PREFACE

Purpose
Many manuals, policies, guides, standards etc. have been published regarding roadway design. Most of these have been written using wide ranges of design recommendations (minimums and maximums) since the contents were intended to apply nationally. The purpose of this manual is to reduce the selection of design alternatives to those most appropriate for the State of Ohio, to document Ohio's interpretation of various policies, and to include design criteria which may be unique to the State of Ohio.

Application
The criterion included in this manual has been developed to closely conform to the following publications:

- AASHTO - A Policy on Geometric Design of Highways and Streets (2011 Green Book)
- AASHTO - A Policy on Design Standards - Interstate System (2005)
- TRB Report 214 - Designing Safer Roads - Practices for Resurfacing, Restoration and Rehabilitation (1987)
- AASHTO - Roadside Design Guide (2011)
- AASHTO – Guide for the Development of Bicycle Facilities (2012)

This manual is neither a textbook nor a substitute for engineering knowledge, experience or judgment. It is intended to provide uniform procedures for implementing design decisions, assure quality and continuity in design of highways in Ohio, and assure compliance with Federal criteria. Although the manual is considered a primary source of reference by personnel involved in highway design in Ohio, it must be recognized that the practices suggested may be inappropriate for some projects because of fiscal limitations or other reasons.

Consideration must also be given to design standards adopted by city, county or other local governments when designing facilities under their jurisdiction.

In lieu of the geometric design guidelines presented in this manual, the geometric design guidelines presented in the **AASHTO publication "Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT $\leq$ 400)" (2001)** may be used for very low-volume local roads with a design average daily traffic volume of 400 vehicles per day or less.

Preparation
The Roadway Design Manual has been developed by the Office of Roadway Engineering. Errors and omissions should be reported to the Office Administrator, Office of Roadway Engineering, Ohio Department of Transportation, 1980 West Broad Street, Columbus, Ohio, 43223.

Format and Revisions
Updating the manual is intended to be a continuous process and revisions will be issued periodically.

Each page has its publishing date shown. Users are encouraged to keep their copies up to date. Updates are available for viewing or downloading only from ODOT's **Design Resource Reference Center**, found on ODOT's web page. ODOT's Internet address is ***http://www.dot.state.oh.us.***

Unit of Measure

Plans are to be prepared using the English system of units. Any metric units are provided for reference only.

For design purposes, the relationships between the two units are not exact or interchangeable. The user is therefore cautioned to work entirely within one system and not attempt to convert directly between the two.

OHIO COUNTIES

County	Code	District	County	Code	District
Adams	ADA	9	Licking	LIC	5
Allen	ALL	1	Logan	LOG	7
Ashland	ASD	3	Lorain	LOR	3
Ashtabula	ATB	4	Lucas	LUC	2
Athens	ATH	10			
Auglaize	AUG	7	Madison	MAD	6
			Mahoning	MAH	4
Belmont	BEL	11	Marion	MAR	6
Brown	BRO	9	Medina	MED	3
Butler	BUT	8	Meigs	MEG	10
			Mercer	MER	7
Carroll	CAR	11	Miami	MIA	7
Champaign	CHP	7	Monroe	MOE	10
Clark	CLA	7	Montgomery	MOT	7
Clermont	CLE	8	Morgan	MRG	10
Clinton	CLI	8	Morrow	MRW	6
Columbiana	COL	11	Muskingum	MUS	5
Coshocton	COS	5			
Crawford	CRA	3	Noble	NOB	10
Cuyahoga	CUY	12			
			Ottawa	OTT	2
Darke	DAR	7			
Defiance	DEF	1	Paulding	PAU	1
Delaware	DEL	6	Perry	PER	5
			Pickaway	PIC	6
Erie	ERI	3	Pike	PIK	9
			Portage	POR	4
Fairfield	FAI	5	Preble	PRE	8
Fayette	FAY	6	Putnam	PUT	1
Franklin	FRA	6			
Fulton	FUL	2	Richland	RIC	3
			Ross	ROS	9
Gallia	GAL	10			
Geauga	GEA	12	Sandusky	SAN	2
Greene	GRE	8	Scioto	SCI	9
Guernsey	GUE	5	Seneca	SEN	2
			Shelby	SHE	7
Hamilton	HAM	8	Stark	STA	4
Hancock	HAN	1	Summit	SUM	4
Hardin	HAR	1			
Harrison	HAS	11	Trumbull	TRU	4
Henry	HEN	2	Tuscarawas	TUS	11
Highland	HIG	9			
Hocking	HOC	10	Union	UNI	6
Holmes	HOL	11			
Huron	HUR	3	Van Wert	VAN	1
			Vinton	VIN	10
Jackson	JAC	9			
Jefferson	JEF	11	Warren	WAR	8
			Washington	WAS	10
Knox	KNO	5	Wayne	WAY	3
			Williams	WIL	2
Lake	LAK	12	Wood	WOO	2
Lawrence	LAW	9	Wyandot	WYA	1

ODOT Districts

DISTRICT	ADDRESS	CITY	ZIP CODE	PHONE NUMBER
District 1	1885 N. McCullough St.	Lima	45801	419-222-9055
District 2	317 E. Poe Rd.	Bowling Green	43402	419-353-8131
District 3	906 N. Clark St.	Ashland	44805	800-276-4188
District 4	2088 South Arlington Rd.	Akron	44306	800-603-1054
District 5	9600 Jacksontown Rd., S.E.	Jacksontown	43030	740-323-4400
District 6	400 E. William St.	Delaware	43015	740-833-8000
District 7	1001 St. Mary's Ave.	Sidney	45365	937-492-1141
District 8	505 S. State Route 741	Lebanon	45036	513-932-3030
District 9	650 Eastern Ave.	Chillicothe	45601	740-773-2691
District 10	338 Muskingum Dr.	Marietta	45750	740-568-3900
District 11	2201 Reiser Ave., S.E.	New Philadelphia	44663	330-339-6633
District 12	5500 Transportation Blvd.	Garfield Heights	44125	216-581-2100

GLOSSARY

Arterial - A functional classification for a facility primarily used for through traffic, usually on a continuous route.

Attenuator (Crash Cushion) - Protective devices that prevent an errant vehicle from impacting fixed objects by gradually decelerating the vehicle to a safe stop or by redirecting the vehicle away from the obstacle.

Backslope - The slope from the back of a ditch to the existing ground surface. (Sometimes referred to as a cut slope.)

Barrier - A device which provides a physical limitation through which a vehicle would not normally pass. It is intended to contain or redirect a vehicle.

Barrier Clearance - The distance required between the face of a barrier and the face of an obstacle to permit adequate shielding.

Barrier Grading - The shaping of the roadside when a barrier is required for slope protection. (See **Figure 307-4**).

Bicycle Lane or Bike Lane – A portion of roadway that has been designated by pavement markings and signs
for preferential or exclusive use by bicycles.

Border - The area between the face of curb and the right of way line. Usually referred to as the border area when no sidewalk is used.

Buffer - The space between the face of the curb and the sidewalk for the purpose of providing snow storage, a buffer between cars and pedestrians, a place for signs and to improve aesthetics.

Clear Zone - The unobstructed, traversable area provided beyond the edge of the *through traveled way* for the recovery of errant vehicles. The clear zone includes shoulders, bike lanes, and auxiliary lanes, except those auxiliary lanes that function like through lanes.

Clear Zone Grading - The shaping of the roadside using 4:1 or flatter foreslopes and traversable ditches within the clear zone area. (See **Figure 307-3**).

Cloverleaf Interchange - An interchange with loop ramps and outer ramps for directional movements. A full cloverleaf has ramps in every quadrant.

Collector - A functional classification for a facility in an intermediate functional category connecting smaller local or street systems with larger arterial systems.

Collector-Distributor (C-D) - A directional roadway adjacent to a freeway used to reduce the number of conflicts (merging, diverging and weaving) on the mainline facility.

Common Grading - The shaping of the roadside using 3:1 or flatter slopes and normal ditches. (See **Figure 307-4**).

Converging Roadway - Separate and nearly parallel roadways or ramps which combine into a single continuous roadway or ramp having a greater number of lanes beyond the nose than the number of lanes on either approach roadway.

Controlled Access - (Partial control of access) - Highway right of way where preference is given to through traffic. In addition to access connections with selected public roads, there may be some private drive connections.

Crest Vertical Curve - A vertical curve such that the point of intersection of the approach grades is above the roadway profile. Crest vertical curves are concave downward.

Critical Slope - A slope, steeper than 3:1, on which vehicles are likely to overturn.

Cross Slope - The rate of change of elevation along a straight line from one point in cross section to another.

Cut Slope - See Backslope.

Decision Sight Distance - The distance required for a driver to detect an unexpected or otherwise difficult to perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the required maneuver safely.

Degree of Curve (Arc Definition) - The angle subtended at the center by an arc of 100 foot length.

Design Exception - A document which explains the engineering and/or other reasons for allowing certain design criteria to be relaxed in extreme, unique, or unusual circumstances.

Design Hour - The 30th highest hourly volume of the design year.

Design Hourly Volume - The total volume of traffic in the design hour, usually a forecast of peak hour volume, measured in vehicles per hour.

Design Speed - A selected speed used to determine the various geometric design features of the roadway.

Diamond Interchange - The simplest and most common type of interchange, formed when one-way diagonal ramps are provided in each quadrant and left turns are provided on the minor highway.

Directional Interchange - An interchange, generally having more than one grade separation, with direct connections for all movements.

Diverging Roadway - Where a single roadway branches or forks into two separate roadways without the use of a speed change lane.

Edge of Traveled Way - The intersection of the mainline pavement with the treated or turf shoulder or the curb and gutter.

Expressway - A divided arterial highway with full or partial control of access and generally with grade separations at major intersections.

Fill Slope - See Foreslope.
Foreslope - The slope from the edge of the graded shoulder to the bottom of the ditch. (Also called Fill Slope.)

Freeway - An expressway with full access control and no at-grade intersections.

Functional Classification - The grouping of highways by the character of service they provide.

Glare Screen - A device used to shield a driver's eye from the headlights of an oncoming vehicle.

Graded Shoulder - The area located between the edge of traveled way and the foreslope.

Headlight Sight Distance - The stopping sight distance required on an unlighted sag vertical curve.

Horizontal Sight Distance - The sight distance available in consideration of various horizontal alignment features, such as: degree of curvature and the horizontal distance to roadside obstructions.

Intersection Sight Distance (ISD) - The sight distance required within the corners of intersections to safely allow a variety of vehicular maneuvers based on the type of traffic control at the intersection.

Interstate - Those roadways on the Federal System which have the highest design speeds and the most stringent design standards.

"K" Factor - The length of a vertical curve divided by the algebraic difference in grades expressed as a percent. "K" factors are only applicable where the length of curve is greater than the necessary stopping sight distance.

Lateral Clearance - The distance measured horizontally from the edge of traveled way to the face of an object (parapet, abutment, pier, wall, etc.).

Legal Speed - The legislated or agency authorized maximum speed limit of a section of roadway.

Length of Need (LON) Point – That point on the terminal or longitudinal barrier at which it will contain and redirect an impacting vehicle along the face of the terminal or barrier.

Level of Service - A qualitative measure describing the operational flow of traffic.

Limited Access (Full control of access) - Highway right-of-way where rights of access of properties abutting the highway are acquired, such that all access to and from the highway are prevented except at designated locations.

Local Road - A functional classification used for rural roadways whose primary function is to provide access to residences, businesses or other abutting properties.

Local Street - A functional classification used for urban roadways whose primary function is to provide access to residences, businesses or other abutting properties.

Non-Recoverable Slope - A slope that a vehicle can traverse, but it is generally too steep to allow the vehicle to stop or return to the roadway. Traversable nonrecoverable slopes are between 1V:4H and 1V:3H and are NOT included in the specified clear zone distance.

Normal Design Criteria - The criteria used for the design of new or reconstructed projects (all projects that do not qualify as 3R).

Normal Ditch - A trapezoidal-shaped ditch having a bottom width of 2 feet and rounding of 4 feet (See **Figure 307-4**).

Passing Sight Distance (PSD) - The visible length of highway required for a vehicle to execute a normal passing maneuver as related to design conditions and design speed.

Peak Hour - The maximum traffic volume hour of the day.

Reconstructed Bridge - Any improvement to an existing bridge involving the replacement of the bridge deck or more.

Recoverable Ditch - A rounded ditch having a radius of either 20 or 40 feet (See **Figure 307-2**).

Recoverable Slope - A slope on which a motorist may, to a greater or lesser extent, retain or regain control of a vehicle. Slopes flatter than 1V:4H are generally considered recoverable.

