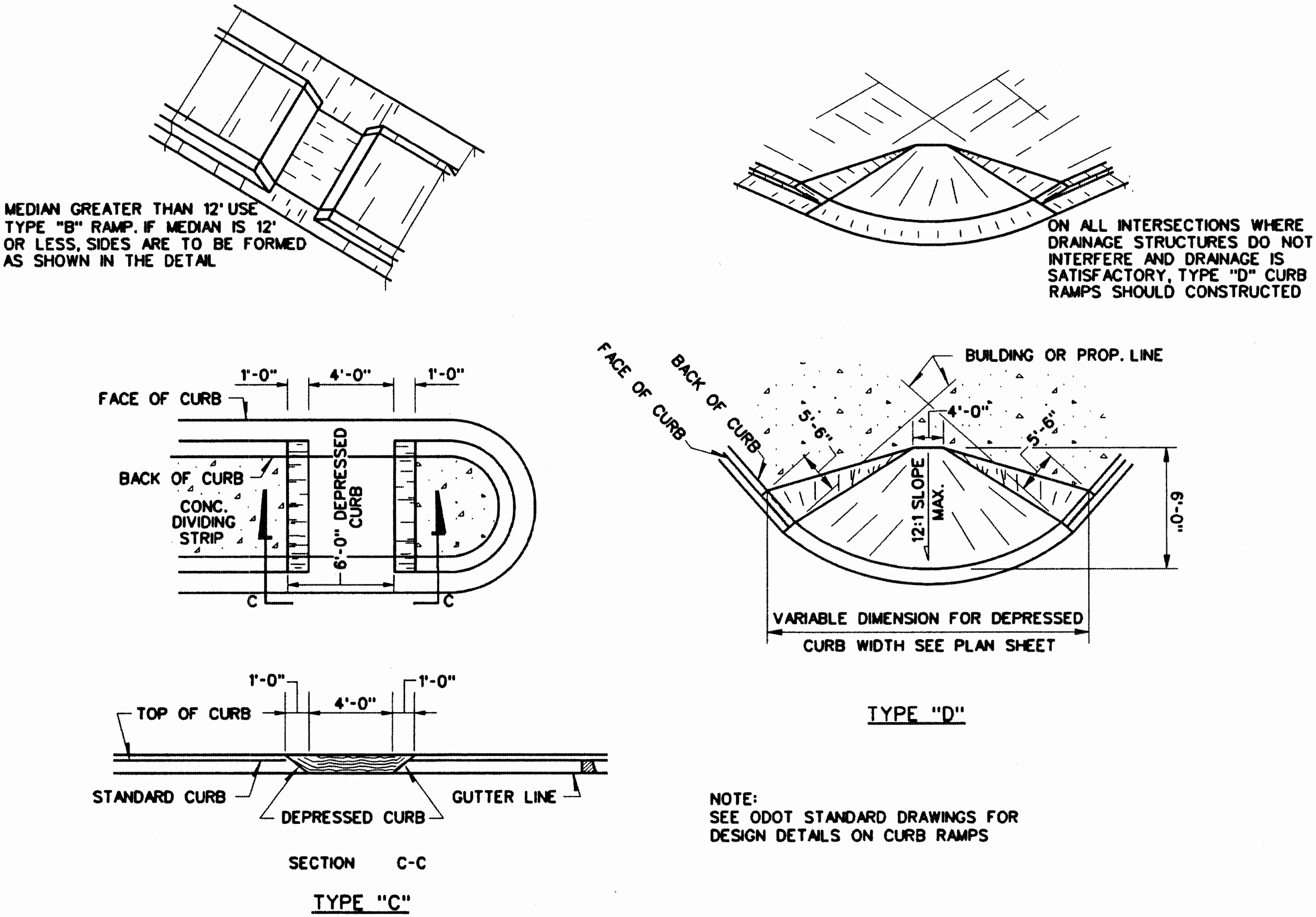
4. Type D. This curb ramp is normally used at intersections with small turning radii and where drainage structures do not interfere with the use of the ramp. The bottom of diagonal curb ramps shall have a 4-ft minimum clear space within the marked crossing as shown in Figure 17.4B. The curb ramp shall also have a 2-ft segment of straight curb located on each side within the markings.

All curb ramps shall be constructed in accordance with the ODOT *Standard Specifications* and the ODOT Standard Drawings.



TYPICAL CURB RAMPS

Figure 17.4C



TYPICAL CURB RAMPS
(Continued)

Figure 17.4C

17.5 REST AREAS

Rest areas, information centers and scenic overlooks are functional and desirable elements of the complete highway development and are provided for the safety and convenience of the highway user. Many have been constructed along freeways and other major arterials in Oklahoma. The location and design of rest areas are considered on an individual highway facility and site basis.

17.5.1 Location

Rest areas may be located on freeways or other major arterials. The following elements should be considered in determining the need and location of rest areas.

17.5.1.1 Spacing

The recommended average spacing of rest areas on rural Interstates is approximately one hour's driving time or 50 to 60 miles. In some situations it may desirable to provide closer spacings for special conditions (e.g., scenic views, information centers). Local conditions may warrant spacings which are greater than 50 to 60 miles (e.g., through major metropolitan areas). In addition, they should not be located within 50 to 60 miles of major metropolitan areas. In national studies it has been found that rest areas close to these areas tend to have higher rates of vandalism and undesirable activities.

17.5.1.2 Site Selections

Once it has been determined that a rest area is required and the general area selected, the actual location of the rest area needs to be finalized based upon the following considerations:

1. Appeal. Rest areas are show places for out-of-state visitors to Oklahoma. If practical, they should be placed to take advantage of natural features (e.g., lakes, scenic views, points of special or historic interest).

2. Water/Sewer. The area should have an adequate water supply. If commercial sanitary treatment plants are unavailable, the site needs to be large enough to provide for adequate sewage treatment facilities.

3. Other Utilities. Utilities, such as telephone and electricity desirably should be provided.

4. Geometrics. The site should be located away from any other interference, such as interchanges. Desirably, the rest area entrance should be at least 2 miles from the nearest interchange. In addition, there should be adequate sight distance available to the exits and entrances.

5. Topography. Rest areas should be located where the natural topography is favorable to their development.

6. Size. The rest area needs to be large enough to provide for sufficient parking capacity, facilities needs, picnic and stretch areas and to retain existing landscaping features.

7. Right-of-way. Right-of-way costs should be factored into the final location decision.

8. Development. Rest areas should not be placed adjacent or near areas zoned residential.

9. Information Centers. It is desirable to locate rest areas close to the State line. These locations provide the opportunity

for providing information on the State of Oklahoma and local attractions.

10. Emergency. The location choice should consider the proximity to emergency services.

17.5.2 Design

Figure 17.5A illustrates a typical layout for a rest area. The following sections present criteria which should be considered in the design of the rest area.

17.5.2.1 Exits and Entrances

The access to and from the rest area should be designed according to the criteria in Section 10.3 "Freeway/Ramp Junctions." Research has shown that a large portion of the vehicles at rest areas are trucks. Consequently, if practical, the acceleration distances for large trucks should be used in designing freeway/entrance junctions.

Adequate signing and pavement markings need to be provided prior to and at the rest area. These traffic control devices should be placed in accordance with the MUTCD and the ODOT Standard Drawings.

17.5.2.2 Rest Area Usage

The following formulas have been developed as a general guide to determine the portion of passing vehicles on the mainline which may use the rest area.

P (interstate) = 0.0024 x DSL
P (rural arterial, recreational) = 0.0016 x DSL
P (rural arterial, other) = 0.0011 x DSL

where:

P = proportion of the mainline traffic entering the rest area

DSL = distance between rest areas (miles)

Example

Given:

Rural Interstate
Spacing between rest areas = 60 miles
One-Way Mainline ADT = 15,000

Problem:

Determine the expected rest area ADT.

Solution:

The following steps apply:

1. Determine the percentage of the mainline traffic that will be expected to use the rest area. Using the first equation and knowing DSL = 60 miles, the equation becomes:

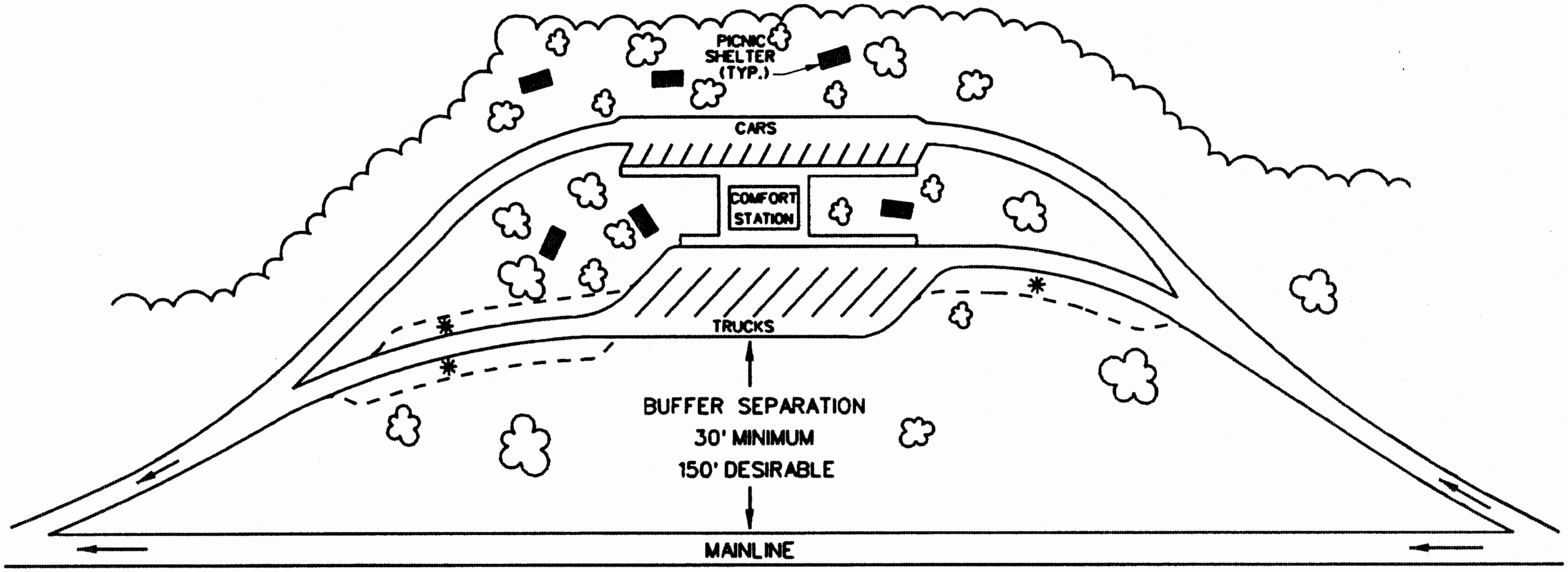
   P(interstate) = 0.0024 x 60 = 0.144

2. Determine the expected rest area ADT.

   ADT = 0.144 x 15,000 = 2,160 vehicles

3. This value than can be used to determine the "design hour" for the rest area.

The above formulas provide only general guidelines. There are several other factors the designer needs to consider in determining the rest area's usage:



* POSSIBLE LOCATIONS FOR WB-114 TRUCK PARKING

Source (11)

TYPICAL REST AREA

Figure 17.5A

1. Traffic Characteristics. Research has found that between 14% to 36% of the vehicles entering rest areas are trucks or recreational vehicles (RV's). As a rule of thumb, the designer can assume that 25% of the entering traffic is trucks or RV's.

2. Highway Characteristics. Rest areas on highways that pass though recreational or historic areas tend to have fewer trucks and a higher percentage of passenger cars and RV's with trailers. Where the general purpose of the highway is to move commercial traffic between cities, rest areas tend to have a higher truck usage.

3. Trip Length. On highways where the trip lengths tend to be less than 100 miles (e.g., between two major cities), there is a significant reduction in the proportion of the passing traffic using the facility.

4. Temporal Factors. In recreational areas, rest area usage commonly is the highest during summer weekends. During the day, passenger cars tend to make up a higher percentage of the rest area usage. At night, trucks and RV's tend to make up the higher percentage of the rest area usage.

5. Length of Stay. During the day, the average length of stay for cars has been found to be 11 minutes, 12.1 minutes for trucks, and 19.5 minutes for RV's. At rest areas with information centers, the time for cars and RV's increased by approximately 2 minutes. At noon, all vehicles tend to stay more than 15 minutes. At night, the mean time for trucks and RV's is between 65 and 75 minutes.

17.5.2.3 Parking

Rest area parking capacity depends upon the type of usage expected for the rest area. Section 17.5.2.2 provides the formula and other factors to consider for determining the appropriate design hour volume for both passenger cars and trucks.

If practical, passenger car parking and truck parking should be separated. This can be accomplished by providing separate parking areas as shown in Figure 17.5A or by pavement markings. Figure 17.1A illustrates typical parking designs for passenger cars. Angular parking is preferred versus parallel parking because it requires less time to enter and exit.

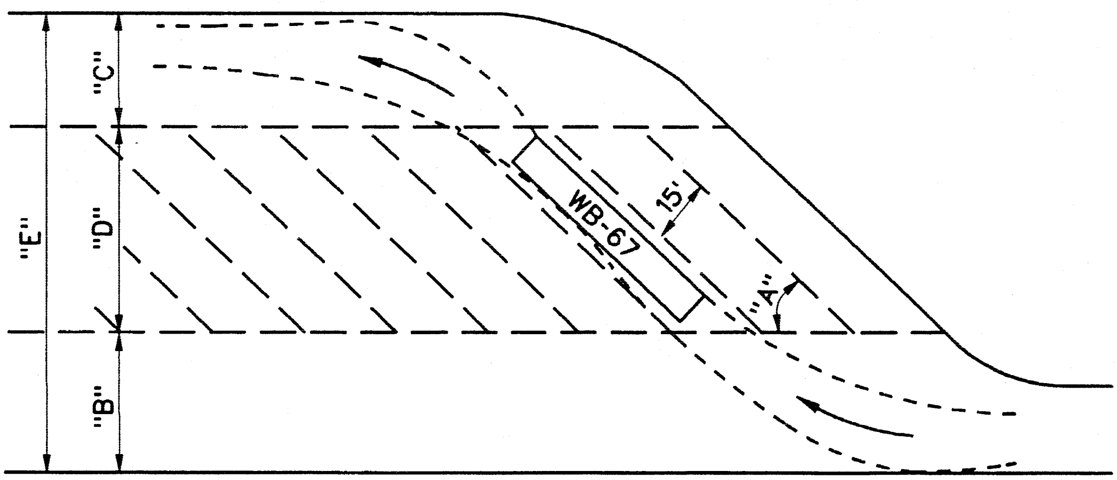
Figure 17.5B illustrates a typical angle parking design for trucks and recreational vehicles with trailers. It should be noted that the design vehicle for angular truck parking is the WB-67 vehicle. In addition to the angular truck parking spaces, one or more parallel slots should be provided for the WB-114 vehicle (see Figure 17.5A).