Resurfacing, Restoration and Rehabilitation (3R) - Improvements to existing roadways, which have as their main purpose, the restoration of the physical features (pavement, curb, guardrail, etc.) without altering the original design elements.

Resurfacing, Restoration, Rehabilitation and Reconstruction (4R) - Much like 3R, except that 4R allows for the complete reconstruction of the roadway and alteration of certain design elements (i.e., lane widths, shoulder width, SSD, etc.).

Roadside - The area between the outside shoulder edge and the right-of-way limits. The area between roadways of a divided highway may also be considered roadside.

Roadway - The portion of a highway, including shoulders, for vehicular use.

Safety Grading - The shaping of the roadside using 6:1 or flatter slopes within the clear zone area and 3:1 or flatter foreslopes and recoverable ditches extending beyond the clear zone (See **Figure 307-1**).

Sag Vertical Curve - A vertical curve such that the point of intersection of the approach grades is below the profile line. Sag vertical curves are concave upward.

Shared Lane – A lane of a traveled way that is open to both bicycle and motor vehicle travel.

Shared Use Path – Facilities physically separated from motor vehicle traffic by an open space or barrier, either
within the highway right-of-way or within an independent right-of-way. Shared use paths may be used by a mix
of non-motorized users such as bicyclists, walkers, runners, wheel chair users and skaters.

Sidepath – A shared use path located immediately adjacent and parallel to a roadway.

Shy Distance – The space adjacent to fixed objects, such as walls, fences, shrubs, buildings, parked cars and other features that pedestrians typically avoid.

Shy-line Offset - The distance from the edge of the traveled way beyond which a roadside object will not be perceived as an obstacle by the typical driver to the extent that the driver will change the vehicle's placement or speed.

Sloped Curb (mountable) - Curbs 6 inches or less in height with a sloping face designed to be traversable by vehicles when required.

Spiral - A transition curve from a tangent to a circular curve, or a circular curve to a circular curve, designed to effect a more gradual change of direction. The Euler spiral (clothoid) is used in design.

Stopping Sight Distance (SSD) - The cumulative distance traversed from the time a driver sees a hazard necessitating a stop, actually applies the brakes and comes to a stop.

Superelevation - The cross-slope of the pavement used to compensate for the effect of centrifugal force on horizontal curves.

System Interchange - An interchange that connects two or more freeways via a network of ramps and connectors.

Service Interchange - An interchange that connects a freeway with local surface streets or arterials.

Temporary Road - Any crossover, ramp, roadway, etc. whose sole purpose is to temporarily maintain traffic during construction which is normally removed upon project completion.

Through Traveleld Way – The portion of roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

Traveled Way - The portion of the roadway for the movement of vehicles, exclusive of shoulders and bicycle lanes.

Traversable Ditch (preferred ditch) - An open ditch with a preferred combination of foreslope, backslope, bottom width and rounding that allows the ditch shape to be used within the clear zone. (See **Figures 307-10 & 307-11**).

Traversable Slope - A slope from which a motorist will be unlikely to steer back to the roadway but may be able to slow and stop safely. Slopes between 1V:3H and 1V:4H generally fall into this category.

Treated Shoulder - That portion of the graded shoulder which has some type of surface treatment.

Tree Lawn - see Buffer.

Trumpet Interchange - A semi-directional “T” interchange.

3R Values - Special values developed for certain design features on 3R improvements.

Vertical Clearance - The distance, measured vertically, from the surface (pavement, shoulder, ground, etc.) to a fixed overhead object (bridge superstructure, sign, signal, etc.).

Vertical Curb (barrier) - A steep faced curb 6 inches or more in height.

DESIGN REFERENCE DOCUMENTS

ODOT Publications

Contact ODOT Office of Contracts (614) 466-3778 to purchase,
or link to them at http://www.dot.state.oh.us/drrc/.

The current revision of those listed should be used.

- Bridge Design Manual (ODOT)
- Construction and Material Specifications
- Location and Design Manual
    - Volume Two - Drainage Design
    - Volume Three - Highway Plan
- Ohio Manual of Uniform Traffic Control Devices
- Pavement Design & Rehabilitation Manual
- Railroad Project Procedure Manual
- Real Estate Policies and Procedures Manual
- State Highway Access Management Manual
- Specifications for Subsurface Investigations

- Standard Construction Drawings
    - o Roadway Engineering Services
    - o Structural Engineering
    - o Traffic Engineering

- Traffic Engineering Manual (and appendices)
    - o Design Manual for Highway Lighting
    - o Design Manual for Directional Guide Signs
    - o Standard Sign Design Manual
    - o Traffic Control Design Information Manual

AASHTO Publications

Phone: (202) 624-5800, Web site: http://www.transportion.org

- Guide for Erecting Mailboxes on Highways (1994)
- Guide for the Development of Bicycle Facilities (2012)
- Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT 400) (2001)
- Policy on Design Standards - Interstate System (2005)
- Policy on Geometric Design of Highways and Streets (2011)
- Roadside Design Guide (2011)
- Highway Safety Manual (2010)

TRB Publications

Phone: (202)334-3213, Web site: http://www.nas.edu/trb/

- Designing Safer Roads - Practices for Resurfacing, Restoration and Rehabilitation (TRB Special Report 214 - 1987)
- Highway Capacity Manual (TRB - 2010)
- Recommended Procedures for the Safety Performance Evaluation of Highway Features (NCHRP

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R113a	Item 606 - Anchor Assembly, Type E
R116	Paving Under Guardrail
R118	Item Special - Mailbox Support
R123	Item 606 - Impact Attenuator, Type 1 (Unidirectional or Bidirectional)
R124	Item 606 - Impact Attenuator, Type 2 [(Model #), (Unidirectional or Bidirectional)]
R125	Item 606 - Impact Attenuator, Type 3
R126	Item 606 - Impact Attenuator, Type 2 LS [(Model #), (Unidirectional or Bidirectional)]

Appendix C

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References

Procedures for Developing Design Designations for Non-Interstate Bridge Replacement/Rehabilitation Projects

Guidelines for Identifying Acceptable Locations for the Disposal of Waste Material and Construction Debris or The Excavation of Borrow Material Within ODOT Right-of-Way

Landscaping Guidelines

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100 INTRODUCTION

In order to determine the criteria to be used for a project, it is necessary to initially identify some basic information about the facility. This information is known collectively as the design designation and includes: functional classification, traffic data, terrain, locale, design speed and legal speed. ***Figure 100-1*** shows how these design controls relate to many of the design features included in this manual.

101 FUNCTIONAL CLASSIFICATION

101.1 General

Functional classification, the systematic grouping of highways by the character of service they provide, is an important tool that has been used for many years in comprehensive transportation planning. Its adoption by highway designers to categorize basic highway systems serves as an effective transition from the planning process to the design process. Under a functional classification system, standards and level of service vary according to the function of the highway facility. Traffic volumes are used to refine the standards for each class.

101.2 Urban & Rural

Functional classification is initially divided into urban and rural categories. Urban areas are comprised of: (1) places with a population of 5,000 or more, that are incorporated as cities, villages, and towns but excluding the rural portions of extended cities; (2) census designated places with 5,000 or more persons; and (3) other territory, incorporated or unincorporated, included in urbanized areas.

Extended cities are those cities whose boundaries include territory that is essentially rural in character (e.g., uncurbed pavement with open drainage, where a rural typical section would be more consistent with the existing roadway).

Urbanized areas consist of one or more places (central places) and the adjacent densely populated surrounding territory (urban fringe) that together have a minimum population of 50,000. The urban fringe generally consists of contiguous territory having a density of at least 1,000 persons per square mile.

Rural areas are those outside the boundaries of urban areas.

101.3 Classification Used in ODOT Design Criteria

The rural and urban functional classifications are further defined for design purposes as follows:

- Interstate
- Other Freeways and Expressways
- Principal Arterial Roads (rural) and Streets (urban)
- Minor Arterial Roads (rural) and Streets (urban)
- Collector Roads (rural) and Street (urban)
- Local Roads (rural) and Streets (urban)

The functional classifications for streets and highways in Ohio are kept on record in the Office of Systems Planning and Program Management.

102 TRAFFIC DATA

102.1 General

Traffic data is the foundation upon which designs are based; consequently, it is important that adequate traffic data be available early in the development of a project's design. It is equally important that this data be coordinated within various geographic regions of the State to avoid inconsistencies between projects under the same traffic influences.

All forecasted traffic data used shall be developed following state traffic forecasting guidelines provided by Division of Planning, Office of Statewide Planning & Research, Modeling & Forecasting section. Documents containing forecasting guidelines are available on the office internet web page.

102.2 Traffic Data Content

The design criteria tables in this manual require basic traffic data for the design year. The traffic design year is generally considered to be the following:

Project Type	Traffic Design Year (After Opening Day)
New Construction	20 years hence
Reconstruction	20 years hence
Major Pavement Rehab.	20 years hence
Minor Pavement Rehab.	12 years hence
Two-Lane Resurfacing	12 years hence

For most projects, the following data are required:

- Average Daily Traffic (ADT) for opening day (for lighting and signal warrants).
- Average Daily Traffic (ADT) for design year.
- Design Hourly Volume (DHV). AM and PM DHV are required for interchange design.
- The percentage of B and C trucks (T24) during the 24-hour period for the design year.
- The percentage of B and C trucks (TD) during the design hour traffic for the design year (for adjusting capacity analyses).

- Directional Distribution Factor (D) for the design year (used to obtain the Directional Design Hour Volume (DDHV) for the design hour).

Projects on low-volume facilities (current ADT<400) without a design year traffic forecast may use the current ADT for design purposes.

Average Daily Traffic (ADT) volumes should be subdivided into the following classes:

P - Passenger Cars - including station wagons, mini-vans, sport utility vehicles and motorcycles.

A - Commercial - including motorized recreational vehicles, school buses, and light delivery trucks such as panel trucks and pick-up trucks which do not use dual tires.

B - Commercial - including tractors, trucks with semi-trailers and truck-trailer combinations.

C - Commercial - including buses or dual tired trucks having either single or tandem rear axles.

Estimated Design Year ADT may be subdivided into P & A vehicles and B & C trucks if data for each vehicle class is not readily available, since these classes have similar operational characteristics. Current ADTs for various sections of Interstate, United States and State Highways for each county are available in the Traffic Survey Report published by the Office of Technical Services. Counts at specific points in the section may vary from the average and are available upon request from the Office of Technical Services.

103 TERRAIN & LOCALE

103.1 General

Many rural design features are significantly influenced by the topography of the land through which the roadway is constructed. To characterize variations, Ohio topography is categorized into three types of terrain: level, rolling or hilly. Locale is used to describe the type of area and generally refers to the character and extent of development in the vicinity. Urban, rural, residential, and commercial/industrial are characteristics often used to describe locale.

103.2 Terrain Types

Level - Any combination of grades and horizontal and vertical alignment permitting heavy vehicles to maintain approximately the same speed as passenger cars. This generally includes grades of no more than 2 percent for a distance of no more than 2 miles.

Rolling - Any combination of grades and horizontal and vertical alignment causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but not causing heavy vehicles to operate at crawl speeds.

Hilly - Any combination of grades and horizontal and vertical alignment causing heavy vehicles to operate at crawl speeds. Hilly terrain in Ohio conforms to mountainous terrain used in the American Association of State Highway Transportation Officials (AASHTO) publications.

100 Design Controls and Exceptions

For design purposes a heavy vehicle is defined as a vehicle with a mass/power ratio of approximately 200 lb/hp. This represents a typical semi-truck. Crawl speed is the maximum sustained speed that a heavy vehicle can maintain on an extended upgrade and varies with the weight of the vehicle and the steepness of the grade.

104 DESIGN & LEGAL SPEED

104.1 General

Design speed is defined in the AASHTO publication, "A Policy on Geometric Design of Highways and Streets" (Green Book), as a selected speed used to determine the various geometric design features of the roadway. The assumed design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use and the functional classification of highway.

104.2 Design Speed Values

The minimum design speed for all projects shall be equal to or greater than the legal speed for the facility and the preferred design speed shall be 5 mph higher than the legal speed. Design speeds shall be specified in 5 mph increments. For resurfacing projects the design speed is the legal speed, or alternately, the 85th percentile speed for individual or series of horizontal and vertical curves. Refer to part 1200 of the Traffic Engineering Manual for guidance establishing the 85th percentile speed. Ramp design speeds are included in ***Section 503.2***.

Design speeds of 50 mph and higher are considered high speed and design speeds less than 50 mph are considered low speed.

105 DESIGN EXCEPTIONS

105.1 General

Designers and engineers are faced with many complex tradeoffs when designing highways and streets. A good design balances cost, safety, mobility, social and environmental impacts, and the needs of a wide variety of roadway users.

Highway design criteria that have been established through years of practice and research form the basis by which roadway designers achieve this balance. These criteria are expressed as minimum dimensional values or ranges of values for various elements of the three-dimensional design features of the highway. The criteria are intended to deliver an acceptable, generally cost-effective level of performance (traffic operations, safety, maintainability, and constructability). The criteria are updated and refined as research and experience increase knowledge in the field of highway engineering, traffic operations, and safety.

A design exception is a documented decision to design a highway element or a segment of highway to design criteria that do not meet minimum values or ranges established for that highway or project. The minimum values or ranges of design criteria, also known as controlling criteria for design, that require design exceptions when they are not met or exceeded are set forth in ***Section 105.2*** and ***Figure 105-1***.

100 Design Controls and Exceptions

The designer should call attention to any design features that require a design exception as soon as possible, but no later than the first stage review submittal as defined in the Location & Design Manual, Volume Three.

Other design values, policies, practices, etc. that are mentioned in this Manual are guidelines intended to promote uniformity and good design. Deviation from these guidelines does not require a formal design exception; however, it may still be necessary to justify or otherwise seek approval from ODOT of the proposed design when deviations are necessary. This should be accomplished through the normal review process.

Ramps do not have continuous design speeds throughout their lengths. However, design exceptions are required for not meeting the lower range for speed related items (see ***Section 503.2*** for directional and loop ramps). In addition, design exceptions for non-speed related items (e.g., lane width, shoulder width, bridge width, and lateral clearance) are required.

A design exception should be reevaluated if the project has not sold within five years of the approved design exception.

Exceptions will not be required for projects that do not change the basic highway cross-section or geometry; e.g. resurfacing, bridge deck overlays, rest areas, lighting, signing, signalization, fencing, guardrail, slide corrections, etc. A change in the basic highway cross section would include any change to the lane width, shoulder width, pavement cross slope or additional earthwork beyond the graded shoulder.

Side roads with more than 600’ of approach work do require design exceptions.

105.2 Design Controlling Criteria that Require a Design Exception

Exceptions must be processed for the following design controlling criteria when they will not be attained:

High Speed Roadways (Interstate highways, other freeways and roadways with a design speed ≥50 mph)

1. Lane Width
2. Shoulder Width
3. Horizontal Curve Radius
4. Maximum Grade
5. Stopping Sight Distance (Horizontal and Crest Vertical Curves)
6. Superelevation Rate
7. Vertical Clearance
8. Pavement Cross Slope
9. Design Loading Structural Capacity

Low Speed Roadways (design speed <50 mph)

1. Design Loading Structural Capacity
2. Lane Width (only if required for the National Network, see ***Section 105.3*** below)

In addition to the above geometric design features, design exceptions are also required when existing non-standard bridge parapets and curb configurations are to be retained. For details on non-standard bridge parapets see ODOT Bridge Design Manual or contact the Office of Structural Engineering.

105.3 National Network (or National Truck Network)

The National Network was established by Congress in 1982 with the Surface Transportation Assistance Act as the network of highways designated for use by large trucks. In Ohio, the National Network consists of all Interstate Routes and the old Federal Aid Primary (FAP) Routes. One of the criteria for the National Network is, “The route consists of lanes designed to be a width of 12 feet or more or is otherwise consistent with highway safety.”

In lieu of providing 12’ lane widths for all lanes on the National Network, a single 12’ lane in each direction can be provided with a design exception for any lane widths that do not meet design criteria. When no 12’ lane in each direction can be provided a design exception will be required. The lane width design exception would be required regardless of whether the facility is low or high speed. The emphasis of the lane width design exception should focus on the impacts of the truck traffic such as truck involved crashes (existing and expected), truck off-tracking, etc.

105.4 Local Projects

Design exceptions for Local-let projects should follow the guidelines in the Local Programs manual on Project Development and Design. Design exceptions for Local ODOT-let projects should follow the L&D Manual. The design exception format for both should follow ***Section 105.5.1***. All Local project (both Local-let and ODOT-let) design exceptions are approved by the District Capital Programs Administrator.

105.5 Design Exception Documentation and Approval Process

105.5.1 Documentation Format

The Design exception document must contain at least the following information:

1. The Design Designation for the project.
2. A Title Sheet Location map and a schematic or plan sheet if needed for clarity.
3. The controlling criteria affected by the proposed design exceptions. (As noted in ***Figure 105-1***, normal design criteria must be used as the basis for all design exceptions.)
4. A description of the project.
5. Proposed mitigation for the deviation (if any).
6. Support for the proposed deviation based upon sound engineering practices, cost comparison/ analysis, impact on the environment, the relationship between any crash patterns and the proposed design exception, etc.
7. The GCAT/CAM Tool must be attached. HSM Analysis may also be required by ORE or CPA based upon the nature of the exception request. Refer to the Safety Analysis Guidelines maintained by the Office of Program Management for information to conduct the analysis.

105.5.2 Processing and Approval Authority

1. All design exception requests shall be prepared or processed by the District using the electronic submission process found on the Office of Roadway Engineering website, http://www.dot.state.oh.us/Divisions/Engineering/Roadway/Pages/default.aspx. Design exceptions must be prepared and sealed by a licensed professional engineer.

2. Design Exceptions for projects in the LPA process will be approved by the District Capital Programs Administrator.

3. Design exceptions for access permit projects are required to be approved by the District Capital Program Administrator.

4. All non-local project Design Exceptions will be approved by Administrator of the Office of Roadway Engineering. The Office of Roadway Engineering will coordinate with FHWA for all projects requiring FHWA approval.

5. The Office of Roadway Engineering will be advised in writing of the action taken by the FHWA on Federal oversight projects. The original of such correspondence will be retained by the Office of Roadway Engineering and copies will be forwarded to the District. The District shall advise all involved LPAs and the Office of Estimating.

6. All exceptions to the 16' vertical clearance standard on rural interstate routes or on a single interstate route through urban areas must be coordinated with the Surface Deployment and Distribution Command Transportation Agency (SDDCTEA) by the District. For details refer to the FHWA Memorandum of April 15, 2009 (https://www.fhwa.dot.gov/design/090415.cfm).

105.5.3 Amendments to Design Exceptions

A previously approved design exception may be amended to accommodate additional elements (that do not invalidate previously approved items) by submitting an addendum to the design exception. The original may be amended to change previously approved items or remove items that no longer require an exception by submitting a revision to the design exception. In either case, the procedure follows the same formatting and approval process as the original design exception.

106 DATA-DRIVEN SAFETY ANALYSIS

106.1 General

The purpose of the Data-Driven Safety Analysis (DDSA) is to better understand the safety performance of a project and each of the alternatives. Additionally, it can be used to determine if there is a pattern or concentration of crashes within the project limits that can be reasonably and practically addressed through the inclusion of countermeasures in the project.

Factors that can affect countermeasures being "reasonable and practical" include but are not limited to:

1. Cost;
2. Environmental or R/W impacts;
3. Countermeasure work type being compatible with the planned project;
4. Schedule impacts

A minimum safety assessment should be performed in the early phases of project development (i.e. project programming). This will allow schedule, scope, and budget considerations to be accounted for when reasonable and practical countermeasures are to be included in the project. Reference Safety Analysis Guidelines maintained by the Office of Program Management for items included in the minimum safety assessment.

106.2 Applicable Projects

(DDSA) is applicable to ODOT Let projects, except those whose primary purpose are noted below. While Local Let projects are exempt from performing DDSA, the analysis is strongly recommended to understand the impacts of the project on crash frequency and severity.

Typical projects where the DDSA is not applicable:

a) Maintenance projects such as guardrail repair, mowing, striping, signing, RPM's, etc.;
b) Pavement surface treatment projects as defined by Section 550 of the Pavement Manual;
c) Spot repairs;
d) Slot paving;

106.3 Data-Driven Safety Analysis

(DDSA) is defined as using real data and established methods to analyze crash and roadway data to estimate the safety impacts of highway projects, assess existing safety conditions, and prioritize locations for safety analysis and/or funding. This allows agencies to target investments with greater confidence that will improve safety on the roadway.

Each project is categorized depending on the project size, complexity, and/or potential impact to the environment. Based on the complexity of the project, one of three safety assessment processes should be followed as part of the project development process to qualitatively assess safety. The analysis process is outlined in the Safety Analysis Guidelines maintained by the Office of Program Management.

A minimum assessment for all projects involves reviewing any applicable studies for the project area, reviewing the ODOT Safety integrated Project (SIP) Maps, documenting any other safety priorities in the area (state or local), and reviewing historical crash trends.

Where in the opinion of the district there is a noteworthy location or pattern of crashes, a determination should be made if there is a reasonable and practical countermeasure(s) that can be incorporated into the project and if a safety funding request will be made. For high priority locations, there may be situations when there are reasonable and practical countermeasures, but they can't be incorporated into the project due to factors such as schedule or work type incompatibility. In these cases, consideration should be given to creating a standalone safety project to address the high priority location.

Projects that have an identified location on the SIP maps or statewide/regional safety priority are eligible for supplemental safety funding up to $500,000 through an abbreviated safety funding application process. Requests exceeding this amount should be submitted through the biannual HSIP Safety Funding Application process.

Refer to the Safety Analysis Guidelines maintained by the Office of Program Management for detailed analysis requirements. The Office of Program Management also maintains the SIP Maps, the Statewide HSIP Priority Location Lists and data related to historical crash trends that can be used to conduct a minimum project assessment. Abbreviated Safety Applications should be coordinated through the District Safety Review Team (DSRT) coordinator.

106.4 Data-Driven Safety Analysis Documentation

While safety should be considered and evaluated for every project, there is no requirement to include safety countermeasures for projects without safety included in the purpose and need. Rather, projects should be evaluated to determine if there is a reasonable and practical countermeasure(s) that can be incorporated into the project without expanding project scope. Decisions should be documented on the appropriate “Data-Driven Safety Analysis Documentation” form. Refer to the Safety Analysis Guidelines maintained by the Office of Program Management for documentation templates.

http://www.dot.state.oh.us/Divisions/Planning/ProgramManagement/HighwaySafety/HSIP/SafetyAnalysisGuidelines/Safety_Analysis_Guidelines.pdf

LIST OF FIGURES

SAMPLES

DESIGN CONTROL/DESIGN FEATURE RELATIONSHIP	100-1
	REFERENCE SECTION 100.1

DESIGN FEATURES	DESIGN CONTROLS				
	Functional Classification	Traffic Data	Terrain	Local	Design Speed
Lane Width (Rural)	X	X	X		X
Lane Width (Urban)	X			X	
Shoulder Width &Type (Rural)	X	X			
Shoulder Width & Type (Urban)	X			X	
Guardrail Offset	X	X			
Degree of Curvature				X	X
Grades	X		X	X	X
Bridge Clearances (Horizontal & Vertical)	X	X			
Stopping Sight Distance					X
Passing & Intersection Sight Distances					X
Decision Sight Distance					X
Superelevation				X	X
Curve Widening					X
Design Speed (Rural)	X	X	X		
Design Speed (Urban)	X			X	
Vertical Alignment	X		X	X	X
Horizontal Alignment				X	X

# APPROPRIATE DESIGN CRITERIA GUIDE	105-1
	REFERENCE SECTION 105.1 & 105.5.1

Key Highway Design Features Requiring Design Exceptions	Normal Design Criteria (1)	
	Section	Figure
Lane Width	301.1.2 & 303.1	301-2 & -4, 303-1
Shoulder Width	301.2.3 & 303.1	301-3 & -4, 303-1
Design Loading Structural Capacity	302.1	See Bridge Design Manual
Horizontal Curve Radius	202.3	202-2
Maximum Grade	203.2	203-1
Stopping Sight Distance (Horizontal & Crest Vertical Curve)	201.2	201-1, 203-3, -4
Pavement Cross Slope	301.1.5	301-6
Superelevation Rate	202.4.1, & .4.3	202-3, -7 thru -10
Vertical Clearance	302.1	302-1, -2, -3

1) Normal design criteria must be used as the basis for all design exceptions.