17.5.2.4 Buffer Separation

The separation between the rest area facilities and the highway mainline should be wide enough to discourage individuals from stopping on the mainline and crossing over to the facilities. At a minimum, a 30-ft buffer area should be provided between the mainline pavement and the parking areas. A buffer separation of 150 ft or more is preferable.

17.5.2.5 Facilities

Rest areas typically provide a building with rest rooms and public information services, picnic tables and shelters, benches, sidewalks, drinking fountains, vending machines and



LEGEND

"A" - ANGLE OF PARKING
"B" - ENTRANCE ROADWAY WIDTH
"C" - EXIT ROADWAY WIDTH
"D" - PARKING WIDTH
"E" - TOTAL WIDTH

DETAILS FOR PARKING SPACE

ANGLE OF PARKING (DEGREES) "A"	ENTRANCE ROADWAY WIDTH (FEET) "B"	EXIT ROADWAY WIDTH (FEET) "C"	PARKING WIDTH (FEET) "D"	TOTAL WIDTH PARKING AREA (FEET) "E"
30	30	30	50	110
45	40	35	70	145
60	50	45	85	180

DESIGNS FOR ANGLE PARKING OF TRUCKS

Figure 17.5B

trash collectors. The designer should ensure that sufficient facilities are available to accommodate the expected usage of the rest area.

17.5.2.6 Utilities

Where permanent sanitary facilities are provided, an adequate water supply, sewage disposal system and power supply will be required. Telephones are commonly also included. Proper lighting provides the patron an added sense of security and safety. Chapter Fourteen provides additional information on the design of lighting.

17.5.2.7 Landscaping

The rest area should be landscaped to take advantage of existing natural features and vegetation. Paths, sidewalks and architectural style should fit naturally into the existing surroundings. The designer should coordinate the landscaping plan with ODOT's Roadside Development Branch.

17.5.2.8 Accessibility for the Handicapped

All rest areas must be designed to properly accommodate physically handicapped individuals, including grounds and buildings. Section 17.4 provides the accessibility criteria for features within rest areas.

17.6 WEIGH STATIONS

Truck weigh station installations are used to weigh trucks, to provide for vehicular safety inspection, and/or to provide a source of data for planning and research. The determination of the need for truck weigh stations is a combined effort of ODOT, Department of Public Safety, Oklahoma Corporation Commission and Oklahoma Tax Commission.

17.6.1 Location

The primary criteria for location of a truck weigh station is dependent upon the type and amount of truck traffic on a route. The selection of a truck weigh station site is also controlled by right-of-way and by geometric and topographic features. It is desirable to select this site in a location where there is adequate right-of-way and where geometric, topographic and environmental features lend themselves to the most economical development without undue site preparation and expense. The possibility of truck traffic circumventing the facility should also be considered in locating the site of the weigh station.

17.6.2 Design

Figure 17.6A illustrates a typical truck weigh station design for a divided highway. In addition, the following should be considered.

1. Exit/Entrance Junctions. Desirably, the exits and entrances should be designed for the WB-67 truck. Section 10.3 provides design criteria for these elements.

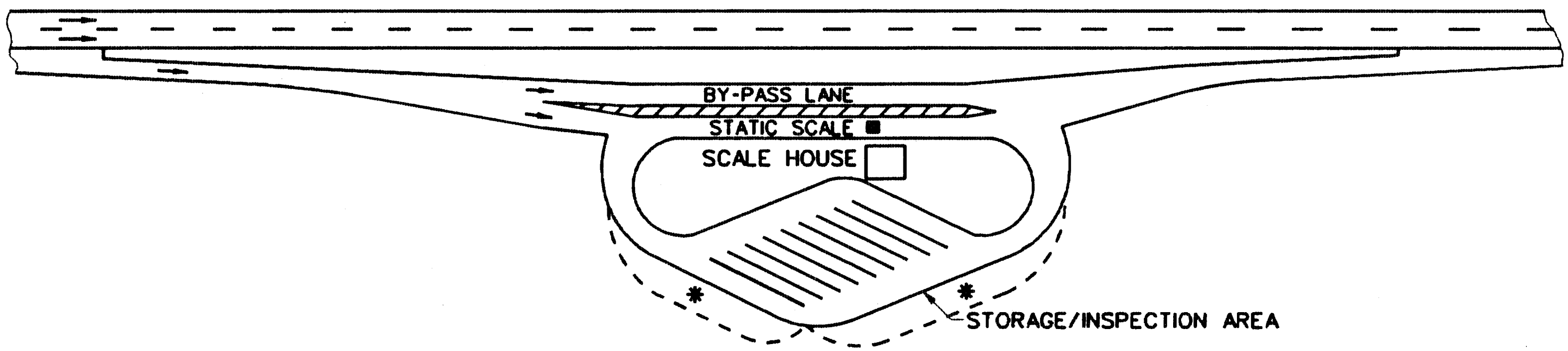
2. Buffer Separation. There should be a minimum of 30 ft between the weigh station facility and the mainline pavement. Wider separations are desirable.

3. Storage for Scales. There should be sufficient space to queue trucks waiting for the scales without backing up onto the mainline. This distance will based on the number of trucks on the mainline, length of trucks, expected hours of operation and time required for actual weighing. For design considerations, the design vehicle can be assumed to be the WB-67 truck. With the rapid advance in research on scales (e.g., weighing-in-motion), the designer should check with the various ODOT Divisions and/or other agencies to determine the most appropriate time factor.

4. Safety Inspection. Weigh stations are often used by the Department of Public Safety (i.e., Oklahoma Highway Patrol) as safety inspection stations. Consequently, there should be sufficient space available to allow trucks to be pulled aside or turned about and inspected. These areas should be designed to accommodate the WB-67 design vehicle. In addition, they should also be able to accommodate the occasional WB-114 vehicle.

5. Violation Storage. A space needs to be provided to store trucks that are either overweight or which have failed the safety inspection. These areas should be designed to accommodate the WB-67 design vehicle. They should also be able to accommodate the occasional WB-114 vehicle. Figure 17.5B provides the design criteria for a WB-67 angular truck storage area. In addition, one or more parallel stalls should be provided for storage of a WB-114 vehicle.

6. Signing and Striping. Adequate signing and pavement markings need to be provided prior to and at the truck weigh station. These traffic control devices should be placed in accordance with the MUTCD and the ODOT Standard Drawings.



* POSSIBLE LOCATION FOR WB-114 TRUCK PARKING

TYPICAL TRUCK WEIGH STATION

Figure 17.6A

7. Lighting. Chapter Fourteen provides information on the design of lighting at truck weigh stations.

8. At-grade Intersection. Weigh stations on undivided facilities should be designed to allow trucks to enter and leave from either direction. The turning design vehicle for these facilities will normally be the WB-67. Chapter Nine presents additional information on the design of at-grade intersections.

9. Utilities. A source of electricity, either from a local power source or remote generator, needs to be provided to the weight station. Desirably, portable water, sanitary facilities and telephones or radios should also be provided.

10. Landscaping. In aesthetically sensitive areas, the weigh station should be landscaped to blend naturally into the surrounding environment. However, care must be given so that the landscape design does not impede the flow of trucks through the facility (e.g., steep grades). The designer should also coordinate the landscaping design with ODOT's Roadside Development Branch.

17.7 MAILBOXES

Mailboxes and newspaper tubes served by carriers in vehicles may constitute a safety hazard either directly or indirectly, depending upon the placement of the mailbox. Therefore, on Federal-aid projects, the designer should make every reasonable effort to replace all non-conforming mailboxes with designs that meet the criteria in the *ODOT Standard Drawings* and with the AASHTO *A Guide for Erecting Mailboxes on Highways*. Removal and replacement of mailboxes can be a sensitive issue and should be evaluated prior to their removal or replacement.

17.7.1 Location

Mailboxes should be placed for maximum convenience to the patron, consistent with safety considerations for highway traffic, the carrier and the patron. Consideration should be given to the minimum walking distance in advance of the mailbox site and possible restrictions to corner sight distance at intersections and driveway entrances. New installations should, where feasible, be located on the far right side of an intersection with a public road or private driveway entrance.

Boxes should be placed only on the right-hand side of the highway in the direction of travel of the carrier, except on one-way streets where they may be placed on the left-hand side. It is undesirable to require pedestrian travel along the shoulder. However, this may be the preferred solution for distances up to 200 feet when compared to the alternatives, such as constructing a turnout in a deep cut, placing a mailbox just beyond a sharp crest vertical curve (poor sight distance), or constructing two or more closely spaced turnouts.

The placing of mailboxes along high-speed, high-volume highways should be avoided if other practical locations are available. Mailboxes should not be located where access is from the lanes of an expressway or where access, stopping or parking is otherwise prohibited by law or regulation. No mailbox should be at a location that would require a patron to cross the lanes of an expressway to deposit or retrieve mail.

Placing a mail stop near an intersection will have an effect on the operation of the intersection. The nature and magnitude of this effect depend on traffic speeds and volumes on each of the intersecting roadways, the number of mailboxes at the stop, extent of traffic control, how the stop is located relative to the traffic control, and the distance the stop is from the intersection. *ODOT Standard Drawings* show possible locations of mail stops at a typical rural intersection.

Mailboxes should be located so that a vehicle stopped at a mailbox is clear of the adjacent traveled way. An exception to this objective may be reasonable on low-volume, low-speed streets and roads. In general, however, a vehicle stopped at a mailbox should be clear of the travelway and, the higher the traffic volume or speed, the greater the clearance should be.

It is also highly desirable to provide a turnout if an 8-ft or wider useable shoulder is unavailable. The *ODOT Standard Drawings* provide additional details in the design of turnouts for mail stops.

17.7.2 Design

The *ODOT Standard Drawings* provide the design criteria for the proper placement and attachment of mailboxes. The designer should also consider the following:

1. Heights. Mailbox heights are usually located so that the bottom of the box is 42 to 48 inches above the mail stop surface.

2. Post. Nominal 4-inch x 4-inch or 4½-inch diameter wood posts or 1½-inch to 2-inch diameter steel or aluminum pipe post, embedded 24 inches into the ground, are the maximum strength supports that should be considered. The use of concrete anchors is not recommended.

3. Multiple Mailboxes. To reduce the possibility of ramping, multiple mailboxes should be separated by a distance at least equal to three-fourths of their heights above ground.

4. Neighborhood Delivery and Collection Box Units (NDCBU). NDCBU is a cluster of 8 to 16 locked boxes mounted on a pedestal or within a framework. These clusters can weigh between 100 to 200 pounds and are considered to be a roadside hazard. Consequently, they should be located outside the clear zone or only on low-speed curbed facilities. Normally, NDCBU are located in trailer parks, apartment complexes and new residential subdivisions.

17.8 RECREATIONAL ROADS

Recreational road criteria are applicable to roads in state parks, recreational areas, national forests and on scenic drives. The objective for this type of facility is to provide a safe highway and still retain the aesthetic, ecological, environmental and cultural amenities of the area.

In the design of recreational roads, the designer should consider the following:

1. Functional Classification. The functional classification determination is based on the characteristics of the highway. For design purposes, the designer often needs to determine this classification (see Chapter Five). If the road falls under the local road classification, then the local road criteria in Chapters Twelve and Thirteen should be used to determine the design criteria. If the road falls under the collector classification, then the collector criteria in Chapters Twelve and Thirteen should be used.

2. Design Criteria. Strict adherence to highway criteria for these types of roads is usually inappropriate and unwarranted. Design speeds are usually low and driver expectancy is such that the reduction of criteria does not produce serious safety concerns. Therefore, the designer needs to use engineering judgment to ensure that the criteria fits the terrain and expected usage of the highway.

3. Design Vehicle. Depending on the nature of the recreational areas, the most common design vehicle will be a motor home with a boat trailer. Where garbage pickup or other maintenance vehicles are required, an SU may be the appropriate design vehicle. In some situations, only a passenger car may be appropriate. The designer needs to use engineering judgment to determine the appropriate design vehicle.

The selected design vehicle should be used to determine lane widths, vertical clearances, intersection designs, etc.

17.9 HAZARDOUS MATERIALS

The treatment of hazardous materials within the highway right-of-way is fast becoming a major design concern in the design and construction of highways. Because cleanup of these sites can be very expensive, early acknowledgement of the hazardous site may significantly reduce problems later during construction.

17.9.1 Coordination

Because cleanup of hazardous materials requires extensive use of manpower, time and funds, early coordination between all units involved is critical. Therefore, it is ODOT policy that, once hazardous materials are discovered on a project, all pertinent individuals on the project will be notified. For additional information, see ODOT's *Policy on Environmental Mitigation*.

17.9.2 Location

Hazardous materials can show up almost anywhere. However, common possible locations include near abandoned storage tanks, abandoned oil lines, illegal dumping sites, abandoned chemical plants, old railroad yards, auto junkyards, landfills, or near bridges with lead base paints. One early indicator of contamination is when hazardous elements are found in the ground water from nearby wells. If there is a reasonable chance a site may contain hazardous materials, the Planning Division should be contacted to determine if detailed testing on the type, size and contaminant level is required.

17.9.3 Cleanup

Once the hazardous material location is known, its location must be shown on the plans. In addition, if known, the type of contamination must also be provided. The specifications or special provisions should include detailed instructions on the procedures for removing the material and properly disposing of the wastes. For example, on bridges with lead-based paints, all waste materials from sand blasting will not be allowed into the air or onto the ground, but instead must be collected and be properly disposed.

Certain special cleanup sites and materials may require a specialist contractor to determine the location and size of the contaminated site, and to provide for the proper removal and disposal of the contaminated materials. For additional information, see ODOT's *Policy on Environmental Mitigation*.

17.10 NOISE BARRIERS/WALLS

Noise barriers are designed and erected to reduce the noise level of traffic adjacent to existing buildings to an acceptable level as determined by Federal guidelines. The Planning Division is responsible for determining the location of the wall. The designer is responsible for the selection and design of the noise barrier, and the designer evaluates the impacts of the noise barrier on the highway design.

17.10.1 Types

Several types of noise barriers are effective in reducing the environmental impact of noise from the highway. The following lists several types that may be used:

1. Earth Berms. Earth berms are graded mounds of soil to redirect the highway sound from nearby areas.

2. Masonry Walls. These walls are constructed from concrete blocks or bricks. Very pleasing architectural designs can be accomplished with this type of wall.

3. Concrete Walls. These walls can be poured in place or precast. The advantage of concrete walls is that decorative designs can be added to the face of the wall.

4. Wood Walls. Wood walls may be significantly cheaper than masonry and concrete walls. However, their life expectancy is considered to be less than masonry or concrete walls. In addition, maintenance costs may be more expensive than concrete or masonry walls. ODOT generally does not use this wall type.

5. Combination Walls. These walls use a combination of an earth berm and a masonry or concrete wall. These type walls are often used to reduce the height of the masonry or concrete wall and for aesthetic purposes.

17.10.2 Design

The following sections list several factors that need to be considered in the design of a noise wall.

17.10.2.1 Roadside Safety

Section 11.2 provides ODOT's design criteria for clear zones. If practical, noise barrier walls should be placed outside of the clear zone value. If the wall is within the clear zone, an integral CMB shape or separate CMB should be considered to protect the wall from run-off-road vehicles. Generally, noise walls are terminated outside the clear zone. However, if the end of the wall is within the clear zone, the designer should consider protecting the end with an appropriate impact attenuator. Chapter Eleven discusses the design of impact attenuators.

If the noise barrier is an earth berm, the toe of the barrier should be traversable by a run-off-the-road vehicle (see Section 11.2).

17.10.2.2 Sight Distances

At at-grade intersections, noise barriers should not be located in the triangle required for corner sight distance. Chapter Nine provides the criteria to determine the required sight distance triangle.

Noise barriers can also restrict sight distances along horizontal curves. Chapter Six provides the detailed criteria to determine the middle ordinate value which will yield the necessary sight distance. The location of the noise barrier should be outside this distance.

17.10.2.3 Structural Design

All noise walls should either meet the criteria set forth in the AASHTO *Standard Specifications for Highway Bridges* and the AASHTO *Guide Specifications for Structural Design of Sound Barriers* or the Local Building Code, whichever yields the least costly approach.

The noise wall should be designed to withstand an 80-mph steady wind pressure with a 30-percent gust factor (i.e., 104 mph). The ODOT Bridge Division will be responsible for checking and ensuring that the wall meets the wind loading criteria. In addition to the wind loading criteria, the designer also needs to utilize the soil tests to determine the appropriate wall design.

17.10.2.4 Interference with Roadside Appurtenances

A noise barrier's proposed location could interfere with proposed or existing roadside features, including signs, sign supports, utilities and luminaire facilities. The designer must determine if these features are in conflict with the noise barrier and must coordinate with applicable ODOT Divisions to resolve any conflicts.

17.10.2.5 Maintenance Considerations

The location and design of a noise barrier should consider the following maintenance factors:

1. The noise barrier must be located so maintenance crews can easily access the wall for routine repairs.
2. The noise barrier should be constructed of materials that discourage vandalism (e.g., graffiti) and allow for easy cleaning.
3. The noise barrier should be designed so that any damage can be easily repaired.
4. The noise barrier should be located adjacent to the right-of-way line so that other maintenance operations can be reasonably performed (e.g., mowing, light bulb replacement, sign cleaning, spraying).

17.11 ENVIRONMENTAL PERMITS/ APPROVALS

Depending upon the specific project impacts, ODOT may need to obtain one or more environmental permits or approvals during project development. This section discusses those permits and approvals which may apply to a road design project. These include:

1. Section 404 Permit (Individual) Discharge of Dredged or Fill Material,
2. Floodplains Development Permit, (OWRB)
3. EIS, FONSI, Categorical Exclusions and Section 4(f) Approvals, and
4. National Pollutant Discharge Elimination System.

17.11.1 Definitions

1. Environmental Impact Statement (EIS). A document which is prepared when it has been determined that a project will have a significant impact on the environment.
2. Categorical Exclusion (CE). A classification for projects that will not induce significant environmental impacts or foreseeable alterations in land use, planned growth, development patterns, traffic volumes, travel patterns, or natural or cultural resources.
3. Environmental Assessment (EA). A study to determine if the environmental impacts of a project are significant, thus requiring the preparation of an EIS.
4. Finding of No Significant Impact (FONSI). A result of an EA that shows a project will not cause a significant impact to the environment.

17.11.2 Responsibilities

The Planning Division is responsible for assessing the environmental impacts of federally assisted highway construction projects and developing, reviewing and approving any special environmental mitigation measures deemed necessary for highway construction projects.

The Bridge Division is responsible for the preparation and submission of a Section 404 permit at or adjacent to bridges.

Where a permit or approval is required for a road design project, the road designer will, in most cases, be responsible for providing the necessary information to the appropriate Division to obtain the permit or approval.

17.11.3 Permit/Approval Information

For each permit or approval, the following basic information is provided in the following pages:

1. Title/Name,
2. Responsible Agency,
3. ODOT Responsible Unit,
4. Purpose,
5. Applicability, and
6. Information Needs.

NAME: Section 404 Permit (Individual) Discharge of Dredged or Fill Material

RESPONSIBLE AGENCY: United States Army Corps of Engineers

ODOT RESPONSIBLE UNIT: Bridge Division — Application
Planning Division — Mitigation

PURPOSE: To protect and preserve water quality and other environmental factors (e.g., impacts on fish and wildlife).

APPLICABILITY: Permit required for the placement of dredged or fill material into the nation's waters, including wetlands.

INFORMATION NEEDS: Permit application for Section 404 requires:

1. project description;
2. permit justification (i.e., in public interest);
3. engineering details (e.g., limit of fill activity, amount of fill, acreage taken, linear feet of disturbance, erosion control plan, disposal of waste material);
4. copy of EIS, EA/FONSI or CE, if prepared;
5. environmental impacts (e.g., soils, water quality, fish and wildlife, groundwater);
6. site photographs, including aerial if necessary (optional);
7. project schedule (design and construction);
8. mitigation plan;
9. other agencies for which a permit is needed and the permit status;
10. names and addresses of adjacent property owners; and
11. one set of drawings with:
    a. title sheet, and
    b. plan and elevation view.

NOTE: The Corps of Engineers, USDOD, makes the decision on the type of permit required and establishes guidelines for the use of the individual or nationwide permit.

NAME: Floodplains Development Permit

RESPONSIBLE AGENCY: Water Resources Board

ODOT REVIEW UNIT: Bridge Division for bridge structures/
Design Support Unit for waterways, channels, etc.

PURPOSE: To evaluate and regulate all potential encroachments onto the 100-year floodplain and/or regulatory floodway.

APPLICABILITY: Approval is required when a proposed road design project results in an encroachment onto the floodplain. The local government entity "City or County" requests an amendment to the floodway map, and the "City or County" government submits a letter of intent to FEMA and files for permit to FEMA in compliance with FEMA regulations. For ODOT projects prepared in-house, the designer will provide the information and coordinate the processing with the Bridge Division or Design Support Unit and the city or county responsible for FEMA compliance. For ODOT projects prepared by consultant contract, the consultant will provide all information, and the designer or project engineer will assist in the processing and coordination.

INFORMATION NEEDS: Floodplains Development Permit requires:

1. project description;
2. detailed hydraulic calculations to define the water surface profile of the 100-year flood assuming implementation of the proposed project design (i.e., any fill into the existing flood fringe area must be incorporated into the revised 100-year floodway model), the flood insurance study, FIS and FIRM maps;
3. engineering details (e.g., limits of fill activity, roadway cross section and profile);
4. copy of EIS, EA/FONSI, CE, the flood insurance study, FIS or FIRM maps, if prepared;
5. if applicable, name of project consultant;
6. project schedule;
7. mitigation plan, if required; and
8. one set of drawings with:
   a. title sheet, and
   b. plan and elevation view.

NAME: Environmental Impact Statement, FONSI, Categorical Exclusion and Section 4(f) Approvals*

RESPONSIBLE AGENCY: Federal Highway Administration (with coordination as needed with the State Historic Preservation Officer, Advisory Council on Historic Preservation and/or Department of Interior)

ODOT RESPONSIBLE UNIT: Planning Division

PURPOSE: To evaluate project environmental impacts, mitigation measures and to protect publicly owned parks, recreational areas, wildlife refuges and historic sites from use for transportation purposes without due consideration of other alternatives.

APPLICABILITY: These approvals are required on all Federal-aid and selected State projects. Those federal-aid projects that include the taking of any land from publicly owned parks, recreational areas, wildlife refuges and historic sites, which are determined to have national, state or local significance, also require the preparation of a Section 4(f) statement which may be prepared as part of an EIS or FONSI or may be a separate document.

INFORMATION NEEDS: These approvals require:

1. project description;

2. engineering and environmental impacts (Table 17.11A provides a listing of the information required in these reports and the responsible Division for providing this information.);

3. project schedule;

4. mitigation plan;

5. other permits required;

6. names and addresses of affected property owners; and

7. a set of drawings with:

   a. title sheet, and
   b. plan and elevation view.

* If Section 6(f) funds from the Federal Land and Water Conservation Act were used to improve any part of the site, and/or facilities, then replacement land and/or facilities of equal value must be provided for any land and/or facilities taken. The application of Section 6(f) will be addressed as part of the Section 4(f) statement.

Table 17.11A
EIS, FONSI, CE and 4(f) INFORMATION NEEDS

REQUIRED INFORMATION	RESPONSIBLE DIVISION
Logical Termini	Planning/Design
Description of Alternate Alignments	Planning/Design/Survey
Right-of-Way Needs for Each Alternate	Design
Adjacent Land Use Description	Planning/Survey
Cost Estimate for Each Alternate	
Construction	Design
Right-of-Way	Right-of-Way
Relocations	Right-of-Way
Utilities	Right-of-Way
Wetland Involvement	Planning
Hazardous Waste	Planning
Underground Storage Tanks Locations	Planning/Right-of-Way
Floodplain Impacts	Planning/Design
Water Quality Impacts	Planning
Air Quality Impacts	Planning
Wild and Scenic River Involvement	Planning
Wildlife Issues	Planning
Archaeological Survey	Planning
Visual Impacts	Planning
Historic Sites	Planning
Farmland Impacts	Planning
Economic Impacts	Planning
Aerial Photograph	Survey
Railroad Involvement	Design/Survey

NAME: National Pollutant Discharge Elimination System

(To be prepared at a later date.)

RESPONSIBLE AGENCY:

ODOT RESPONSIBLE UNIT:

PURPOSE:

APPLICABILITY:

INFORMATION NEEDS:

17.12 EPA STORMWATER DISCHARGE

At the time of publication, ODOT is still developing procedures which will meet current requirements for stormwater discharge into streams, rivers, lakes, etc. Once these procedures have been developed, they will be incorporated into this section. For additional information, the designer should contact the Planning Division.

17.13 WETLANDS

Highway, bridge or other projects which traverse wetlands require significant planning and design to mitigate any impacts the project will have on the wetlands. At the time of publication, ODOT is still developing appropriate wetland mitigation procedures that will meet current requirements. Once these procedures have been developed, they will be incorporated into this section. For additional information, the designer should contact the Planning Division.

17.14 EROSION CONTROL

17.14.1 General

Permanent erosion control measures are important factors in highway design. They involve a consideration of many elements, including:

1. natural drainage patterns;
2. hydrology and hydraulics;
3. local vegetative characteristics;
4. native soil properties (e.g., slope stability);
5. terrain characteristics;
6. agronomy;
7. impacts of sediment pollution (e.g., water quality);
8. landscape architecture;
9. construction costs; and
10. maintainability.

The designer should coordinate the road design project with the appropriate units within the Department for the needed expertise in permanent erosion control measures. The primary input will be from the Roadside Development Branch. This Section presents basic information and criteria which will apply to erosion control considerations in project design.

17.14.2 Road Design

Proper design of the roadway elements can significantly minimize erosion problems with the completed facility. The following should be considered.

17.14.2.1 Alignment

On new construction and reconstruction projects, the horizontal and vertical alignment should be properly blended and fitted to the natural landscape to minimize cut and fill sections. The highway alignment should be selected such that both ground and surface water can pass through the highway right-of-way or can be intercepted with a minimum disturbance to streams.

17.14.2.2 Cross Section

Side slopes of the roadway cross section should be as flat as practical and be consistent with local soil stability, climatic exposure, geology, the proposed landscape treatment and maintenance. With some types of soils (e.g., clay, gumbo), 3:1 or flatter slopes are mandatory to avoid erosion. The cross section shape should be varied, if necessary, to minimize erosion and to facilitate safety and drainage.

Severe erosion of earth slopes is usually caused by a concentration of storm water flowing from the area at the top of cut or fill slopes. A dike, preferably of borrow material to avoid disturbance of the natural ground, in conjunction with a grassed channel or paved ditch, should be considered at the top of the cut or fill to prevent water from running down the slope. Water can be spread over the natural slope or transported to lower elevations in chutes or, preferably, closed pipes. Outlets for high velocity chutes must be protected from scour.

In some cases, it may be advantageous to provide serrated cut slopes to aid in the establishment of vegetative cover. Serrations may be constructed in any material that can be ripped or that will hold a vertical face for a few weeks until vegetation becomes established. Where vegetation cannot be established or water flow down the fill slope is objectionable, provision should be made for collecting the runoff at the shoulder edge and directing it to an adequate inlet or chute.