Figure 107-1 Non-Complex Projects (No Alternative Analysis)

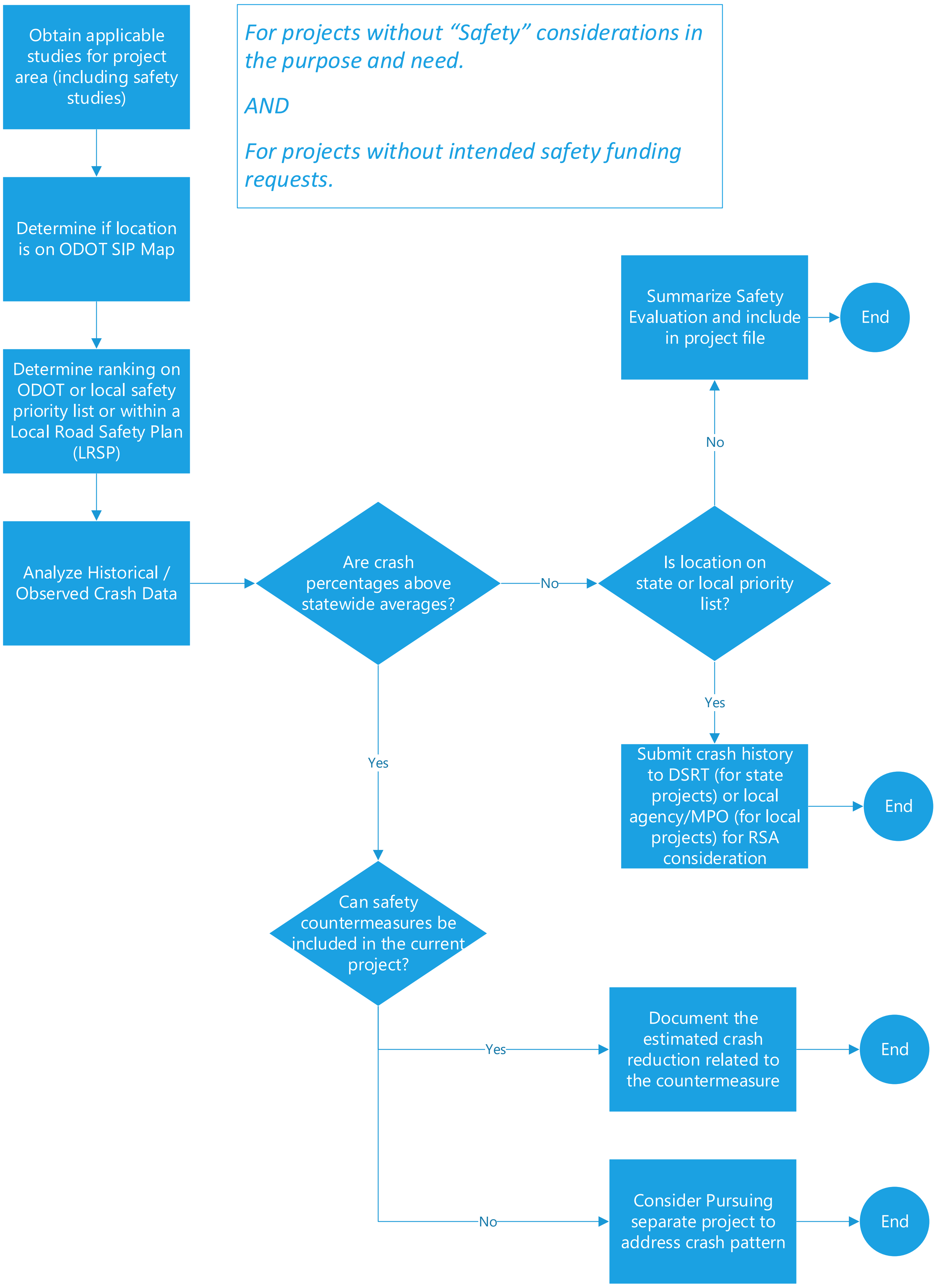
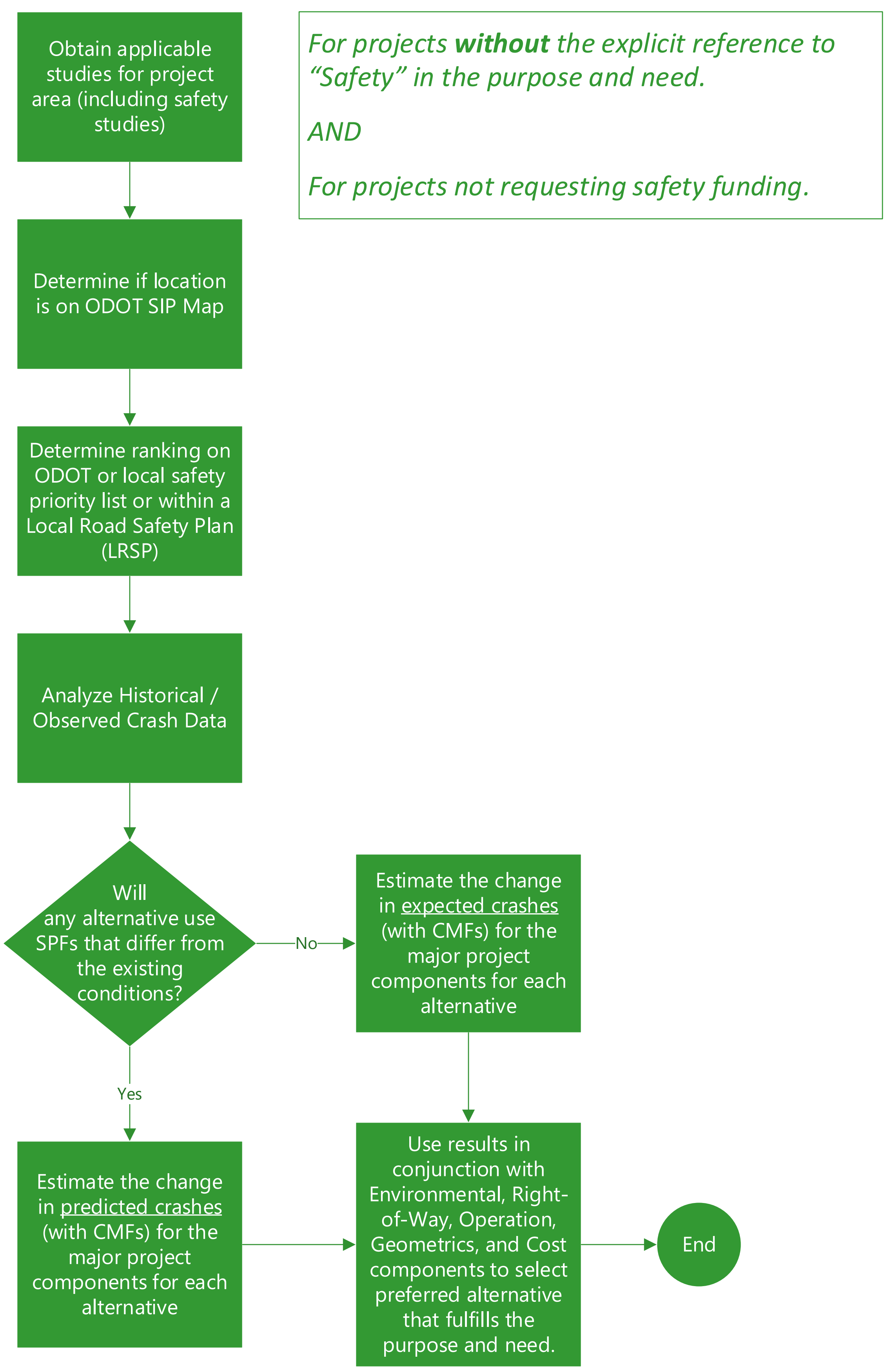


Figure 107-2 Complex Projects Assessment with Alternatives Analysis without "Safety" in the Purpose and Need Statement



January 2019

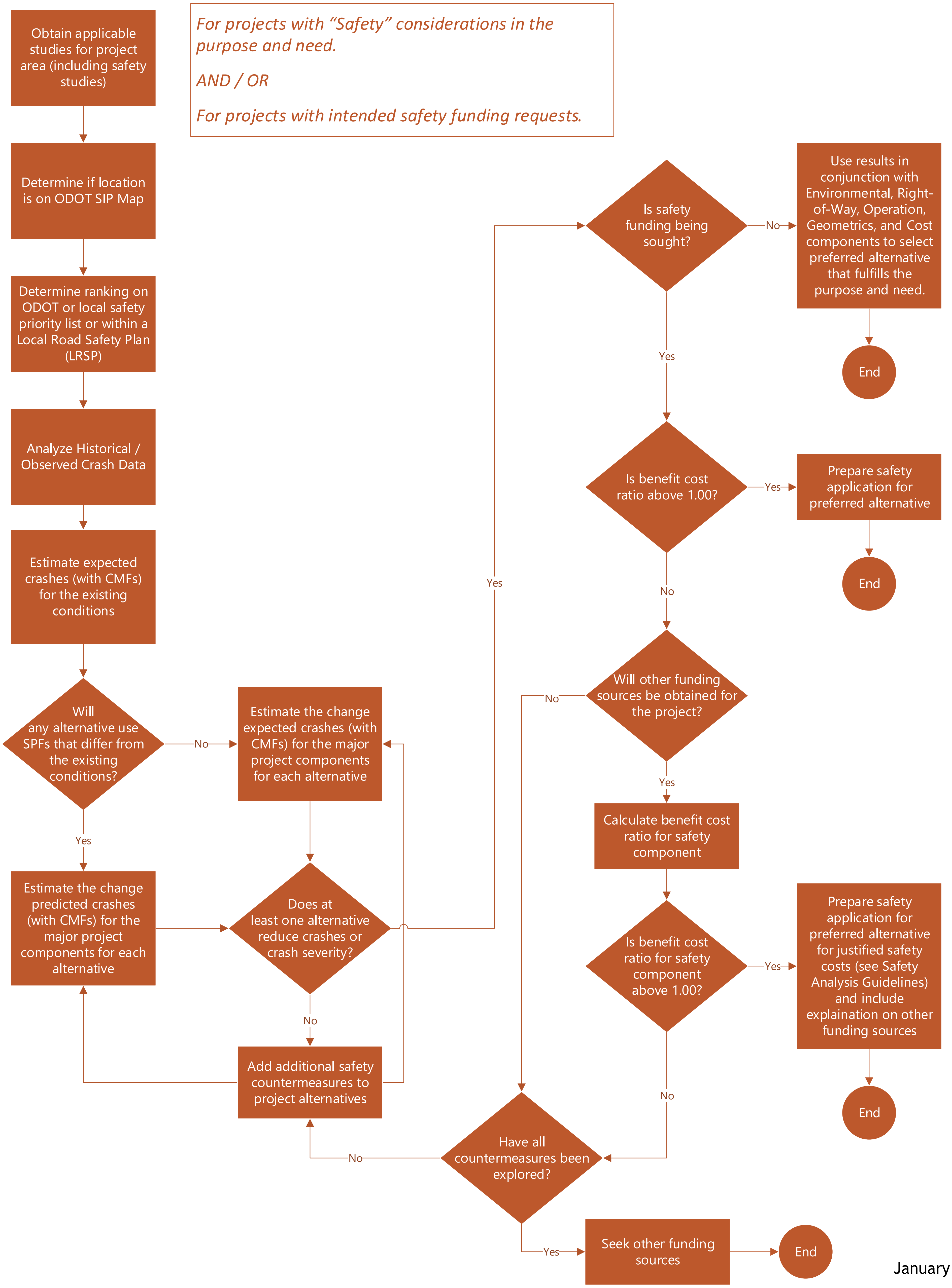
Definitions:

Crash modification factor (CMF): value which quantifies the change in crash frequency at a site as a result of implementing a specific countermeasure or treatment. It can be single value or function and may apply to all crashes or specific crash type(s).

Expected average crash frequency: the estimate of long-term expected average crash frequency of a site, facility, or network under a given set of geometric conditions and traffic volumes (AADT) in a given period of years.

Predicted average crash frequency: the estimate of long-term average crash frequency – based on the average number of crashes of a peer group (exact same base conditions) with a given AADT

Figure 107-3 Complex Projects Assessment with Alternative Analysis and Safety Component



January 2019

Definitions:

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<u>Predicted average crash frequency</u>: the estimate of long-term average crash frequency – based on the average number of crashes of a peer group (exact same base conditions) with a given AADT

Design Exception Request

SCI-140-7.28

PID: 93794

Letting Type: ODOT-Let

Design Designation				
Current ADT (2016)	4,310		Td	0
Design Year ADT (2035)	4,840		Design Speed	55
Design Hourly Volume (2016)	460		Legal Speed	55
Directional Distribution	0.57		Design Functional Class	5
Trucks (24hr B&C)	0.05		Functional Class Area Type	Urban
			NHS Project	No

Submitted by:

____

(Engineer of Record)

Engineer of Record Seal

Approved by:

____

Design Exception Request

SCI-140-7.28

PID: 93794

	Controlling Criteria Identification		
Controlling Criteria	Standard	Existing (a.)	Proposed
Lane Width			
Shoulder Width			
Horizontal Curve Radius	955’	Curve 1-1300’; Curve 2 – 495’	Curve 1 – 1300’; Curve 2 – 495’
Maximum Grade			
SSD (horizontal & Crest Vertical)	495’	Curve 1 – 367’; Curve 2 - 221’	Curve 1 – 381’; Curve 2 - 345’
Superelevation Rate	Curve 1 – 0.074; Curve 2 – 0.080	Curve 1 – 0.042; Curve 2 – 0.077	Curve 1 – 0.050; Curve 2 – 0.079
Pavement Cross Slope			
Vertical Clearance			
Design Loading Structural Capacity			
“Existing” may be N/A (i.e. New alignment or new ramp)			

Project Description

The proposed project will improve safety by widening the lane and shoulder width and improving the stopping sight distance by increasing the guardrail offset through the reverse horizontal curves.

Proposed Mitigation

Oversized curve warning signs, chevrons and arrow signs were installed in 2011 and will be reinstalled with this project. Flashing beacons will be installed on the two curve signs and the two arrow signs for Curve 2.

Support for Deviation (Benefit-cost, R/W, Environmental, Constructability, Coordination with Other Projects, Relationship between any crash patterns and proposed design exception, etc.):

The lane width is being increased from 11.1’ to 12’ and the paved shoulders are being increased from 2.8’ to 4’. To correct the horizontal alignment, the roadway would need to be relocated through a sizable hill with cuts as deep as 31’.

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200 Horizontal and Vertical Design

Table of Contents

200 Horizontal and Vertical Design

200.1 Introduction

This section provides a brief discussion together with several figures of design criteria needed to properly design horizontal and vertical alignments. More detailed information can be found in the 2004 edition of **A Policy on Geometric Design of Highways and Streets** (AASHTO Green Book) and the 2001 edition of **Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT < 400).**

201 SIGHT DISTANCE

201.1 General

A primary feature in highway design is the arrangement of the geometric elements so that sufficient sight distance is provided for safe and efficient operation. The most important sight distance considerations are: distance required for stopping, distance required for operation at intersections, distance required for passing vehicles and distance needed for making decisions at complex locations.

Stopping Sight Distance (SSD) is the cumulative distance traversed by a vehicle from the instant a motorist sights an unexpected object in the roadway, applies the brakes, and is able to bring the vehicle to a stop.

Intersection Sight Distance (ISD) is the distance a motorist should be able to see other traffic operating on the intersecting roadway in order to enter or cross the roadway safely and to avoid or stop short of any unexpected conflicts in the intersection area.

Passing Sight Distance (PSD) is the distance a motorist should be able to observe oncoming traffic on a two-lane, two-way road in order to pass a vehicle safely.

Decision Sight Distance (DSD) is the distance needed for a motorist to detect, recognize, select, initiate and complete an appropriate course of action for an unexpected or otherwise difficult-to-perceive condition in the roadway.

When evaluating sight distance, the two most critical features to be considered are the height of eye and the height of the object. The driver's height of eye remains constant at 3.5 ft. for each of the sight distance categories. The height of the object, on the other hand, varies between 2 ft. and 3.5 ft. The 2 ft. object height, used for the decision and stopping sight distances, represents the taillight of the typical passenger vehicle. Research has shown that object heights below 2 ft. would result in longer crest vertical curves without providing documented safety benefits. The 3.5 ft. height, used for the intersection and passing sight distances, represents the portion of the vehicle that needs to be visible for another driver to recognize that vehicle.

201.2 Stopping Sight Distance

Stopping sight distance is the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied; and, (2) the distance needed to stop the vehicle from the instant brake application begins. These two are referred to as brake reaction distance and braking distance, respectively. The recommended brake reaction time to compute the brake reaction distance is 2.5 seconds. The recommended deceleration rate to compute the braking distance is 11.2 feet per second squared.

Figure 201-1 lists the recommended sight distance values for the given design speeds along with the corresponding equation.

201.2.1 Horizontal Sight Distance

The sight distance on horizontal curves may be restricted by obstructions on the inside of a curve, such as bridge piers, buildings, median barriers, guardrail, cut slopes, etc. ***Figure 201-2*** shows the relation of sight distance, horizontal curvature, line of sight, and obstruction offset. In using this figure, the designer should enter the required stopping sight distance from ***Figure 201-1*** and the degree of curvature or radius [curve radius]. Where these two lines intersect, the offset of the obstruction needed to satisfy the sight distance requirements may be read from the curved lines.

Where the horizontal sight distance is restricted by a cut slope in the inside of the curve, the offset shall be measured to a point on the cut slope that is at the same elevation as the roadway. This would allow a line of sight which is 3.5 ft. above the roadway to pass over a cut slope with 2.75 ft. of vegetative growth and view a 2.0 ft. high object on the far side.

When a combination of spirals, tangents and/or curves is present, the horizontal sight distance should be determined graphically.

201.2.2 Vertical Stopping Sight Distance

The sight distance on crest vertical curves is based on a driver's ability to see a 2.0 ft. high object in the roadway without being blocked out by the pavement surface. The height of eye for the driver used in the calculation is 3.5 ft. See ***Figures 203-4 & 203-7***.

The sight distance on sag curves is dependent on the driver's ability to see the pavement surface as illuminated by headlights at night. The height of headlight is assumed to be 2.0 ft., the height of object 0" and the upward divergence angle of the headlight beam is assumed to be 1°00'. See ***Figure 203-6** & **203-7***.

201.3 Intersection Sight Distance (ISD)

Intersections generally have a higher potential for vehicular conflict than a continuous section of roadway due to the occurrence of numerous traffic movements. Providing adequate sight distance at the intersection can greatly reduce the likelihood of these conflicts.

The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection and sufficient lengths along the intersecting highway to permit the driver to anticipate and avoid potential collisions. When entering or crossing a highway, motorists should be able to observe the traffic at a distance that will allow them to safely make the desired movement.

The methods for determining sight distance needed by drivers approaching an intersection are based on the same principles as stopping sight distance, but incorporate modified assumptions based on observed driver behavior at intersections.

To enhance traffic operations, intersection sight distance should be provided at all intersections. If intersections sight distance cannot be provided due to environmental or right-of-way constraints, then as a minimum, the stopping sight distance for vehicles on the major road should be provided. By providing only stopping sight distance, this will require the major-road vehicle to stop or slow down to accommodate the maneuver of the minor-road vehicle. If the intersection sight distance cannot be attained, additional safety measures should be provided. These may include, but are not limited to, advance warning signs and flashers and/or reduced speed limit zones in the vicinity of the intersection.

201.3.1 Sight Triangles

Specified areas along intersection approach legs and across their included corners should be clear of obstructions that might block a driver's view of potentially conflicting vehicles. These unobstructed areas are known as sight triangles (see ***Figure 201-4***). The waiting vehicle is assumed to be located at a minimum of 14.4 ft. and preferably 17.8 ft. from the through road edge of traveled way. The position of the waiting vehicle is the vertex of the sight triangle on the minor road, otherwise referred to as the decision point. It represents the typical position of the minor-road driver's eye when a vehicle is stopped relatively close to the major road. The left edge of the moving vehicle on the through road is assumed to be a ½ lane width for vehicles approaching from the left, or 1½ lane widths for vehicles approaching from the right. The design speed of the through road is used to select the appropriate ISD length (see ***Figure 201-5***). The dimension "b" in ***Figure 201-4*** is the ISD length.

The provision of sight triangles allows the driver on the major road to see any vehicles stopped on the minor road approach and to be prepared to slow or stop, if necessary.

201.3.1.1 Identification of Sight Obstructions with Sight Triangles

The profiles of the intersecting roadways should be designed to provide the recommended sight distances for drivers on the intersection approaches. Within a sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered, if practical. Particular attention should be given to the evaluation of sight triangles at interchange ramps or crossroad intersections where features such as bridge railings, piers, and abutments are potential sight obstructions.

The determination of whether an object constitutes a sight obstruction should consider both the horizontal and the vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 3.5 ft. above the roadway surface and the object to be seen is 3.5 ft. above the surface of the roadway. When the object height and the driver's eye are equivalent, the intersection sight distances become reciprocal (i.e., if one driver can see another vehicle, then the driver of that vehicle can also see the first vehicle).

201.3.2 Intersection Control

The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection, because different types of control impose different legal constraints on drivers and, therefore, result in different driver behavior.

At signalized intersections and all-way stop control, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left turning vehicles should have sufficient sight distance to select gaps in oncoming traffic and complete left turns. Generally, sight distances are not needed for signalized intersections.

The most critical intersection control is the stop control on the minor roadway. Sight triangles for intersections with stop control on the minor road should be considered for three situations:

1. Left turns from the minor road
2. Right turns from the minor road
3. Crossing the major road from the minor road approach.

201.3.2.1 Left Turn from the Minor Road

The intersection sight distance along the major road is determined by the following formula:

English Units: ISD = 1.47x Vmajor x tg

ISD = intersection sight distance (length of the leg of sight triangle along the major road) (ft)

Vmajor = design speed of major road (mph)

tg = time gap for minor road vehicle to enter the major road (sec.)

The design values for intersection sight distance for passenger cars are shown in ***Figure 201-5***.

The values for tg can vary (see ***Figure 201-5***) due to deviations of the intersection approach grade, truck usage, and the numbers of lanes of the facility. The values provide sufficient time for the minor-road vehicle to accelerate from a stop and complete a left turn without unduly interfering with major-road traffic operations. Where substantial volumes of heavy vehicles enter the major road (such as a ramp terminal), the tg value for the single-unit or combination truck values should be considered.

Sight distances for left turns at divided highway intersections have special considerations. If the design vehicle can be stored in the median with adequate clearance to the through lanes, a sight triangle to the right for left turns should be provided for that design vehicle turning left from the median roadway. Where the median is not wide enough to store the design vehicle, a sight triangle should be provided for that design vehicle to turn left from the minor-road approach.

Also, the median width should be considered in determining the number of lanes to be crossed. The median width should be converted to equivalent lanes.

201.3.2.2 Right Turn from the Minor Road

The intersection sight distance for right turns is determined using the same methodology as that used for left turns, except that the time gaps differ. The time gap for right turns is decreased by 1.0 second. Also, the sight triangle for traffic approaching from the left should be used for right turns onto a major road. The design values for intersection sight distance for passenger cars are shown in ***Figure 201-5***.

201.3.2.3 Crossing Maneuver from the Minor Road

In most cases, the sight distance provided by the sight triangles (for right or left turns) are adequate for a minor road vehicle to cross a major roadway. However, if the following situations exist, the sight distance for a crossing maneuver should, in of itself, be checked:

1. Where left and or right turns are not permitted from a particular approach and the crossing maneuver is the only legal maneuver

2. Where the crossing vehicle would cross the equivalent of more than six lanes

3. Where substantial volumes of heavy vehicles cross the highway and steep grades that might slow the vehicle while its back portion is still in the intersection are present on the departure roadway on the far side of the intersection

The formula for the sight distance at a crossing maneuver is the same as that for right turns. The time gap adjustments listed in ***Figure 201-5*** must be used to modify the formula for a crossing maneuver.

201.3.3 Vertical ISD

Also shown on ***Figure 201-5*** are "K" curvature rates for crest vertical curves based on ISD. The K rates are derived using the height of eye as 3.50 ft. and height of object as 3.50 ft. Appropriate equations are shown on ***Figure 201-5***.

If a road or drive intersection occurs on or near a crest vertical curve, the length of curve should be at least as long as that calculated from the K rate for ISD or the K rate for stopping sight distance, whichever is greater.

In some areas, the sight distance will be limited due to projections above the pavement surface, such as raised medians, curb and sidewalks. An illustration of this type of obstruction is shown in ***Figure 201-4***, Diagram B, where the left sight distance is limited by a portion of the bridge abutment. Locations such as this should be checked graphically and corrected by lengthening the vertical curve, eliminating the obstruction or moving the intersection.

201.4 Passing Sight Distance

Figure 201-3 lists the distance required for passing an overtaken vehicle at various design speeds. These distances are applicable to two-lane roads only. It is important to provide adequate passing sight distance for as much of the project length as possible to compensate for missed opportunities due to oncoming traffic in the passing zone.

Figure 201-3 also contains "K" curvature rates for crest vertical curves based on passing sight distance. The K rates are derived using a 3.50 ft. height of eye and a 3.50 ft. height of object. Appropriate equations are included on ***Figure 201-3***.

201.4.1 Available Passing Sight Distance

On 2-lane highways with design hourly volume (DHV) exceeding 400, the designer should investigate the effect of available passing sight distance on highway capacity using the procedures contained in the current edition of TRB **Highway Capacity Manual**. The designer should select the level of service to be used for design in accordance with ***Figure 301-1***.