In Oklahoma, side slopes steeper than 2:1 are too steep for grassing; therefore, concrete slope walls, vines or other ground cover will control erosion if properly adapted. Slopes between 2:1 and 6:1 accept grass if soil and climate are favorable; however, on slopes 2:1 to 3:1, the turf should be of the type which requires infrequent mowing. On 3:1 or flatter slopes, sprig sodding can be performed provided that soil and climate are satisfactory. Mulches may be required on all slopes. Interception ditches and dikes are recommended where the ground slopes toward the cut slopes. If these interceptions are not provided, the runoff may wash the topsoil which will likely eliminate any form of erosion control. Shoulder curbs are advisable on steep, long, superelevated fill slopes to eliminate the fill erosion problems.

17.14.2.3 Drainage

The design of the roadway drainage system is critical to minimizing erosion. Roadway drainage appurtenances include roadside ditches, channels, chutes and culverts. *ODOT Drainage Manual* provides information on the proper hydraulic design of roadway drainage elements.

17.14.2.4 Soils

Soil texture influences erosion control methods in several ways:

1. sandy or silty soils are susceptible to both wind and water erosion and may erode so rapidly that erosion control measures should begin as soon as practical; however, fine-textured soils such as clays do not erode as rapidly;

2. sandy or silty slopes will often require protection during the time between construction of the slope and its sodding or seeding (e.g., vegetative mulch over the topsoil);

3. sandy or silty slopes may require special protection during the time interval between planting and establishment; options include concrete ditch liner or sod alone on 2:1 channel slopes or vegetative mulch over bermuda sprigs on 6:1 or 4:1 slopes; and

4. a sod ditch liner may be theoretically feasible in a roadway ditch but unsuitable in actual practice because of washing out of the sod before it grows deep roots and becomes well established; in this case, a concrete liner may be necessary rather than sod.

Low fertility soils should be fertilized at the time bermuda sod and sprigs are established and at regular intervals thereafter. Naturally fertile soil should be turfed economically by seeding or sprigging in lieu of expensive mulch sodding.

Slowly permeable soils usually require special treatment. There are some partial solutions such as chemical soil conditions (e.g., gypsum), flattening of slopes, or mechanical manipulation which will increase the rate of water infiltration. These operations should be performed before turfing operations.

17.14.2.5 Vegetation

Every effort should be made to use vegetation that will survive in a particular area with minimum maintenance. The use of grasses or other plants for landscaping and erosion control which are not ecologically adapted to a particular area usually results in poor erosion control and increased maintenance. Proven soil conservation practices, including the use of mulches and temporary protective measures, are all important in developing permanent vegetative covers. Irrigation is often required to establish ground cover or maintain a satisfactory stand in semi-arid areas. Dust palliatives may be effective in erosion control, particularly in arid areas where wind erosion is a problem. Section 17.14.3 discusses specific types of plantings which are adaptable to the geographic regions in Oklahoma.

In addition to the selection of compatible plantings, the designer should limit the area of unprotected soil exposure and should limit the duration of exposure.

17.14.3 Landscaping

17.14.3.1 ODOT Procedures

The Roadside Development Branch has the primary responsibility for determining the landscaping treatment for road design projects. During the plan-in-hand field inspection on major projects, an Agronomist or Landscape Architect will typically attend the inspection to determine the landscaping treatment for the project. The Roadside Development Branch will submit recommendations and landscaping details to the road designer for incorporation into the project design.

17.14.3.2 General

Roadside landscaping can greatly enhance the aesthetic value of a highway. Landscaping treatments should be considered early in project development so that they can be easily and inexpensively incorporated into the project design. Landscaping treatments will be considered on a project-by-project assessment. The designer should also reference the AASHTO *A Guide for Highway Landscape and Environmental Design* for more information on landscaping.

17.14.3.3 Benefits

Roadside landscaping can be designed advantageously to yield several benefits. The most important objective is to fit the highway naturally into the existing terrain. The existing landscape should be retained to the extent practical. The following is a brief discussion of the benefits of proper landscaping:

1. <u>Aesthetics</u>. Gentle slopes, mountains, parks, bodies of water and vegetation have an obvious aesthetic appeal to the highway user. Landscaping techniques can be used effectively to enhance the view from the highway. In rural areas, the landscaping should be natural and should eliminate construction scars. The planting shape and spacing should be irregular to avoid a cosmetic appearance.

   In urban areas, the smaller details of the landscape predominate and plantings become more formal. The interaction between the occupants of slow-moving vehicles and pedestrians with the landscape determines the scale of the aesthetic details. In some cases, the designer may be able to provide walking areas, small parks, etc. Landscaping should be pleasant, neat and sometimes ornamental, and it should require low maintenance.

2. <u>Erosion</u>. As discussed in Section 17.14.2, landscaping and erosion control are interrelated. Flat and rounded slopes and vegetation serve to both prevent erosion and provide aesthetic value.

3. <u>Maintenance</u>. Landscaping decisions will greatly affect roadside maintenance. Maintenance activities for mowing, fertilizing and using herbicides should be considered when designing the roadside landscape.

4. Other Benefits. When properly coordinated with the highway design, landscaping may provide additional benefits. See Section 17.14.3.8 for more information.

17.14.3.4 Geographic Application (Grasses)

Oklahoma can be divided into five belts from west to east, based on effective precipitation, and into three zones from north to south, based on effective temperature. This creates a total of 11 geographical regions. Figure 17.14A presents the regions, and it can be used as a general guide to the use of plants for erosion control. The following discusses those types of grasses which are recommended for use in each region.

1. Regions 1 and 2. "Guymon" bermuda-grass seeding with asphalt mulching overtop has been very effective in Regions 1, 2 and 3A. Other regions are still being evaluated. The key to success is heavy watering directly prior to application of the asphalt mulch. Since rhizome spread is relatively slow, two seasons may be required to completely cover an area. For sandy soil, little bluestem, sand bluestem, sideoats grama and sand lovegrass are adapted; for fine-textured soil, bluegrama, sideoats grama and buffalograss are adapted. Caucasian bluestem is adapted to most soils. Western wheatgrass is suitable for areas with better than average moisture relationships. Seeded areas must be protected with a non-competitive mulch.

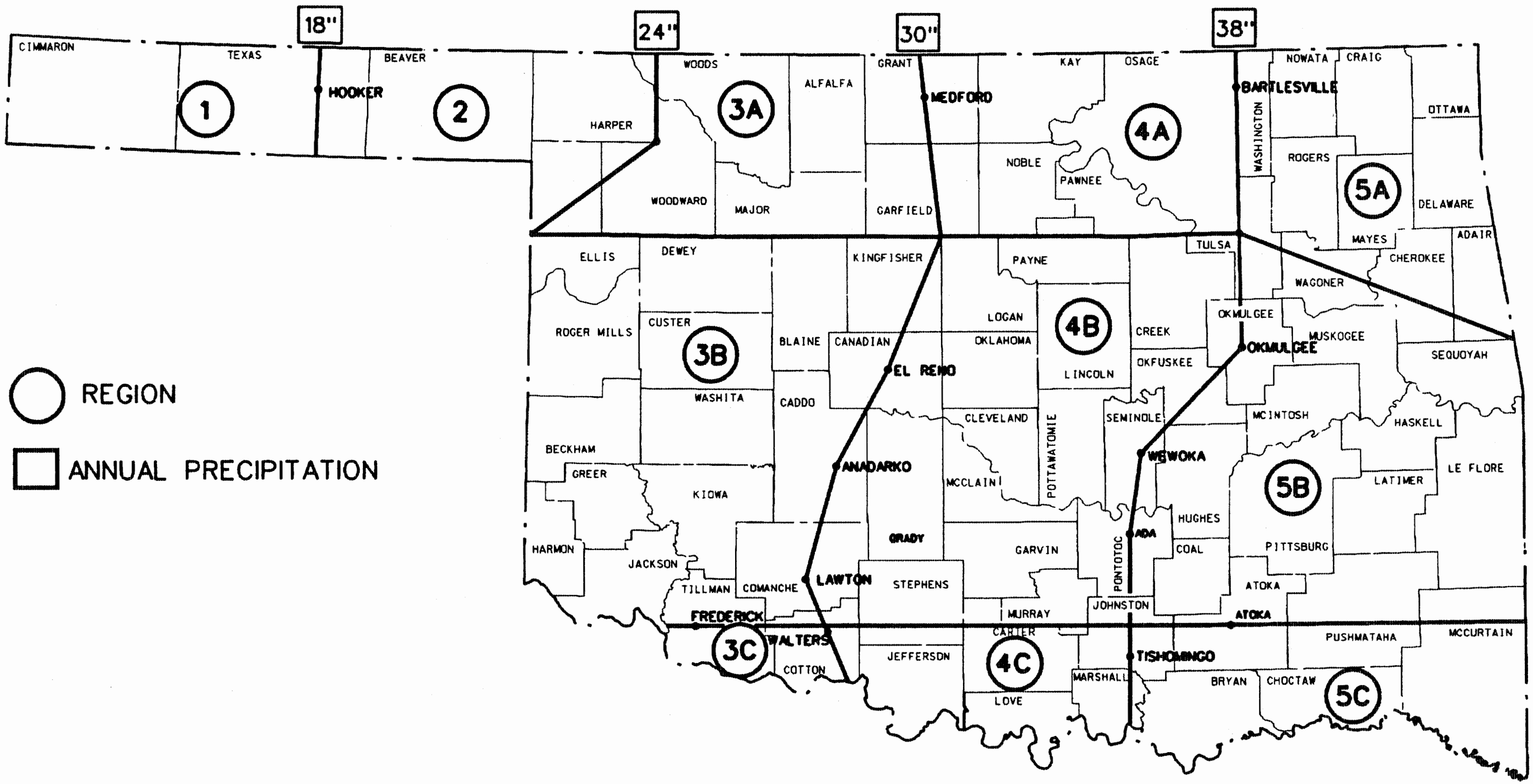
2. Regions 3A, 3B and 3C. Both bermuda and non-bermuda grasses can be used. All seeded areas must be protected with a non-competitive vegetative mulch.

   In Region 3A and the northern two-thirds of Region 3B, bermuda grass is recommended for soils with better than average moisture relationships. This includes relatively flat areas, ditch bottoms and areas along the shoulder that receive additional water from roadway runoff. In all of Belt 3, bermuda can be established by mulch sodding during spring and early summer, but sprigging is more reliable when performed during March-April-May. In Region 3C and the southern third of Region 3B, bermuda is adapted to most soils with average or better moisture relationships. Bermuda can be established in Region 3C by seeding during April and May. Bermuda seeding in Region 3B is not recommended because the grass will likely freeze out.

   The following species can be established by seeding: Little bluestem, sand bluestem, sand lovegrass and switchgrass are adapted to sandy soils; bluegrama, sideoats grama and buffalo are adapted to fine-textured soils; weeping lovegrass is adapted to a wide variety of soils; King Ranch bluestem and Caucasian bluestem are both adapted to a wide variety of soils, but Caucasian bluestem should be used north of Interstate 40 and King Ranch bluestem used south.

3. Regions 4A, 4B and 4C. Both bermuda and non-bermuda grass species can be used. All seeded areas should be protected with a non-competitive vegetative mulch.

   Bermuda can be used on most soils if at least moderately fertile or fertilized, provided soil permeability conditions are not unfavorable. It may be established either by sodding or sprigging. Mulch sodding may be carried out during the spring and early summer, but sprigging operations should be carried out during the spring. Bermuda can be seeded during April and May in Region 4C, but



GEOGRAPHIC REGIONS FOR GRASSES

Figure 17.14A

seedlings will likely be winter killed in Regions 4A and 4B unless confined to south-facing slopes.

Weeping lovegrass is well adapted throughout the belt and will grow on most soils if properly fertilized. King Ranch bluestem will grow on poor soil and is adapted south of Interstate 40. Adapted native grasses that may be established by seeding are little bluestem, switchgrass and sideoats grama for sandy soils, and buffalograss and sideoats grama for fine-textured soils.

4. Regions 5A, 5B and 5C. Bermuda grass, native and tame grasses and ground covers can be used effectively in this belt. If grasses are seeded under optimum conditions, protective mulches may not be necessary.

   Bermuda is adapted to all moderately fertile or fertilized soil if at least moderately well drained. It can be established by mulch sodding during the spring and up to about mid-summer and by sprigging during the spring. Bermuda can be seeded during April and May in Region 5C.

   Tall fescue can be fall seeded throughout the belt if kept on deep, moist and at least moderately fertile soil; big bluestem, little bluestem, indiangrass and switchgrass can be seeded throughout the belt and will grow on a wide range of soil types; Korean lespedeza, a reseeding annual legume, is well adapted to the rocky slopes of Region 5A. Striata lespedeza, a reseeding annual legume, is adapted in Regions 5B and 5C; and sericea lespedeza is adapted to the poor acid soils of Regions 5B and 5C. Weeping lovegrass will grow on many of the soils throughout the belt. Short leaf pine is very effective in controlling erosion on steep, rock slopes in Region 5B.

5. Availability of Good Turfing Material. In western Oklahoma, sprig sod is satisfactory, but slab and mulch sod are excessively sandy. In northern Oklahoma, bermuda of any form is scarce; in other parts of the State, bermuda is available but contaminated with Johnson grass or other noxious weeds. Seeds such as weeping lovegrass and some forms of native grass seeds are periodically scarce, and a redesign might be required based on availability.

17.14.3.5 Plant Establishment Policy

Projects which include planting may have a special provision which requires the contractor to be responsible for a plant establishment period of at least one year. Longer establishment periods may be required where survival is considered essential to the function of the plantings (e.g., junkyard screening, urban landscaping).

17.14.3.6 Protection of Existing Vegetation

The Department's general policy is that, wherever practical, existing trees and other landscaping features will not be removed on highway projects. This objective, however, must be compatible with other considerations such as roadside safety, geometric design, utilities, terrain, public acceptance and costs. The plans should clearly designate all existing landscape features which will be saved.

In the event that the existing plant material conflicts with these considerations, where applicable, the plant material should be evaluated by an ODOT Agronomist or Landscape Architect for possible relocation to a more suitable portion of the right-of-way.