If the available passing sight distance restricts the capacity from meeting the design level of service, adjustments should be made to the profile to increase the available passing sight distance. If, after making all feasible adjustments to the profile, capacity is still restricted below the design level of service due to the lack of sufficient passing sight distance, consideration should be given to providing passing lane sections or constructing a divided multi-lane facility.

201.5 Decision Sight Distance (DSD)

Although stopping sight distance is usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances, it may not provide sufficient visibility distances for drivers when information is difficult to perceive, or when unexpected maneuvers are required. In these circumstances, decision sight distance provides the greater length needed by drivers to reduce the likelihood for error in either information reception, decision making, or control actions.

The following are examples of locations where decision sight distance should be provided: entrance ramps and exit ramps at interchanges; diverging roadway terminals; changes in cross section such as toll plazas and lane drops; and areas of concentrated demand where there is apt to be "visual noise" (i.e., where sources of information compete, as those from roadway elements, traffic, traffic control devices, and advertising signs).

The decision sight distances in ***Figure 201-6***: (1) provide values for sight distances that are appropriate at critical locations and (2) serve as criteria in evaluating the suitability of the available sight distances at these critical locations. It is recommended that decision sight distances be provided at critical locations or that critical decision points be moved to locations where sufficient decision sight distance is available. If it is not practical to provide decision sight distance because of horizontal or vertical curvature constraints or if relocation of decision points is not practical, special attention should be given to the use of suitable traffic control devices for providing advance warning of the conditions that are likely to be encountered.

The decision sight distances listed in ***Figure 201-6*** vary depending on whether the location is on a rural or urban road and on the type of avoidance maneuver required to negotiate the location properly. For example, the recommended decision sight distance for a rural entrance ramp would be found in the avoidance maneuver C column opposite the appropriate design speed for the ramp location in question.

202 HORIZONTAL ALIGNMENT

202.1 General

A horizontal change in direction should, as far as economically feasible, be accomplished in a safe and comfortable manner. In addition to sight distance requirements, the most important design features to horizontal alignment design are degree of curve [curve radius], superelevation and spirals.

202.2 Maximum Centerline Deflection without Horizontal Curve

Figure 202-1 lists the maximum deflection angle which may be permitted without the use of a horizontal curve. The angle varies with the design speed of the facility.

202.3 Degree of Curve (Curve Radius)

The maximum degree of curve [minimum curve radius] is a limiting value of curvature for a given design speed and a maximum rate of superelevation. ***Figure 202-2*** shows this relationship.

202.4 Superelevation

202.4.1 Superelevation Rate

Superelevation rates for horizontal curves vary with location (urban/rural), degree of curvature [curve radius], and design speed.

Recommended superelevation rates for horizontal curves are shown in ***Figures 202-7, 202-8, 202-9, 202-9a and 202-10***. The rates in ***Figure 202-7*** apply to all rural highways and are based on a maximum superelevation rate of 0.08. Figure 202-8 contains the rates for high-speed urban highways (design speeds of 50 mph or greater). These are based on a maximum rate of 0.06. ***Figure 202-10*** is an extension of ***Figure 202-8*** in that it provides superelevation rates for curves with design speeds of 25-45 mph based on the maximum rate of 0.06. This table is to be used only for ramps or other interchange connector roadways in urban areas where horizontal alignment constraints preclude a higher design speed. The rates for low speed urban highways (45 mph or less) are contained in ***Figures 202-9*** and ***202-9a*** and are based on a maximum rate of 0.04

The table rates are derived by first calculating the maximum degree of curvature [minimum curve radius] for the design speed and assigning the maximum rate of superelevation to this curve. The maximum rates for flatter curves with the same design speed are then derived using AASHTO Method 5 (***Figures 202-7, 202-8 and 202-10***) or AASHTO Method 2 (***Figures 202-9 and 202-9a***) as described in the AASHTO Green Book under "Horizontal Alignment".

In attempting to apply the recommended superelevation rates for low-speed urban streets (***Figures 202-9 & 202-9a***) in built-up areas, various factors may combine to make these rates impractical to obtain. These factors would include wide pavements, adjacent development, drainage conditions, and frequent access points. In such cases, curves may be designed with reduced or no superelevation, although crown removal is a recommended minimum.

A design exception for superelevation rate is required whenever the superelevation rate required by ***Figures 202-7*** through ***202-10*** is not provided. A design exception for superelevation rate will not be required if a higher superelevation rate than what is required by ***Figures 202-7*** through ***202-10*** is provided as long as the respective maximum superelevation rate (0.08, 0.06 or 0.04) is not exceeded. Prior to the current update of this Manual, the maximum superelevation rate for rural highways was 0.083. A design exception for superelevation rate will not be required for existing rural highways that provide a superelevation rate greater than 0.08 but less than or equal to 0.083.

202.4.2 Effect of Grades on Superelevation

On long and fairly steep grades, drivers tend to travel somewhat slower in the upgrade direction and somewhat faster in the downgrade direction than on level roadways. In the case of divided highways, where each pavement can be superelevated independently, or on one-way roadways, such as ramps, this tendency should be recognized to see whether some adjustment in the superelevation rate would be desirable and/or feasible. On grades of 4 percent or greater with a length of 1000 ft. or more and a superelevation rate of 0.06 or more, the designer may adjust the superelevation rate by assuming a design speed which is 5 mph less in the upgrade direction and 5 mph higher in the downgrade direction, providing that the assumed design speed is not less than the legal speed. On two-lane, two-way roadways and on other multi-lane undivided roadways, such adjustments are less feasible, and should be disregarded.

202.4.3 Maximum Curvature Without Superelevation (Minimum Curve Radius Without Superelevation)

Figure 202-3 gives the maximum degree of curvature [minimum curve radius] which does not require superelevation based on the design speed and the rural/urban condition. This figure should be used in conjunction with ***Figures 202-7, 202-8, and 202-9*** to determine at what point in the "e_d" columns that superelevation becomes a design consideration. The corresponding data for ***Figure 202-10*** is contained on the figure.

202.4.4 Superelevation Methods

Figure 202-5 shows four methods by which superelevation is developed leading into and coming out of horizontal curves. Method 1 involves revolving the pavement about the centerline and is the most commonly used method. This method could be applied to multi-lane divided roadway sections where the divided segments are not crowned in a normal section. In this case, the median pavement edge acts as the "centerline".

Method 2 shows the pavement being revolved about the inner edge of traveled way and Method 3 uses the outer edge of traveled way as a rotation point. Both of these methods are used on a multi-lane divided roadway where the divided segments are crowned in a normal section. Since the control point for revolving the pavement is the median pavement edge, Method 2 would apply to the outer lanes and Method 3 would apply to the inner lanes. Method 2 is also used on undivided roadways where drainage problems preclude the use of Method 1. Method 4 revolves the pavement having a straight cross slope about the outside edge of traveled way. Method 4 would apply to single-lane or multi-lane ramps or roadways that are not crowned.

In reference to the above discussion on the superelevation of divided roadways, it is always preferable to use the median edge of traveled way as the rotation point. This greatly reduces the amount of distortion in grading the median area.

202.4.5 Superelevation Transition

The length of highway needed to change from a normal crown pavement section to a fully superelevated pavement section is referred to as the superelevation transition. The superelevation transition is divided into two parts - the tangent runout and the superelevation runoff.

The tangent runout (”Lt”) is the length required to remove the adverse pavement cross slope. As is shown on Method 1 of ***Figure 202-5***, this is the length needed to raise the "outside" edge of traveled way from a normal slope to a half-flat section (cross section A to cross section B of ***Figure 202-5***, Method 1).

The superelevation runoff ("Lr") is the length required to raise the "outside" edge of traveled way from a "half flat" section to a fully superelevated section (cross section B to cross section E of ***Figure 202-5***, Method 1). The length of transition required to remove the pavement crown is the distance between cross section A and cross section C **Figure 202-5** and is generally equal to twice the ”L_t” distance.

The minimum superelevation transition length is determined by multiplying the edge of traveled way correction by the equivalent slope rate ("G") shown on ***Figure 202-4***. The rate of change of superelevation should be constant throughout the transition. The values for "Lr" given in ***Figures 202-7***, **202-8** and **202-9** are based on two 12-foot lanes revolved about the centerline. "Lr" in ***Figure 202-10*** is based on one 16-foot lane revolved about the edge of traveled way. Use the equations provided on ***Figure 202-4*** to determine “Lr” for cases involving other lane widths or where more than one lane is being revolved about the centerline or baseline.

Figures 202-5a through ***202-5d*** have been provided to show the designer how to develop the superelevation transitions for a two-lane undivided highway (***Figure 202-5a***), a four-lane divided highway (**Figure 202-5b**) and a six-lane divided highway (***Figures 202-5c & d***). ***Figure 202-5c*** could also be used for a four-lane divided highway with future median lanes and ***Figure 202-5d*** could also be used for a four-lane divided highway with future outside lanes.

202.4.6 Superelevation Position

Figures 202-5a through ***202-5d*** show the recommended positioning of the proposed superelevation transition in relationship to the horizontal curve.

For those curves with spirals, the transition from adverse crown removal to full superelevation shall occur within the limits of the spiral. In other words, the spiral length shall equal the "Lr" value.

For simple curves without spirals, the "Lr" transition shall be placed so that 50 percent to 70 percent of the maximum superelevation rate is outside the curve limits (P.C., P.T.). It is recommended that, whenever possible, 2/3 of the full superelevation rate be present at the P.C. and P.T. In addition, whenever possible, full superelevation should be maintained for at least 1/3 the length of the curve.

202.4.7 Profiles and Elevations

Breakpoints at the beginning and end of the superelevation transition should be rounded to obtain a smooth profile. One suggestion is to use a "vertical curve" on the edge of traveled way profile with a length in feet equal to the design speed in mph (i.e., 45 ft. for 45 mph).

The final construction plans should have superelevation tables or pavement details showing the proposed elevations at the centerline, edges of traveled way, and if applicable, lane lines or other breaks in the cross slope. Pavement or lane widths should be included where these widths are in transition.

Edge of traveled way profiles should be plotted to an exaggerated scale within the limits of the superelevation transition to check calculations and to determine the location of drainage basins. Adjustments should be made to obtain smooth profiles. These profiles should be submitted as part of the Stage 1 submission in order to facilitate review of the proposed data. Special care should be used in determining edge elevations in a transition area when the profile grade is on a vertical curve.

202.4.8 Superelevation Between Reverse Horizontal Curves

Figure 202-6 illustrates schematically two methods for positioning the superelevation transition between two reverse horizontal curves. In both diagrams each curve has its own "L_r" value (L_r1, L_r2) depending on the degree of curvature, and the superelevation is revolved about the centerline.

The first (top) diagram involves two simple curves. In the case of new or relocated alignment, the P.T. of the first curve and P.C. of the second curve should be separated by enough distance to allow a smooth continuous transition between the curves at a rate not exceeding the "G" value in the table on ***Figure 202-4*** for the design speed. This requires that the distance be not less than 50 percent nor greater than 70 percent of $L_r1 + L_r2$. Two-thirds is the recommended portion. When adapting this procedure to existing curves where no alignment revision is proposed, the transition should conform as closely as possible to the above criteria. These designs will be reviewed on a case by case basis.

The second (or lower) diagram involves two spiral curves. Where spiral transitions are used, the S.T. of the first curve and the T.S. of the second curve may be at, or nearly at, the same point, without causing superelevation problems. In these cases, the crown should not be re-established as shown in ***Figure 202-5***, but instead, both edges of traveled way should be in continual transition between the curves, as shown in ***Figure 202-6***.

202.5 Spirals

The combination of high speed and sharp curvature leads to longer transition paths, which can result in shifts in lateral position and sometimes actual encroachment on adjoining lanes. Spirals make it easier for the driver to keep the vehicle within its own lane. Spirals are to be used on projects involving new alignment or substantial modifications to the existing alignment based on the maximum degree of curve as shown in ***Figure 202-11***. The length of the spiral should be equal or to greater than the superelevation runoff length "L_r" for the curve, as determined in ***Section 202.4.5.*** This section also discussed the role of the spiral in attaining proper superelevation for the curve.

The above criterion for using spirals is not intended to discourage their use in other design situations. In fact, spirals are recommended for use as a good mitigation feature in achieving full superelevation regardless of design speed or degree of curvature [curve radius]. See ***Figure 1303-2*** of the **Location and Design Manual, Volume 3**, for more details on spiral curve elements and layout.

203 VERTICAL ALIGNMENT

203.1 General

In addition to sight distance requirements, design features most important to vertical alignment design are grades and vertical curves.

203.2 Grades

203.2.1 Maximum Grades

Maximum percent grades based on functional classification, terrain and design speed are shown in ***Figure 203-1***. The maximum design grade should be used infrequently, rather than a value to be used in most cases.

203.2.2 Minimum Grades

Flat and level grades on uncurbed pavements are virtually without objection when the pavement is adequately crowned to drain the surface laterally. With curbed pavements, sufficient longitudinal grades should be provided to facilitate surface drainage. The preferred minimum grade for curbed pavements is 0.5 percent, but a grade of 0.3 percent may be used where there is a high-type pavement accurately crowned and supported on firm subgrade.

203.2.3 Critical Lengths of Grades

Freedom and safety of operation on 2-lane highways are adversely affected by heavily loaded vehicles operating on grades of insufficient lengths to result in speeds that could impede following vehicles.

The term “critical length of grade” is used to describe the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed.

The length of any given grade that will cause the speed of a typical heavy truck (200 lb/hp) to be reduced by various amounts below the average running speed of all traffic is shown graphically in ***Figure 203-1a***. The curve showing a 10-mph speed reduction is used as the general design guide for determining the “critical lengths of grade”.

If after investigation of the project grade line, it is found that critical length of grade must be exceeded, an analysis of the effect of long grades on the level of service should be made. Where speeds resulting from trucks climbing up long grades are calculated to fall within the range of service level D, or lower, consideration should be given to constructing added uphill lanes on critical lengths of grade. When uphill lanes are added for truck traffic, the lane should extend a sufficient distance past the crest of the hill to allow truck traffic to obtain a reasonable speed before being required to merge into the through lanes.

Where the length of added lanes needed to preserve the recommended level of service on sections with long grades exceeds 10 percent of the total distance between major termini, consideration should be given to the ultimate construction of a divided multi-lane facility.

203.3 Vertical Curves

203.3.1 General

A vertical curve is used to provide a smooth transition between vertical tangents of different slope rates. It is a parabolic curve and is usually centered on the intersection point of the vertical tangents.

One of the basic principles of parabolic curves is that the rate of change of grade at successive points on the curve is a constant amount for equal increments of horizontal distance. The total length (L) of a vertical curve divided by the algebraic difference in its tangent grades (A) reflects the distance along the curve at any point to effect a 1 percent change in gradient and is, therefore, a measure of curvature. The rate L/A, termed "K", is useful in determining minimum lengths of vertical curves for the various required sight distances.

203.3.2 Grade Breaks

The maximum break in grade permitted without using a vertical curve is shown in ***Figure 203-2***. The maximum grade change is based on comfort control and varies with the design speed.

203.3.3 Crest Vertical Curves

The major control for safe operation on crest vertical curves is the provision for ample sight distances for the design speed.

Figure 203-3 includes "K" values for crest vertical curves along with other appropriate equations.

Figure 203-4 shows the relationship between the length of crest vertical curve to the stopping sight distance.

In addition to being designed for safe stopping sight distance, crest vertical curves should be designed for comfortable operation and a pleasing appearance. Accordingly, the length of a crest vertical curve in feet should be, as a minimum, 3 times the design speed in mph.

203.3.4 Sag Vertical Curves

For sag vertical curves the primary design criteria is headlight sight distance. When a vehicle traverses an unlighted sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. For overall safety on highways, the required headlight sight distance is assumed to be equal to stopping sight distance. Based on a headlight height of 2 ft., and a 1 degree upward divergence of the light beam, equations showing the relationship of curve length, algebraic grade difference, and stopping sight distance are included on ***Figure 203-6***. ***Figure 203-7*** shows this same relationship in graphic form.

It should be noted that, for sag curves, when the algebraic difference of grades is 1.75 percent or less, stopping sight distance is not restricted by the curve. In these cases the formula on ***Figure 203-6*** will not provide meaningful answers.

Minimum lengths of sag vertical curves are necessary to provide a pleasing general appearance to the highway. Accordingly, the length of sag vertical curves in feet in should be, as a minimum, 3 times the design speed.

203.3.5 Tangent Offsets for Vertical Curves

For the designer's convenience, ***Figure 203-8***, showing tangent offsets, is included.

204 HORIZONTAL AND VERTICAL ALIGNMENT CONSIDERATIONS

204.1 General

There are many controls to consider when designing horizontal and vertical alignments. These controls are separated into horizontal, vertical and horizontal/vertical coordination. It would be virtually impossible to meet each of these. Some even tend to conflict, and compromises will have to be made. The considerations listed in each category are guidelines and suggestions to assist the designer in obtaining a more optimal design.

204.2 Horizontal Considerations

- Alignment should be as directional as possible while still being consistent with topography and the preservation of developed properties and community values.
- Use of maximum degree of curvature [minimum curve radius] should be avoided wherever possible.
- Consistent alignment should be sought.
- Curves should be long enough to avoid the appearance of a sudden or abrupt change in direction.
- Tangents and/or flat curves should be provided on high, long fills.
- Compound curves should only be used with caution.

204.3 Vertical Considerations

- A smooth grade with gradual changes consistent with the type of facility and character of terrain should be strived for.
- The "roller-coaster" or the "hidden-dip" type of profile should be avoided.
- Undulating gradelines involving substantial lengths of steeper grades should be appraised for their effect on traffic operation since they may encourage excessive truck speeds.
- Broken-back gradelines (two crest or sag vertical curves separated by short tangent grade) generally should be avoided.
- Special attention should be given on curbed sections to drainage where vertical curves having a K value in excess of 167 are used.
- It is preferable to avoid long sustained grades by breaking them into shorter intervals with steeper grades at the bottom.

204.4 Coordination of Horizontal and Vertical Alignments

Curvature and grades should be properly balanced. Normally horizontal curves will be longer than vertical curves.

- Vertical curvature superimposed on horizontal curvature is generally more pleasing. P.I.'s of both vertical and horizontal curves should nearly coincide.
- Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve or at or near the low point of a pronounced sag vertical curve.
- On two-lane roads, long tangent sections are desirable to provide adequate passing sections.
- Horizontal and vertical curves should be as flat as possible at intersections.
- On divided highways the use of variable median widths and separate horizontal and vertical alignments should be considered
- In urban areas, horizontal and vertical alignments should be designed to minimize nuisance factors.
- These might include directional adjustment to increase buffer zones and depressed roadways to decrease noise.
- And vertical alignments may often be adjusted to enhance views of scenic areas.

LIST OF FIGURES

* Note: For design criteria pertaining to Collectors and Local Roads with ADT’s less than 400, please refer to the AASHTO Publication - Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ▫ 400).

STOPPING SIGHT DISTANCE

201-1

REFERENCE SECTION 201.2 & 201.2.1

HEIGHT OF EYE 3.50′

HEIGHT OF OBJECT 2.00′

$SSD = 1.47Vt + 1.075V^2 \div a$

SSD = stopping sight distance, ft;
t = brake reaction time, 2.5s;
V = design speed, mph;
a = deceleration rate, $11.2 ft/s^2$

DESIGN SPEED (mph)	DESIGN SSD (feet)	DESIGN SPEED (mph)	DESIGN SSD (feet)
20	115	48	400
21	120	49	415
22	130	50	425
23	140	51	440
24	145	52	455
25	155	53	465
26	165	54	480
27	170	55	495
28	180	56	510
29	190	57	525
30	200	58	540
31	210	59	555
32	220	60	570
33	230	61	585
34	240	62	600
35	250	63	615
36	260	64	630
37	270	65	645
38	280	66	665
39	290	67	680
40	305	68	695
41	315	69	715
42	325	70	730
43	340	71	745
44	350	72	765
45	360	73	780
46	375	74	800
47	385	75	820

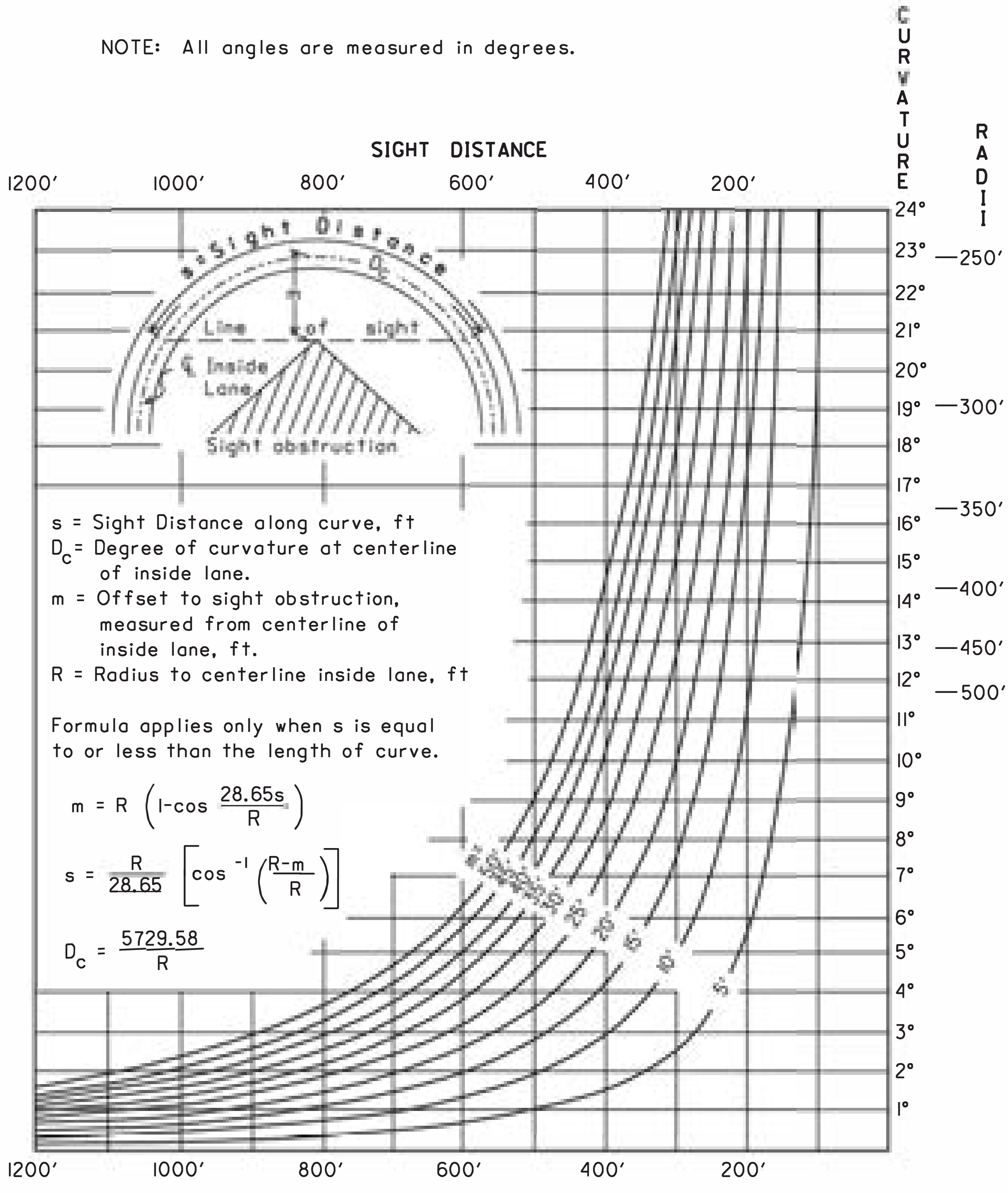
HORIZONTAL SIGHT DISTANCE

201-2

REFERENCE SECTION

201.2.1

NOTE: All angles are measured in degrees.



s = Sight Distance along curve, ft

D_c = Degree of curvature at centerline of inside lane.

m = Offset to sight obstruction, measured from centerline of inside lane, ft.

R = Radius to centerline inside lane, ft

Formula applies only when s is equal to or less than the length of curve.

$$m = R\left(1-\cos\frac{28.65s}{R}\right)$$

$$s = \frac{R}{28.65}\left[\cos^{-1}\left(\frac{R-m}{R}\right)\right]$$

$$D_c = \frac{5729.58}{R}$$

SIGHT DISTANCE

When a combination of spirals, tangents and/or curves are present, the horizontal sight distance should be determined graphically.