17.14.3.7 Disturbed Areas

In areas disturbed by construction work, the designer should specify that the turf be reestablished. Turf establishment refers to the revegetation of disturbed areas. The designer should use the guidance in the following comments to determine the appropriate turf establishment, depending upon individual site conditions:

1. Topsoil. Topsoil is typically placed in disturbed areas to a depth of 5 inches.

2. Planting of Grass. All areas disturbed by construction, except exposed rock surfaces and areas to be sodded, will be limed, seeded, fertilized and mulched, regardless of the presence or absence of topsoil. See ODOT *Standard Specifications* for additional details.

3. Sodding. Where developed properties and/or areas of intensive mowing abut the highway project, all adjacent areas disturbed by construction may be slab sodded and watered sufficiently to establish growth.

17.14.3.8 Coordination with Design Elements

All landscaping activities should be properly coordinated with other project design elements. The objectives are two-fold: 1) other design elements should not be compromised by landscaping, and 2) secondary benefits may be gained by the proper application of the landscaping features. Examples of coordination between landscaping and project design are briefly discussed:

1. Geometric Design. On new construction and reconstruction projects, the geometric design of the highway should be blended to fit the natural topography and landscaping features of the area. As practical, existing landscaping elements should be preserved and enhanced. As discussed in Section 17.14.2, the roadway cross section design should be compatible with the landscaping objectives. Other examples of potential impacts on geometric design are intersection and horizontal sight distance. Landscaping features should not interfere with either element.

2. Roadside Safety. The introduction of landscaping features into the project should not compromise the objectives of roadside safety. Chapter Eleven presents the Department's criteria for roadside safety design. The most significant roadside safety element is clear zones relative to the use of landscaping features. No roadside hazards should be located within the designated clear zones. Trees are considered a roadside hazard if its diameter *at maturity* will be 4 inches or greater.

3. Screening for Headlight Glare. Depending upon roadway alignment and the selected type of vegetation, landscaping features may be used to effectively screen headlight glare in, for example, freeway medians.

4. Screening for Noise Abatement. Although the effect may be more psychological than real, landscaping features may have some benefit in masking undesirable noise.

5. Screening of Undesirable Views. Screening of junkyards and/or other undesirable views may be practical through the use of landscaping features.

6. Snow and Sand Drift. Landscaping features may assist in preventing snow

and sand from drifting and accumulating on the roadway.

17.15 REFERENCES

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

2. *Transportation and Traffic Engineering Handbook*, Institute of Transportation Engineers, 1982.

3. *Source Parking*, ENO Foundation, 1957.

4. NCHRP Report 155, *Bus Use of Highways; Planning and Design Guidelines*, Transportation Research Board, 1975.

5. *Guide for Development of New Bicycle Facilities*, AASHTO, 1981.

6. *Guide for Development of New Bicycle Facilities*, Draft, July 1990.

7. *Pedestrian and Bicycle Planning with Safety Considerations*, Transportation Research Record 1141, Transportation Research Board, 1987.

8. *Uniform Federal Accessibility Standards*, Federal Register, August 7, 1984.

9. FHWA-IP-84-6 *Guidelines for Making Pedestrian Crossings Structures Accessible*, FHWA, 1984.

10. *A Guide on Safety Rest Areas for the National System of Interstate Highways and Streets*, AASHTO, 1968.

11. NCHRP Synthesis 20, *Rest Areas*, Transportation Research Board, 1973.

12. FHWA-IP-81-1, *Safe Rest Areas: Planning, Location and Design*, FHWA, 1981.

13. *Compilation and Evaluation of Rest Area Issues and Designs*, Transportation Research Record 1224, Transportation Research Board, 1989.

14. *Profile of Highway Rest Area Usage and Users*, Transportation Research Record 1224, Transportation Research Board, 1989.

15. *A Guide for Erecting Mailboxes on Highways*, AASHTO, 1984.

16. *Standard Specifications for Highway Bridges,* AASHTO, 1989, with interims.

17. *Guide Specifications for Structural Design of Sound Barriers*, AASHTO, 1989, with interims.

18. *A Guide for Highway Landscape and Environmental Design*, AASHTO, 1970.

Chapter Eighteen

Non-Standard Construction Features (New, Revised or Experimental)

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Chapter Eighteen

NON-STANDARD CONSTRUCTION FEATURES (New, Revised or Experimental)

18.1 GENERAL

The highway industry in general and ODOT specifically utilizes a myriad of products, processes and methodologies to accomplish its goal of building highways and streets.

Most of the products, processes and methods have evolved over time from practical applications of well proven materials and construction techniques. In general, these products, processes and methods have produced very satisfactory results. However, due to heightened concern over cost, highway and motor carrier safety, traffic congestion and pavement and structural deterioration, it is imperative that ODOT and contractors utilize the best technology available. The highway industry is continually developing new materials and new technology which may have a beneficial impact on the management and performance of the highway system.

The following lists several methods on how new or existing products may be developed or refined:

1. products and/or processes may be developed specifically for highway construction;
2. products and/or processes may be developed or evolved from an existing acceptable product;
3. products and/or processes may be adapted from other industries (e.g., building construction); and
4. products and/or processes may be developed, improvised or adapted by agency personnel to solve specific problems (e.g., maintenance personnel).

The designer continuously needs to expand his knowledge of these products or procedures to ensure that the best technology is used in design. There are several methods of obtaining this knowledge, including:

5. contacting the appropriate Division (e.g., Research and Development Division, Materials Division, Construction Division, Maintenance Division and/or appropriate personnel in the Field Division) for their expertise;
6. reviewing research reports published by AASHTO, TRB, FHWA, ITE and other groups;
7. reviewing field test results from demonstration projects;
8. attending workshops/courses offered by ODOT, FHWA, universities, suppliers, etc.; and/or
9. contacting the supplier directly for samples, specifications and publications.

The following sections provide information on ODOT's Experimental Features Program, ODOT's Divisional Testing and Evaluation, and the role of Special Provisions in these projects.

18.2 ODOT'S EXPERIMENTAL FEATURES PROGRAM

18.2.1 General

18.2.1.1 Overview

The experimental features program is an FHWA sponsored program that allows ODOT to innovatively use new materials, processes, methods, etc., on Federal-aid highway construction projects. This section briefly describes the experimental program used by ODOT.

18.2.1.2 Definition

An experimental feature is defined as a material, process, method, equipment item, traffic operation device or other feature that:

1. has not been sufficiently tested under actual service conditions to merit acceptance, without reservation, for normal transportation construction; or

2. has already been accepted but includes alternate acceptable features which need testing to determine their relative merits under comparable conditions.

Experimental features are incorporated into Federal-aid highway construction projects to determine the suitability of the features as standard construction items.

18.2.2 Implementation Procedures

18.2.2.1 Initialization

At the time of the decision to construct, install or otherwise incorporate an experimental feature into an existing construction contract, the initiating Division notifies the Research & Development Division, in writing, of its proposal to develop such a project. The proposal for an experimental feature shall include:

1. name of the project engineer;

2. project information such as location, project number, expected submission date, etc.;

3. description of the feature to be incorporated into the construction project;

4. reasons for the selection of the particular type or brand and/or a description of the methods that will be used that differ from normal construction practices;

5. aspects to be evaluated;

6. criteria for acceptance of the feature; and

7. description of the control sections or other alternative procedure that are needed to provide satisfactory performance comparisons.

18.2.2.2 Responsibilities

18.2.2.2.1 Research and Development Division

ODOT's participation in the FHWA experimental features program is administered through the Research and Development Division. The Division's responsibilities include:

1. incorporating experimental projects into the Annual Research Program;

2. developing and submitting a Work Plan to FHWA for any experimental feature incorporated in a construction project; and

3. implementing the Work Plan.

18.2.2.2.2 Design Divisions

Relative to the use of experimental features, the project engineer within the applicable Design Division is responsible for:

1. ensuring that all proposals for experimental features in construction projects are initiated with sufficient lead time to properly develop all necessary materials and procedures;

2. working with the Research and Development Division, as needed, in the development and implementation of the Work Plan;

3. developing with the Research and Development Division's cooperation, any necessary special provisions, plan notes, design details, etc., to incorporate into the final PS&E assembly; and

4. notifying the Research and Development Division of any changes in the project status, or any modifications to the plans and/or specifications.

18.3 ODOT'S DIVISIONAL TESTING AND EVALUATION

Most product or process testing, evaluation or applications are the direct responsibility of the Research and Development Division or the Materials Division. However, the Division Engineer in conjunction with the project engineer may decide to test or "include" a particular product or procedure on a sample project. This inclusion may be a possible solution to a specific problem or to test a product used elsewhere. Often, these inclusions fall outside of the Research and Materials Divisions' responsibilities and must be evaluated by the Division requesting the product or procedure.

Although the Research and Materials Divisions' concurrence is not required, the designer is encouraged to contact these Divisions for additional information and guidance. These Divisions may have valuable information that may lead to a successful test program. For example, they may have additional research reports, know of local suppliers or have test results from other States. In addition, they can also help the designer set up an appropriate evaluation program and determine the applicable sample size that will produce statistically significant results.

If practical, the Research and Materials Divisions should be included in evaluation and data collection of a product or procedure. Their involvement will help produce acceptable test results and will encourage information sharing among the various Divisions. If these Divisions are not included in the evaluation of the product or process, they should be provided with a copy of the final evaluation report.

Before including a product or procedure on a project, the designer should consider the effect it may have on construction and maintenance. Unique construction procedures may prohibitively increase the cost of the product. Specialty items may require extensive inventories which may require substantial storage space or may not be available for timely maintenance if stored elsewhere.

The designer should also be aware that, with most new products, the manufacturer and/or local supplier may offer it to ODOT for testing purposes at no or at a substantially reduced cost. The designer should contact the local supplier or manufacturer for additional information.

The various Field Divisions and Maintenance Units should be contacted early in the selection process for their input. Their involvement in the evaluation process is critical to the success of this testing and evaluation procedure.

18.4 SPECIAL PROVISIONS (Specifying Guidance)

18.4.1 General

A special provision is required for all new, revised or modified products and procedures that are not covered in the *ODOT Standard Specifications for Highway Construction* (Standard Specifications) with Supplemental Specifications. These special provisions, and the plan notes and special detail drawings, provide guidance to the contractor on how to properly use, construct and bid the new, revised or modified product or procedure.

To resolve any possible discrepancies among the contract documents, Sections 104.02 and 105.04 of the *Standard Specifications* define the order of authority among the contract documents as follows:

1. Special Provisions,
2. Plans (Notes and Details),
3. Supplemental Specifications, and
4. *Standard Specifications*.

The designer should note that the most recent contract items (Plans and Special Provisions) take precedent over the earlier published *Standard Specifications* and Supplemental Specifications.

As Special Provisions change, evolve and are commonly used, they may become adopted by the Specifications Committee for inclusion into the Supplemental Specifications and ultimately into the latest version of the *Standard Specifications*.

18.4.2 Special Provision Preparation

The designer is referred to ODOT Policy Directive D-402-2 for guidance on writing, reviewing and obtaining approval for specifications, special provisions and plan notes. This directive provides guidance on the standard format, preferred wording and content for these contract documents. In addition, it also provides the procedure for obtaining the Specification Engineer's approval for these items.

In addition to Directive D-402-1, the designer should consider the following when developing new special provisions:

1. Generic. The designer should strive to write the special provision and/or plan notes in a manner considered to be generic so that they may apply to several items versus a singular proprietary item. This stimulates competitive bidding versus having the contractor paying user fees for the proprietary item. On the other hand, for a very few products or processes, it may be cost effective to stipulate in the special provision the use of a proprietary item. However, the designer must ensure that there are no other comparable products or processes available.

2. Exclusions. In an effort to stimulate development, distribution and usage of innovative products or processes, the special provision may inadvertently create a lockout situation for other suppliers. The designer must ensure that the special provision includes the opportunity for other suppliers or distributors to provide an "equal" product.

3. Availability. In stipulating a product, the designer needs to consider the availability of the product. Few and distant manufacturing plants, limited technical experts and few local or regional distributors may result in higher initial or long-term costs.

4. Existing Special Provisions. Often existing special provisions may be applicable with little or no revisions. Before developing a

new special provision, the designer should contact the Specification Engineer to determine if an existing specification may be acceptable.

5. Manufacturer Specifications. The designer may often use the manufacturer's product specifications to compose the special provision with the appropriate technical and manufacturing data. However, the designer must be careful in writing the special provision so that it does not to exclude other reasonable alternatives (see Comment No. 2).

6. Plan Notes. Material specifications and other highly technical data should not be included in the plan notes but, instead, incorporated into the special provisions. Changes to the process or product rarely are reflected in the plan notes but often are reflected in the latest version of the special provisions.

18.5 REFERENCES

1. *Standard Specifications for Highway Construction*, Oklahoma Department of Transportation, 1988, and Supplement, 1991.

2. Policy Directive No. D-402-1, *Construction Specifications*, Oklahoma Department of Transportation, May 12, 1983.

3. Policy Directive No. D-402-2, *Writing, Reviewing, and Approving Specifications, Special Provisions and Plan Notes*, Oklahoma Department of Transportation, July 6, 1983.

Chapter Nineteen

Manual Updates

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Chapter Nineteen

MANUAL UPDATES

19.1 GENERAL

ODOT has developed the *ODOT Roadway Design Manual* to reflect its design policies and procedures as of the date of its initial publication. To meet ODOT's changing needs and to remain current, the *Design Manual* must be continuously updated. A static *Design Manual* quickly becomes of little value to the users because it will soon be superseded by changes in the Department's actual operations. Consequently, ODOT has determined that a concerted effort is necessary to maintain this document.

Revisions to the *Design Manual* will be issued periodically by the Assistant Director — Design or his designee. The designee will be responsible for evaluating changes in the highway design literature (e.g., the issuance of new research publications) and for determining the need for revisions in the ODOT *Design Manual*. In addition, it is vital that the users of the *Manual* inform the designee of any inconsistencies, errors, need for clarification or new ideas so that the *Design Manual* may reflect the best and most up-to-date information practical.

19.2 TEMPORARY UPDATES

As new policies, procedures, directives and/or criteria are developed, it is often desirable to provide this information quickly to the designer. Because the *Design Manual* will typically only be updated on an annual basis, this information often needs to be published before it can formally be incorporated into the *Design Manual*. To expedite these changes, this information will typically be issued to the designer on ODOT memorandum. The user should insert these temporary memoranda near the appropriate *Manual* sections until the *Manual* can be updated.

To keep this *Manual* current, the user should use the registration card provided in the front of the *Manual* for registration with ODOT's Assistant Director — Design. In addition, the user should also inform ODOT of any changes in ownership or address.

19.3 MANUAL UPDATES

19.3.1 Manual Format

The *Design Manual's* format was developed to facilitate changes quickly and easily. Some of these format features are as follows:

1. Loose-Leaf Binder. The loose-leaf binder was selected so that pages can be easily added, removed or replaced at a later date. This allows any changes to be directly incorporated into the *Manual*.

2. Page Numbering. The page numbering system is re-initiated for each major section of the *Manual* (e.g., 13.1(1), 13.1(2), 13.2(1), 13.2(2)...). This was developed so that, if there are pages

added to or deleted from a section, only that section needs to be reissued and not the entire chapter.

3. Revision Date. In the upper right-hand corner of every page is the latest revision date for that page. For the original submission of the *Design Manual*, this date is July 1992. As the *Manual* is updated and new pages are issued, the revision date will reflect the latest issuance date for that page (e.g., June 1993).

19.3.2 Updating Procedures

To revise the ODOT *Design Manual*, the following procedure will be used:

1. Users should submit, in writing, all requests for possible revisions, corrections, additions, errata, omissions, etc., of this *Design Manual* to the Assistant Director — Design or his designee.

2. The designee of the Assistant Director — Design will determine if there is a need to update the *Manual*. This will usually be done on an annual basis or as a specific need arises. The designee will review all material and prepare any changes or additions to match the *Design Manual*'s format. The designee will then submit the proposed changes or corrections to the Assistant Director — Design for his approval, and, if appropriate, to the FHWA for their review and/or concurrence.

3. The Assistant Director — Design will approve all changes to the *Design Manual*.

4. After the changes have been approved, the designee will issue the changes to all holders of the *Design Manual* on the Design Manual Transmittal Memorandum. Figure 19.3A provides the format of the Design Manual Transmittal Memorandum.

5. It is the user's responsibility to maintain his *Design Manual* in an up-to-date condition. Once the user receives the Design Manual Transmittal Memorandum, the user should immediately make the changes and/or corrections to his *Design Manual*. The Design Manual Transmittal Memorandum then should be inserted at the end of Chapter Nineteen. See Section 19.4.

19.4 DESIGN MANUAL TRANSMITTAL MEMORANDA

For a complete history of the changes made to the *Design Manual*, the Design Manual Transmittal Memoranda should be retained in the back of Chapter Nineteen.

Oklahoma Department of Transportation

Design Manual Transmittal Memorandum	Date

To:	All *Design Manual* Holders
From:	Assistant Director - Design
Subject:	*Design Manual* Update

Please make the following changes in your *Design Manual* :

Page(s)	Revision Date	Remarks (substitution of material)

Place this instruction sheet in Section 19.4.

Distributed by: Rural Design Division Design Support Branch	Signature (Assistant Director - Design)

DESIGN MANUAL TRANSMITTAL MEMORANDUM
Figure 19.3A

Chapter Twenty

Glossary/Index

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Chapter Twenty

GLOSSARY/INDEX

20.1 GLOSSARY

20.1.1 General

1. 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 a building, structure or area, including the site on which such building, structure or area is located, wherein specific services are provided or activities performed.

2. Arterials. Highways which are characterized by a capacity to quickly move relatively large volumes of traffic but are often restricted capacity to serve abutting properties. The arterial system typically provides for high travel speeds and the longest trip movements.

3. Authority. The Oklahoma Turnpike Authority.

4. Average Running Speed. The average running speed is the distance summation for all vehicles over a specified section of highway divided by the running time summation for all vehicles.

5. Average Travel Speed. Average travel speed is the distance summation for all vehicles divided by the total time summation for all vehicles.

6. 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.

7. 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.

8. 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 are to be shared with other transportation modes.

9. Board. The board of county commissioners of a county.

10. Bridge. A structure, including supports, erected over a depression or obstruction, such as water, a highway, or a railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 ft between undercopings of abutments or spring lines or arches or extreme ends of openings for multiple boxes; may include multiple pipes where the clear distance between openings is less than half of the smaller contiguous opening.

11. Bridge Length. The length of a bridge structures is the overall length measured along the line of survey stationing back to the back of backwalls of abutments, if

present, otherwise end to end of the bridge floor; but in no case less than the total clear opening of the structure.

12. Bridge Roadway Width. The clear width of the structure measured at right angles to the center of the roadway between the bottom of curbs or, if curbs are not used, between the inner faces of parapet or railing.

13. Bridge to Remain in Place. An "existing bridge to remain in place" refers to any bridge work which does not require the total replacement of both the substructure and superstructure.

14. Collectors. Highways which are characterized by a roughly even distribution of their access and mobility functions.

15. Commission. The Oklahoma Transportation Commission.

16. County Road System. All rural roads which are not on the State highway system.

17. Crosswalk. A marked lane for passage of pedestrians, bicycles, etc., traffic across a road.

18. Department. The Oklahoma Department of Transportation.

19. Design Speed. Design speed is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern.

20. Direct Connection. A ramp that does not deviate greatly from the intended direction of travel.

21. Directional Interchanges. An interchange where one or more left-turn movements are provided by direct connection, even if the minor left-turn movements are accommodated by loops.

22. Directional Islands. Islands which control and direct traffic movements and guide the driver into the proper channel.

23. Director. The chief executive officer of the Oklahoma Department of Transportation.

24. Divided Highway. A highway with separated roadways for traffic moving in opposite directions.

25. Divisional Islands. Islands which separate opposing traffic flows, alert the driver to the crossroad ahead and regulate traffic through the intersection.

26. Driveway. A private road giving access from a public way to a building or abutting grounds.

27. 85th-Percentile Speed. The speed below which 85 percent of vehicles travel on a given highway.

28. Experimental Feature. A material, process, method, equipment item, traffic operation device or other feature that has not been sufficiently tested under actual service conditions to merit acceptance, without reservation, for normal transportation construction; or has already been accepted but includes alternate acceptable features which need testing to determine their relative merits under comparable conditions.

29. Expressway. A divided arterial highway for through traffic with full or partial control of access and generally with grade separations at major intersections.

30. Freeways. 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.

31. Frontage Road. A road constructed adjacent and parallel to but separated from the highway and connected thereto at least at each end, for service to abutting property and for control of access.

32. Fully Directional Interchange. An interchange where all left-turn movements are provided by direct connections.

33. Governing Body. The legislative authority of any city or town for all streets and highways within the corporate limits of such city or town; and the board of county commissioners of each county as to all county highways; and the Oklahoma Transportation Commission as to all State highways.

34. Grade Separation. A crossing of two highways, or a highway and a railroad, at different levels.

35. Highway, Street or Road. A general term denoting a public way for purposes of vehicular travel, including the entire area within the right of way. (*Recommended usage: in urban areas* - highway or street, *in rural areas* - highway or road).

36. Interchange. A system of interconnecting roadways in conjunction with one or more grade separations, providing for the movement of traffic between two or more roadways on different levels.

37. Intersection. The general area where two or more highways join or cross, within which are included the roadway and roadside facilities for traffic movements in that area.

38. Local Road (Oklahoma Statutes). A road constructed to provide access to property abutting on or adjacent to the highway and which has but one connection to the highway.

39. Local Roads and Streets (AASHTO). All public roads and streets not classified as arterials or collectors.

40. Loop Ramp. A one-way roadway that curves about 270 degrees to the right to accommodate a left-turning movement.

41. Municipal Corporation or Municipality. All cities and towns organized under the laws of Oklahoma, but shall not include any other political subdivisions.

42. Municipal Street System. The street system which consists of all local city streets within the corporate limits not on the State highway system.

43. National Highway System (NHS). A system of highways determined to have the greatest national importance to transportation, commerce and defense in the United States.

44. Off-System. Roads and streets which are any public facilities not on any Federal-aid system and not a Federal-aid road.

45. Operating Speed. A term commonly used to characterize prevailing vehicular speeds on a highway segment, either through field measurements of speed or through informal field observations.

46. Overpass. A grade separation where the subject highway passes over an intersecting highway or railroad.

47. Parkway. An arterial highway for non-commercial traffic, with full or partial control of access, and usually located

within a park or a ribbon of park-like development.

48. Posted Speed Limit. The recommended speed limit for a highway as determined by engineering and traffic investigations.

49. Public Road (Oklahoma Statutes). A road constructed to connect other public roads or streets, but not connected to the highway.

50. Ramp. A short roadway connecting two or more legs of an intersection or connecting a frontage road and main lane of a highway.

51. Refuge Islands. Islands at or near crosswalks which aid or protect pedestrians crossing a wide roadway.

52. Roadway. (General) The portion of a highway including shoulders, for vehicular use. A divided highway has two or more roadways. (Construction) The portion of a highway within limits of construction.

53. Running Speed. 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).

54. Rural Areas. Those places outside the boundaries of urban areas.

55. Semi-Direct Connection. A ramp that is indirect in alignment, yet more direct than a loop ramp.

56. Semi-Directional Interchange. An interchange where one or more left-turn movements are provided by semi-direct connections, even if the minor left-turn movements are accommodated by loops.

57. Shared Roadway. Any roadway upon which a bicycle lane is not designated and which may be legally used by bicycles regardless of whether such facility is specifically designated as a bikeway.

58. State Highway System. The highway system under the jurisdiction of the Oklahoma Department of Transportation consisting of those intercounty and interstate highways, including their extensions through incorporated areas, as designated by the Oklahoma Transportation Commission.

59. Substructure. All that part of a bridge structure below the bearings of simple and continuous spans, skewbacks or arches and top of footings of rigid frames; including backwalls, wingwalls and wing protection railings.

60. Superstructure. All that part of a bridge structure above the bearings of simple and continuous spans, skewbacks of arches and top of footings of rigid frames; excluding backwalls, wingwalls and wing protection railings.

61. Surface Transportation Program (STP). Federal funds set aside for Federal-aid roads which are not on the NHS and are not functionally classified as a minor rural collector or a local road or street.

62. Turning Roadways. Channelized areas (separated by an island) at at-grade intersections which allow a moderate-speed, free-flowing right turn.

63. Underpass. A grade separation where the subject highway passes under an intersecting highway or railroad.

64. Urban Areas. Those places within boundaries set by the responsible State

and local officials having a population of 5000 or more.

20.1.2 Qualifying Words

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

2. Criteria. A term typically used to apply to design values, usually with no suggestion on the criticality of the design value.

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

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

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

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

7. 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.

8. 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.

9. 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.

10. 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.

11. 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.

12. 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.

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

14. 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. (Note: Not to be confused with "Standard Drawings.")

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

16. 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.

17. 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.

20.1.3 Abbreviations

1. AASHTO. American Association of State Highway and Transportation Officials.

2. ANSI. American National Standards Institute.

3. APWA. American Public Works Association.

4. AREA. American Railway Engineering Association.

5. ASCE. American Society of Civil Engineers.

6. ASTM. American Society of Testing and Materials.

7. COE. Corps of Engineers, USDOD.

8. FAA. Federal Aviation Administration.

9. FEMA. Federal Emergency Management Agency.

10. FHWA. Federal Highway Administration.

11. HCM. *Highway Capacity Manual.*

12. HEC. Highway Engineering Circulars and Hydraulic Engineering Center, USDOD, COE, Davis California.

13. HUD. Housing and Urban Development.

14. IES. Illuminating Engineering Society.

15. ITE. Institute of Transportation Engineers.

16. ISTEA. Intermodal Surface Transportation Efficiency Act of 1991.

17. MUTCD. *Manual of Uniform Traffic Control Devices.*

18. NEMA. National Electrical Manufacturers Association.

19. NCHRP. National Cooperative Highway Research Program.

20. OAC. Oklahoma Aeronautics Commission.

21. OCC. Oklahoma Corporation Commission.

23. ODOT. Oklahoma Department of Transportation.

24. OSHA. Occupational Safety and Health Administration.

25. OTA. Oklahoma Turnpike Authority.

26. OTC. Oklahoma Transportation Commission.

27. OWD. Oklahoma Wildlife Department.

28. TRB. Transportation Research Board.

29. UFAS. Uniform Federal Accessibility Standards.

30. USDOD. United States Department of Defense.

31. USDOT. United States Department of Transportation.

20.1.4 Project/Plan Development

1. ARTEMIS®. A proprietary computerized project management system used by ODOT to track project schedules, resources and finances.

2. Award. The acceptance by the Commission of a bid.

3. Consultant. A firm or person, hired by ODOT to conduct special studies or design projects.

4. Contractor. A company or firm hired by ODOT to construct the project in the field according to the plans and specifications.

5. Designer. The person who performs the majority of the project design work. Depending upon the project type, the designer may be from the Bridge Division, Rural or Urban Design Divisions, Traffic Engineering Division or the Consultant.

6. Engineer's Estimate. The ODOT cost estimate for construction of a project.

7. Force Account Work. Prescribed work paid for on the basis of actual costs and appropriate additives.

8. Letting (Bid Opening). The time appointed for the opening of the proposals submitted by bidders.

9. Notice to Proceed. Written notice given to the contractor to begin the contract work.

10. Plan-in-Hand Field Inspection. A multi-purpose function during which the Project Engineer presents his proposals for consideration by others, checks for compatibility with existing features, assures completeness of survey, determines the maintenance of traffic during construction, discusses the new facility with the local political subdivision and advises of any road or street closure and checks the compatibility of plans.

11. Plans. The contract drawings which show the location, character and dimensions of the prescribed work, including layouts, profiles, cross sections and other details.

12. Project. An undertaking by the Oklahoma Transportation Commission, governing body or other governmental instrumentality for highway construction, including preliminary engineering, acquisition of right-of-way and actual construction, or for highway planning and research, or for any other work or activity to carry out the provisions of the law for the administration of highways.

13. Project Engineer. The person in charge or responsible for the design of a project.

14. Proposal. The written offer of the bidder to perform the work described in the Plans and Specifications, and to furnish

the labor and materials at the prices quoted by the bidder.

15. Project Scoping Team. A group of individuals assembled from various Divisions to review and identify the basic project design parameters, major design exceptions and environmental requirements.

16. Public Hearing/Meeting. A meeting conducted by ODOT to inform the general public on the Department's proposed plan of action or design proposal.

17. Scoping Report. A report prepared by the Project Scoping Team defining the project scope of work, project costs, proposed procedures, major design exceptions, etc.

18. Special Provisions. Additions and revisions to the *Standard* and *Supplemental Specifications* applicable to an individual project.

19. Specifications. The compilation of provisions and requirements for the performance of prescribed work.

20. Standard Drawings. Drawings approved for repetitive use, showing details to be used where appropriate.

21. Standard Specifications. *ODOT Standard Specifications for Highway Construction*. A book of specifications approved for general application and repetitive use.

22. Supplemental Specifications. Approved conditions and revisions to the *Standard Specifications.*

20.1.5 Planning

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

2. 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.

3. Capacity. The maximum number of vehicles which can reasonably be expected to traverse a point or uniform section of a road during a given time period under prevailing roadway, traffic and control conditions.

4. Categorical Exclusion (CE). A classification for projects that will not induce significant environmental impacts or foreseeable alterations in land use, planned growth, development patterns, traffic volumes, travel patterns, or natural or cultural resources.

5. Delay. The primary 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.

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

7. Design Hourly Volume (DHV). The 1-hour volume in both directions of travel in the design year selected for determining the highway design.

8. (Design) Service Flow Rate. The maximum hourly vehicular volume which can pass through a highway element at the selected level of service.

9. Directional Design Hourly Volume (DDHV). The 1-hour volume in one direction of travel during the DHV.

10. Directional Distribution (D). The division, by percent, of the traffic in each direction of travel during the DHV, ADT or AADT.

11. Environmental Assessment (EA). A study to determine if the environmental impacts of a project are significant, thus requiring the preparation of an EIS.

12. Environmental Impact Statement (EIS). A document which is prepared when it has been determined that a project will have a significant impact on the environment.

13. Finding of No Significant Impact (FONSI). A result of an EA that shows a project will not cause a significant impact to the environment.

14. K. The proportion of AADT occurring in the design hour. K will vary based on the hour selected for design and the characteristics of the specific highway facility.

15. Level of Service (LOS). A qualitative concept which has been developed to characterize acceptable degrees of congestion as perceived by motorists.

16. New Construction. Horizontal and vertical alignment construction on new location.

17. Peak-Hour Factor (PHF). A ratio of the volume occurring during the peak hour to the peak rate of flow during a given time period within the peak hour (typically, 15 minutes).

18. Project Scope of Work. The basic intent of the highway project which determines the overall level of highway improvement.

19. Rate of Flow. The 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 (typically, 15 minutes).

20. Reconstruction. 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 the highway will remain essentially within the existing highway corridor.

21. 3R Projects (Freeways). Resurfacing, restoration, and/or rehabilitation projects on existing freeways. These projects are primarily intended to extend the service life of the existing facility and to enhance highway safety.

22. 3R Projects (Non-Freeways). Rehabilitation, restoration and/or resurfacing projects on non-freeways facilities. These projects are primarily intended to extend the service life of the existing facility and to enhance highway safety.

23. Spot Improvements. Improvements which are intended to correct an identified deficiency at an isolated location.

24. Truck Factor (T). A factor which reflects the percentage of heavy vehicles (trucks, buses and recreational vehicles) in the traffic stream during the DHV, ADT and/or AADT.

20.1.6 Geometric

1. Auxiliary Lane. The portion of the roadway adjoining the through traveled way for parking, speed change, turning, storage for turning, weaving, truck climbing or for other purposes supplementary to through traffic movement.

2. 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.

3. Barrier Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Barrier curbs may range in height between 6 inches and 12 inches with a face steeper than 1 horizontal to 3 vertical.

4. Comfort Criteria. Criteria which is based on the comfort effect of change in vertical direction in a sag vertical curve because of the combined gravitational and centrifugal forces.

5. Critical Length of Grade. The maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed.

6. Cross Slope Rollover. The algebraic difference between the slope of the through lane and the slope of the adjacent pavement within the traveled way or gore.

7. Curb Offset. On curbed facilities, the portion of the roadway section from the edge of travel lane to the gutter line when that distance is less than 4 ft.

8. Cuts. Sections of highway located below natural ground elevation thereby requiring excavation of earthen material.

9. Decision Sight Distance (DSD). The length of highway required to safely stop a vehicle traveling at design speed where information is difficult to perceive or where unexpected maneuvers are required.

10. Depressed Median. A median that is lower in elevation than the traveled way and so designed to carry a certain portion of the surface water.

11. Design Vehicle. The vehicle used to determine turning radii, off-tracking characteristics, pavement designs, etc.

12. Fill Slopes. Slopes extending outward and downward from the edge of the shoulder to intersect the natural ground line.

13. Flush Median. A median where its vertical elevation above the surface of the adjacent roadway pavement is 1 inch or less.

14. Gore Area. The paved triangular area between the through lane and the exit ramp, plus the graded area which may extend a few hundred feet downstream beyond the gore nose.

15. Grade Slopes. The rate of slope between two adjacent VPI's expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in ft for each 100 ft of horizontal distance. Upgrades in the direction of stationing are identified as plus (+). Downgrades are identified as minus (-).

16. Graded Shoulder Width. The width of the shoulder measured from the edge of

travelway to the intersection of the shoulder slope and fill slope planes.

17. Intersection Sight Distance (ISD). The sight distance along the major highway required for a vehicle on the minor highway to safely turn onto or cross the major highway.

18. K-Values. The horizontal distance needed to produce a 1% change in gradient.

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

20. Low-Speed Urban Streets. Streets within an urban or urbanized area where the design speed (V) ≤45 mph and low truck volumes.

21. Maximum Superelevation (e_{max}). The overall superelevation control used on a specific facility. Its selection depends on several factors including climatic conditions, terrain conditions, type of area (rural or urban) and highway functional class.

22. Median. The portion of a divided highway separating the traveled ways for traffic in opposite directions.

23. Mountable Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Mountable curbs have a height of 6 inches or less with a face no steeper than 1 horizontal to 3 vertical.

24. Mountainous Terrain. Longitudinal and transverse changes in elevation are abrupt, and benching and side hill excavation are frequently required to provide the desirable highway alignment.

25. Normal Crown (NC). The typical cross section on a tangent section (i.e., no superelevation).

26. Open-Roadway/High-Speed Highways. All rural facilities and any urban facilities where the design speed (V) >45 mph.

27. Outer Separation. The portion of an arterial highway between the traveled ways of a roadway for through traffic and a frontage street or road.

28. Parking Lane. An auxiliary lane primarily for the parking of vehicles.

29. PC. Point of curvature (beginning of curve).

30. PCC. Point of compound curvature.

31. PI. Point of intersection of tangents.

32. POST. Point on sub-tangent.

33. PRC. Point of reverse curvature.

34. Profile Grade Line. A series of tangent lines connected by vertical curves. It is typically placed along the roadway centerline of undivided facilities and at the two median edges on divided facilities.

35. PT. Point of tangency (end of curve).

36. Raised Median. A median which contains a raised portion (greater than 1 inch) within its limits.

37. Reverse Crown (RC). A superelevated roadway section which is sloped across the entire traveled way in the same direction and at a rate equal to the cross slope on a tangent section.

38. Roadside. A general term denoting the area adjoining the outer edge of the

roadway. Extensive areas between the roadways of a divided highway may also be considered roadside.

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

40. Shoulder. The portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use, and for lateral support of base and surface courses.

41. Sidewalk. That portion of the roadway primarily constructed for the use of pedestrians.

42. Stopping Sight Distance (SSD). The length of highway required to safely stop a vehicle traveling at design speed.

43. Superelevation (S). The amount of cross slope or "bank" provided on a horizontal curve to help counter-balance the outward pull of a vehicle traversing the curve.

44. Superelevation Breakover. The algebraic difference (A) between the superelevated travel lane slope and shoulder slope on the outside of a horizontal curve.

45. Superelevation Runoff (L). The change in cross slope from the end of the tangent runout (adverse crown removed) to a section that is sloped at the design superelevation (S).

46. Superelevation Transition Length. The distance required to transition the roadway from a normal crown section to the full superelevation (S) needed. Superelevation transition length is the sum of the tangent runout and super-elevation runoff.

47. Tangent Runout (TR). The change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed.

48. Travel/Traffic Lane. The portion of the traveled way for the movement of a single line of vehicles.

49. Traveled Way. The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

50. Turning Template. A graphic representation of a design vehicle's turning path for various angles of turns.

51. Usable Shoulder Width. The width of the shoulder than can be used by a driver for emergency parking or stopping.

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

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

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

20.1.7 Roadside Safety

1. Back Slope. The side slope created by the connection of the ditch bottom, upward and generally outward, to the natural ground.

2. Barrier Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Barrier curbs may range in height between 6 inches and 12

inches with a face steeper than 1 horizontal to 3 vertical.

3. Barrier Warrant. A criterion that identifies an area of concern which should be shielded by a traffic barrier, if judged to be practical.

4. Concrete Median Barrier (CMB). See Median Barrier.

5. Critical Parallel Slope. Slopes which cannot be safely traversed by a run-off-the-road vehicle. For most embankment heights, fill slopes steeper than 3:1 are considered critical.

6. Cut Slope. The side slope created by going directly upward and outward from the shoulder edge to the natural ground.

7. Experimental System. A roadside barrier, end terminal or crash cushion which has performed satisfactorily in full-scale crash tests, but has not yet been installed in sufficient quantity and/or has not been exposed to traffic long enough to evaluate its in-service performance adequately.

8. Fill Slope. The side slope created by connecting the graded shoulder at hinge point to the ditch bottom or natural ground line, downward and outward.

9. Fore Slope. The side slope created by connecting the graded shoulder at the hinge point to the ditch bottom, downward and outward.

10. Impact Angle. (Longitudinal Barriers) The angle between a tangent to the face of the barrier and a tangent to the vehicle's path at impact. (Crash Cushions) The angle between the axis of symmetry of the crash cushion and a tangent to the vehicle's path at impact.

11. Impact Attenuator (Crash Cushion). A traffic barrier used to safely shield fixed objects or other hazards from approximately head-on impacts by errant vehicles.

12. 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.

13. Median Barrier. A longitudinal barrier used to prevent an errant vehicle from crossing the portion of a divided highway separating the traveled ways for traffic in opposite directions.

14. Mountable Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Mountable curbs have a height of 6 inches or less with a face no steeper than 1 horizontal to 3 vertical.

15. Non-Recoverable Parallel Slope. Slopes which can be safely traversed but upon which an errant motorist is unlikely to recover. For most embankment heights, if a fill slope is between 3:1 (inclusive) and 4:1 (exclusive), it is considered a non-recoverable parallel slope.

16. Operational System. A roadside barrier, end terminal or crash cushion which has performed satisfactorily in full-scale crash tests and has demonstrated satisfactory in-service performance.

17. Parallel Slopes. Cut and fill slopes for which the toe runs approximately parallel to the flow of traffic.

18. Portable Concrete Median Barrier (PCMB). See Median Barrier. Normally used in construction zones.

19. 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.

20. Roadside. A general term denoting the area adjoining the outer edge of the roadway. Exterior areas between roadways of a divided highway may also be considered roadside.

21. Roadside Barrier. A longitudinal barrier used to shield hazards located within an established clear zone. Roadside barriers include guardrail, concrete barriers, etc.

22. Roadside Clear Zone (CZ). The distance beyond the edge of travel lane that should be clear of any non-traversable hazards or fixed objects.

23. Roadside Hazard. A general term to describe a roadside feature which cannot be safely impacted by a run-off-the-road vehicle. Roadside hazard types include both fixed objects and non-traversable roadside features.

24. Run/Rise Ratio or Slope Value. The relative steepness on a slope normal to the edge of the traveled way expressed as a ratio of run to rise.

25. Shy Distance. Distance from the edge of the 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.

26. Toe of (Fill) Slope. The intersection of the fill slope or fore slope with the natural ground or ditch bottom, before the recommended rounding is applied.

27. Top of (Cut) Slope. The intersection of the back slope with the natural ground, before the recommended rounding is applied.

28. Transverse Slopes. Cut and fill slopes for which the toe runs approximately perpendicular to the flow of traffic. Transverse slopes are typically formed by intersections between the mainline and driveways, median crossovers or side roads.

20.1.8 Traffic Signals

1. Controller. (1) (Traffic) A device that controls the sequence and duration of indications displayed by traffic signals; (2) (computer) under computer supervision, a device that switches the signal circuits according to the computer's instructions.

2. Coordination. The establishment of a definite timing relationship between adjacent traffic signals.

3. Cycle Length. The time required for one complete sequence of signal indications.

4. Delay. (1) A measure of the time that elapsed between the stimulus and the response; (2) traffic delay -- the time lost by vehicles due to traffic friction or control devices.

5. Demand. The need for service; for example, the number of vehicles desiring to use a given segment of roadway during a specified unit of time.

6. Detection. The process used to identify the presence or passage of a vehicle at a specific point or to identify the presence of one or more vehicles in a specific area.

7. Detector. A device for indicating the presence or passage of vehicles or pedestrians (e.g., loop detector, microloop detector, calling detector, pushbutton, etc.).

8. Interval. A discrete portion of the signal cycle during which the signal indications remain unchanged.

9. Interval Sequence. Specifies the order in which the various intervals are displayed.

10. Interval Timing. The passage of time that occurs during an interval.

11. Loop Detector. A device capable of sensing a change in inductance of a loop sensor imbedded in the roadway caused by the passage or presence of a vehicle over the loop.

12. Offset. The time difference or interval in seconds between the start of the green indication at one intersection as related to the start of the green interval at another intersection or from a system time base.

13. Pattern. A unique set of traffic parameters (cycle, split and offset) associated with each signalized intersection within a predefined group of intersections.

14. Phase. A part of the traffic signal time cycle allocated to any combination of traffic movements receiving right-of-way simultaneously during one or more intervals.

15. Phase Overlap. Refers to a phase that operates concurrently with one or more other phases.

16. Phase Sequence. The order in which a controller cycles through all phases.

17. Point Detection. The detection of a vehicle as it passes a point or spot on a street or highway.

18. Preemption. The term used when the normal signal sequence at an intersection is interrupted and/or altered in deference to a special situation such as the passage of a train, bridge opening or the granting of the right-of-way to an emergency vehicle.

19. Presence Detection. The ability of a vehicle detector to sense that a vehicle, whether moving or stopped, has appeared in its field.

20. Recall. An operational mode for an actuated intersection controller whereby a phase, either vehicle or pedestrian, is displayed each cycle whether demand exists or not. Usually a temporary or emergency situation.

21. Split. A percentage of the cycle length allocated to each of the various phases in a signal sequence.

22. Yield. The action of allowing a semi-actuated controller, or an actuated controller operating in the semi-actuated mode, to terminate the main street phase so as to begin satisfying existing cross-street demand.

20.1.9 Highway Lighting

1. Average Initial Illuminance. The average level of horizontal illuminance on the

pavement area of a traveled way at the time the lighting system is installed when lamps are new and luminaires are clean; expressed in average footcandles for the pavement area.

2. Average Maintained Illuminance. The average level of horizontal illuminance on the roadway pavement when the output of the lamp and luminaire is diminished by the maintenance factors; expressed in average footcandles for the pavement area.

3. Candela. The unit of luminous intensity. Formerly the term "candle" was used.

4. Candlepower. A measure of luminous intensity in a specified direction; expressed in candelas.

5. Equipment Factor (EF). A factor used in the illuminance or luminance calculations which compensates for light losses due to normal production tolerances of commercially available luminaires when compared with laboratory photometric test models.

6. Footcandle. The illuminance on a surface one square foot in area on which there is uniformly distributed a light flux of one lumen.

7. Footlambert. The unit of photometric brightness (luminance). It is equal to $1/\pi$ candela per square foot, or the uniform luminance of a perfectly diffusing surface emitting or reflecting light at the rate of one lumen per square foot.

8. Glare. The optical sensation produced by luminance within the visual field that is sufficiently greater than the luminance to which the eyes are adapted to cause annoyance, discomfort or loss in visual performance and visibility.

9. Illuminance. The density of the luminous flux incident on a surface. It is the quotient of the luminous flux by the area of the surface when the latter is uniformly illuminated.

10. Lamp Lumen Depreciation Factor (LLD). A depreciation factor that indicates the decrease in a lamp's initial lumen output over time. For design calculations, the initial lamp lumen value is reduced by a lamp lumen depreciation factor (LLD) to compensate for the anticipated lumen reduction. This factor is usually found in the manufacturer's test data.

11. Longitudinal Roadway Line. A line along the roadway parallel to the curb or shoulder line.

12. Lumen. A unit of measure of the quantity of light. One lumen is the amount of light which falls on an area of one square foot every point of which is one foot from the source of one candela (candle). A light source of one candela emits a total of 12.57 lumens.

13. Luminaire. A complete lighting unit consisting of a lamp or lamps together with the parts designed to distribute the light, to position and protect the lamps and to connect the lamps to the power supply.

14. Luminaire Dirt Depreciation Factor (LDD). A depreciation factor that indicates the expected reduction of a lamp's initial lumen output due to the accumulation of dirt on or within the luminaire over time.

15. Luminance. The luminous intensity of any surface in a given direction per unit of projected area of the surface as viewed from that direction.

16. Luminous Efficiency. The quotient of the luminous flux emitted by the total lamp power input. It is expressed in lumens per watt.

17. Maintenance Factor (MF). A combination of factors used to denote the reduction of the illumination for a given area after a period of time compared to the initial illumination on the same area (MF = EF + LLD + LDD).

18. Mounting Height. The vertical distance between the roadway surface and the center of the light source in the luminaire.

19. Nadir. The vertical axis which passes through the center of the luminaire light source.

20. Spacing. The distance in feet between successive lighting units.

21. Transverse Roadway Lane. Any line across the roadway that is perpendicular to the curb or shoulder line.

22. Uniformity of Illuminance. The ratio of average footcandles of illuminance on the pavement area to the footcandles at the point of minimum illuminance on the pavement. It is commonly called the uniformity ratio. A uniformity ratio of 3:1 means that the average footcandle value on the pavement is three times the footcandle value at the point of least illuminance on the pavement.

23. Uniformity of Luminance. The Average Level-to-Minimum Point method uses the average luminance of the roadway design area between two adjacent luminaires. The luminance uniformity (avg./min. and max./min.) considers traveled portion of the roadway, except for divided highways having different designs on each side.

24. Visibility. The quality or state of being perceivable by the eye. In outdoor applications, visibility is defined in terms of the distance at which an object can be just perceived by the eye.

25. Veiling Luminance. A luminance superimposed on the retinal image which reduces its contrast. It is this veiling effect produced by bright sources or areas in the visual field that results in decreased visual performance and visibility.

20.1.10 Right-of-Way

1. Abandonment. The relinquishment of the public interest in right-of-way or activity thereon with no intention to reclaim or use again for highway purposes.

2. Access Control. The condition where the public authority fully or partially controls the right of abutting owners to have access to and from the public highway.

3. Acquisition or Taking. The process of obtaining right-of-way.

4. Channel Right-of-Way. Right-of-Way acquired specifically for channel construction and maintenance, which provides the State with a permanent right of ingress and egress. The property owner relinquishes the right to modify the channel dimensions (e.g., slopes).

5. Control by Regulation. A practice exercised by ODOT on the State highway system and by local jurisdictions on other facilities to determine where private interests may have access to and from the public road system.

6. Full Control. The authority to control access is exercised to give preference to through traffic by providing access

connections with selected frontage roads or local roads only and by prohibiting crossings at grade or direct driveway connections.

7. Improvement. Any dwelling, out-building, other structure or fence, or part thereof, but not including public utilities, which lie within an area to be acquired for highway purposes.

8. Limited Access Facility. Includes limited access highways, expressways, arterial highways, frontage roads, public roads, and the auxiliary service highway.

9. Limited Access Highway. A street or highway especially designed for through traffic, and over, from, or to which neither owners nor occupants of abutting lands nor other persons have any right or easement of access, light, air or view.

10. Partial Control. The authority to control access is exercised to give preference to through traffic to a degree that, in addition to access connections with selected frontage or local roads, there may be some crossing at grade and some private driveway connections.

11. Permanent Right-of-Way. Right-of-Way acquired for permanent ownership by the State for activities which are the responsibility of the State for an indefinite period of time. The State obtains the title to the property.

12. Right of Access. The right of ingress to a highway from abutting land and egress from a highway to abutting land.

13. Right of Survey Entry. The right to enter property temporarily to make surveys and investigations for proposed highway improvements.

14. Right-of-Way. A general term denoting land, property, or interest therein, usually a strip acquired for or devoted to a highway use.

15. Right-of-Way Appraisal. A determination of the market value of property including damages, if any, as of a specified date, resulting from an analysis of facts.

16. Right-of-Way Easements. Right-of-Way acquired with the perpetual right to construct and maintain a public highway and incidental facilities.

17. Right-of-Way Estimate. An approximation of the market value of property including damages, if any, in advance of an appraisal.

18. Severance Damages. Loss in value of the remainder of a parcel resulting from an acquisition.

19. Temporary Right-of-Way. Right-of-Way acquired for the legal right of usage by the State to serve a specific purpose for a limited period of time (e.g., maintenance and protection of traffic during construction). Once the activity is completed, the State yields its legal right of usage and returns the land to its original condition as close as practical.

20.1.11 Drainage

1. Allowable Headwater. The depth or elevation of the impoundment of cross-drainage flow above which damage or some other unfavorable result could occur.

2. Annual Flood. The highest peak discharge in a water year.

3. Backwater. The increase in water-surface profile, relative to the elevation occurring under natural channel and floodplain conditions, induced upstream from a structure, bridge or culvert, that obstructs or constricts a channel. It also applies to the water surface profile in a channel or conduit.

4. Base Flood. The 100-Year flood.

5. Bridge. A structure including supports erected over a depression or an obstruction, such as water, highway or railway, and having a tract or passageway for carrying traffic or moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes.

6. Catch Basin. A structure with a sump for inletting drainage from a gutter or median and discharging the water through a conduit. In common usage it is a grated inlet with or without a sump.

7. Channel. The bed and banks that confine the surface flow of a natural or artificial stream. Braided streams have multiple subordinate channels, which are within the main stream channel.

8. Cover. The extent of soil above the crown of a pipe or culvert.

9. Cross Drainage. The runoff from contributing drainage areas both inside and outside the highway right-of-way and the transmission thereof from the upstream side of the highway facility to the downstream side.

10. Culvert. A structure which is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through embankments. A structure, as distinguished from bridges, which is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Also, a structure which is 20 feet or less in centerline length between extreme ends of openings for multiple boxes.

11. Design Discharge or Flow. The rate of flow for which a facility is designed.

12. Design Flood Frequency. The recurrence interval that is expected to be accommodated without contravention of the adopted design constraints. The return interval (recurrence interval or reciprocal of probability) used as a basis for the design discharge.

13. Discharge. The rate of the volume of flow of a stream per unit of time, usually expressed in cubic feet per second.

14. Flanking Inlets. Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purposes of these inlets are to intercept debris as the slope decreases and to act as relief of the inlet at the low point.

15. Floodplain. The alluvial land bordering a stream, formed by stream processes, that is subject to inundation by floods.

16. Freeboard. The vertical distance between the level of the water surface, usually corresponding to design flow and a point of interest such as a low chord of a bridge beam or specific location on the roadway grade.

17. Headwater (H_w). That depth of water impounded upstream of a culvert due to the influence of the culvert construction, friction and configuration.

18. Hydraulics. The characteristics of fluid mechanics involved with the flow of water in or through drainage facilities.

19. Hydrology. The study of the occurrence, circulation, distribution and properties of the waters of the earth and its atmosphere.

20. Inlet. A structure for capturing concentrated surface flow. May be located along the roadway, in a gutter, in the highway median or in a field.

21. Intensity. The rate of rainfall upon a watershed, usually expressed in inches per hour.

22. Outfall. The point location or structure where drainage discharges from a channel, conduit or drain.

23. Peak Discharge. (1) The highest value of discharge attained by a flood. (2) Maximum discharge rate on a runoff hydrograph for a given flood event.

24. Spread. The accumulated flow in and next to the roadway gutter.

25. Tailwater (TW). The depth of flow in the stream directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway constriction. Term is usually used in culvert design and is the depth measured from the downstream flow line of the culvert to the water surface.

26. Time of Concentration (T_c). The time it takes water from the most distant point (hydraulically) to reach a watershed outlet. T_c varies, but often used as constant.

20.1.12 Pavement Design

1. Analysis Period. The period of time for which the economic analysis is to be made; ordinarily will include at least one rehabilitation activity.

2. Base Course. The layer or layers of specified or selected material of designed thickness placed on a subbase or a subgrade to support a surface course.

3. Composite Pavement. A pavement structure composed of an asphalt concrete wearing surface and Portland Cement Concrete slab; an asphalt concrete overlay on a PCC slab is also referred to as a composite pavement.

4. Construction Joint. A joint made necessary by a prolonged interruption in the placing of concrete.

5. Contraction Joint. A joint normally placed at recurrent intervals in a rigid slab to control transverse cracking.

6. Deformed Bar. A reinforcing bar for rigid slabs conforming to "Requirements for Deformations," in AASHTO Designations M31, M42 or M53.

7. Dowel. A load transfer device in a rigid slab, usually consisting of a plain round steel bar.

8. Drainage Coefficients. Factors used to modify layer coefficients in flexible pavements or stresses in rigid pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration.

9. Expansion Joint. A joint located to provide for expansion of a rigid slab, without damage to itself, adjacent slabs, or structures.

10. Flexible Pavement. A pavement structure which maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.

11. Load Transfer Device. A mechanical means designed to carry loads across a joint in a rigid slab.

12. Longitudinal Joint. A joint normally placed between traffic lanes in rigid pavements to control longitudinal cracking.

13. Maintenance. The preservation of the entire roadway, including surface, shoulders, roadsides, structures, and such traffic control devices as are necessary for its safe and efficient utilization.

14. Panel Length. The distance between adjacent transverse joints.

15. Pavement Rehabilitation. Work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing material and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy. This could include the complete removal and replacement of the pavement structure.

16. Pavement Structure. A combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.

17. Performance Period. The period of time that an initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal serviceability; this is also referred to as the *design period*.

18. Prepared Roadbed. In-place roadbed soils compacted or stabilized according to provisions of applicable specifications.

19. Pumping. The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.

20. Reinforcement. Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.

21. Resilient Modulus. A measure of the modulus of elasticity of roadbed soil or other pavement material.

22. Rigid Pavement. A pavement structure which distributes loads to the subgrade, having as one course a Portland Cement Concrete slab of relatively high-bending resistance.

23. Roadbed. The graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder.

24. Roadbed Material. The material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.

25. Select Borrow. Material meeting the requirements provided for in the *Standard Specifications*, with specific soil groups, group characteristics, or sandrock

formation requirements as specified in the construction plans.

26. Serviceability. The ability at time of observation of a pavement to serve traffic (autos and trucks) which use the facility.

27. Subbase. The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course (or in the case of rigid pavements, the Portland Cement Concrete slab).

28. Subgrade. The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

29. Surface Course. One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called "wearing course."

30. Tandem Axle Load. The total load transmitted to the road by two consecutive axles whose centers may be included between parallel vertical planes spaced more than 40 inches and not more than 96 inches apart, extending across the full width of the vehicle.

31. Tie Bar. A deformed steel bar or connector embedded across a joint in a rigid slab to prevent separation of abutting slabs.

32. Triple Axle Load or Tridem Axle Load. The total load transmitted to the road by three consecutive axles whose centers may be included between parallel vertical plans spaced more than 40 inches and not more than 96 inches apart, extending across the full width of the vehicle.

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