MINIMUM PASSING SIGHT DISTANCE

HEIGHT OF EYE 3.50' - HEIGHT OF OBJECT 3.50'

DESIGN SPEED (mph)	PASSING SIGHT DISTANCE (PSD)	
	Minimum PSD (ft.)	K-Crest Vert. Curv.
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1000	357
65	1100	432
70	1200	514

Using: S = Minimum Passing Sight Distance
L = Length of Crest Vertical Curve
A = Algebraic Difference in Grades (%), Absolute Value
K = Rate of Vertical Curvature

- For a given design speed and an "A" value, the calculated length "L" = K x A

- To determine "S" with a given "L" and "A", use the following:
  For S<L: $S = 52.92\sqrt{K}$, where K = L/A
  For S>L: S = 1400/A + L/2

INTERSECTION SIGHT TRIANGLES

201-4

REFERENCE SECTION 201.3.1 & 201.3.3

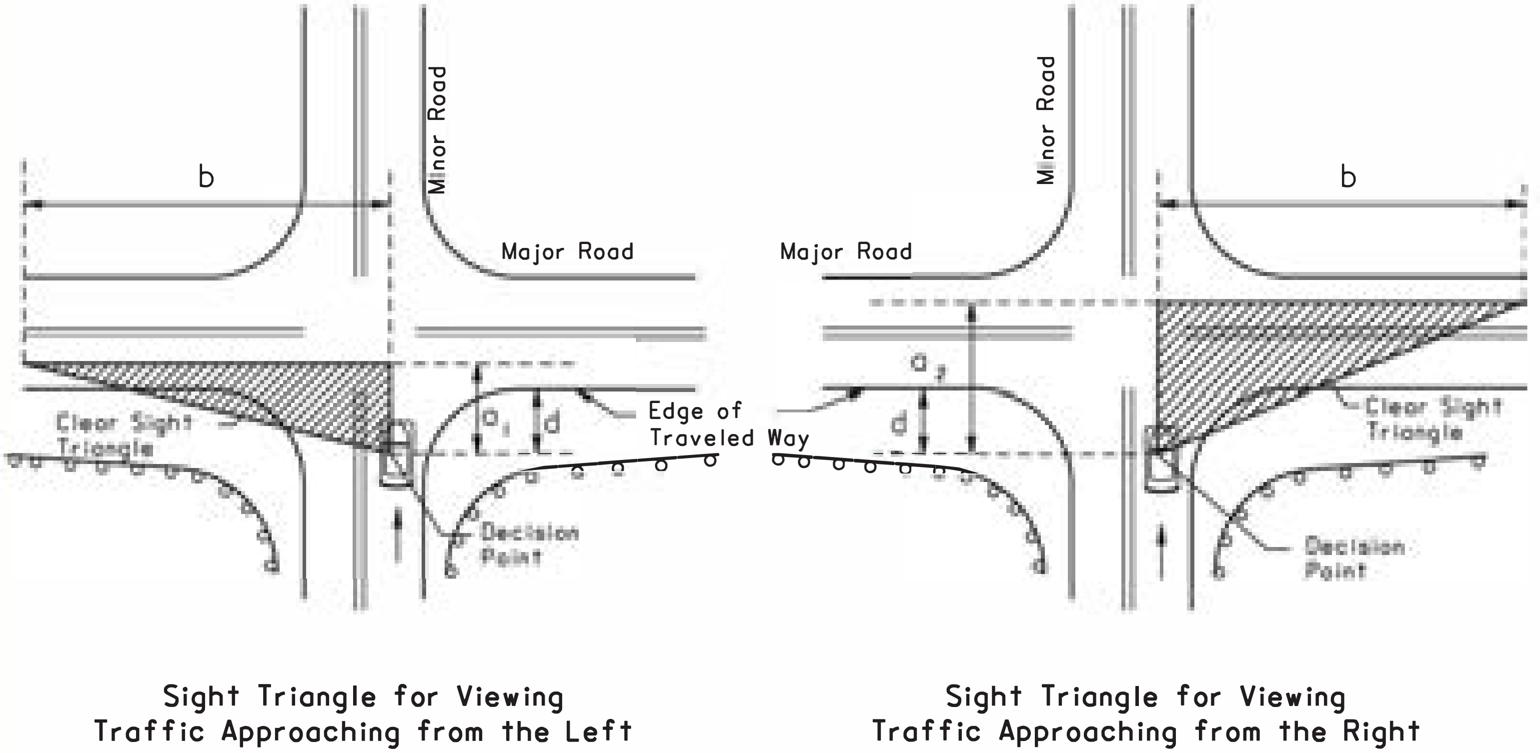


DIAGRAM A - SIGHT TRIANGLES

a_1 = The distance, along the minor road, from the decision point to 1/2 the lane width of the approaching vehicle on the major road.

a_2 = The distance, along the minor road, from the decision point to 1 1/2 the lane width of the approaching vehicle on the major road.

b = Intersection Sight Distance

d = The distance from the edge of the traveled way of the major road to the decision point. The distance should be a minimum of 14.4' and 17.8' preferred.

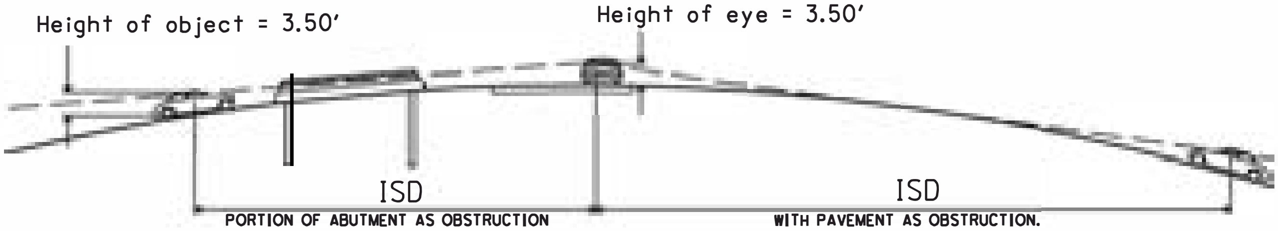


DIAGRAM B - VERTICAL COMPONENTS (Sec. 201.3.3)

INTERSECTION SIGHT DISTANCE

201-5

REFERENCE SECTION
201.3, 201.3.1,
201.3.2 & 201.3.3

(See Following Page for Additional Figures & Notes)

HEIGHT OF EYE 3.50′ HEIGHT OF OBJECT 3.50′

DESIGN SPEED (mph)	Passenger Cars Completing a Left Turn from a Stop (assuming a t_g of 7.5 sec.)		Passenger Cars Completing a Right Turn from a Stop or Crossing Maneuuver (assuming a t_g of 6.5 sec.)	
	ISD (ft.)	K-CREST VERT. CURVE	ISD (ft.)	K-CREST VERT. CURVE
15	170	10	145	8
20	225	18	195	14
25	280	28	240	21
30	335	40	290	30
35	390	54	335	40
40	445	71	385	53
45	500	89	430	66
50	555	110	480	82
55	610	133	530	100
60	665	158	575	118
65	720	185	625	140
70	775	214	670	160

If ISD cannot be provided due to environmental or R/W constraints, then as a minimum, the SSD for vehicles on the major road should be provided.

$$\boxed{ISD = 1.47 \times V_{major} \times t_g}$$

ISD = intersection sight distance (ft.)

V_{major} = design speed of major road (mph)

t_g = time gap for minor road vehicle to enter the major road (sec.)

Using: S = Intersection Sight Distance
L = Length of Crest Vertical Curve
A = Algebraic Difference in Grades (%), Absolute Value
K = Rate of Vertical Curvature

- For a given design speed and an "A" value, the calculated length "L" = K x A

- To determine "S" with a given "L" and "A", use the following:

For S<L: $S = 52.92\sqrt{K}$, where K = L/A

For S>L: S = 1400/A + L/2

INTERSECTION SIGHT DISTANCE

(Continued Figures & Notes)

Time Gaps

		Design Vehicle	Time gap(s) at design speed of major road (t_g)
(A)	Left Turn from a Stop	Passenger car	7.5 sec.
		Single-unit truck	9.5 sec.
		Combination truck	11.5 sec.
(B)	RightTurn from a Stop or Crossing Manuever	Passenger car	6.5 sec.
		Single-unit truck	8.5 sec.
		Combination truck	10.5 sec.

A. Note: The ISD & time gaps shown in the above tables are for a stopped vehicle to turn left onto a two-lane highway with no median and grades of 3 % or less. For other conditions, the time gap must be adjusted as follows:

For multilane highways:

For left turns onto two-way highways with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.

For minor road approach grades:

If the approach grade is an upgrade that exceeds 3 %, add 0.2 seconds for each % grade for left turns.

B. Note: The ISD & time gaps shown in the above tables are for a stopped vehicle to turn right onto a two-lane highway with no median and grades of 3 % or less. For other conditions, the time gap must be adjusted as follows:

For multilane highways:

For crossing a major road with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane to be crossed and for narrow medians that cannot store the design vehicle.

For minor road approach grades:

If the approach grade is an upgrade that exceeds 3 %, add 0.1 seconds for each % grade.

# DECISION SIGHT DISTANCE	201-6 REFERENCE SECTION 201.5

HEIGHT OF EYE 3.50’ HEIGHT OF OBJECT 2.00’

DESIGN SPEED (mph)	DECISION SIGHT DISTANCE (ft) AVOIDANCE MANEUVER A	B	C	D	E
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445
75	875	1545	1180	1365	1545

The Avoidance Maneuvers are as follows:

A – Rural Stop
B – Urban Stop
C – Rural Speed/Path/Direction Change
D – Suburban Speed/Path/Direction Change
E – Urban Speed/Path/Direction Change

Decision Sight Distance (DSD) is calculated or measured using the same criteria as Stopping Sight Distance; 3.50 ft eye height and 2.00 ft object height.
Use the equations on Figures 201-2, 203-3, and 203-6 to determine DSD at vertical and horizontal curves.

MAXIMUM CENTERLINE DEFLECTION WITHOUT HORIZONTAL CURVE	202-1
	REFERENCE SECTION 202.2

DESIGN SPEED (mph)		MAX. DEFLECTION *
LOW SPEED	25	5° 30'
	30	3° 45'
	35	2° 45'
	40	2° 05'
	45	1° 40'
HIGH SPEED	50	1° 05'
	55	1° 00'
	60	0° 55'
	65	0° 50'
	70	0° 45'
	75	0° 45'

* ROUNDED TO NEAREST 5'

Based on the Allowable Pavement Transition formulae (301.1.4):
High Speed: $\tan\Delta = 1.0/V$
Low Speed: $\tan\Delta = 60/V^2$
Where: V = Design Speed
Δ = Deflection Angle

Note:
The recommended minimum distances between consecutive horizontal deflections is:
High Speed – 200’
Low Speed – 100’

MAXIMUM DEGREE OF CURVE

202-2

REFERENCE SECTION 202.3

DESIGN SPEED (mph)	MAXIMUM DEGREE OF CURVE (A)			
	RURAL	HIGH-SPEED URBAN	LOW-SPEED URBAN RAMPS/ INTERCHANGE	LOW-SPEED URBAN
	MAXIMUM SUPERELEVATION RATE			
	.08	.06	.06	.04
20	--	--	--	66°30'
21	--	--	--	58°15'
22	--	--	--	51°15'
23	--	--	--	45°15'
24	--	--	--	40°15'
25	42°30'	--	39°45'	37°00'
26	38°00'	--	36°00'	33°00'
27	34°00'	--	32°45'	29°30'
28	31°15'	--	29°45'	26°45'
29	28°30'	--	27°15'	24°30'
30	26°45'	--	24°45'	22°45'
31	24°30'	--	23°00'	21°00'
32	22°30'	--	21°15'	19°15'
33	21°15'	--	19°30'	18°00'
34	19°45'	--	18°15'	16°45'
35	18°15'	--	16°45'	15°30'
36	16°45'	--	15°45'	14°15'
37	15°45'	--	14°30'	13°15'
38	14°45'	--	13°30'	12°30'
39	13°45'	--	12°45'	11°30'
40	12°45'	--	11°45'	10°45'
41	12°15'	--	11°15'	10°15'
42	11°45'	--	10°30'	9°45'
43	11°00'	--	10°00'	9°00'
44	10°30'	--	9°30'	8°45'

DESIGN SPEED (mph)	MAXIMUM DEGREE OF CURVE (A)			
	RURAL	HIGH-SPEED URBAN	LOW-SPEED URBAN RAMPS/ INTERCHANGE	LOW-SPEED URBAN
	MAXIMUM SUPERELEVATION RATE			
	.08	.06	.06	.04
45	9°45'	--	9°00'	8°00'
46	9°15'	--	--	--
47	8°45'	--	--	--
48	8°15'	--	--	--
49	8°00'	--	--	--
50	7°30'	6°45'	--	--
51	7°15'	6°30'	--	--
52	6°45'	6°15'	--	--
53	6°30'	6°00'	--	--
54	6°15'	5°45'	--	--
55	6°00'	5°30'	--	--
56	5°45'	5°15'	--	--
57	5°30'	5°00'	--	--
58	5°15'	4°45'	--	--
59	5°00'	4°30'	--	--
60	4°45'	4°15'	--	--
61	4°30'	4°00'	--	--
62	4°15'	4°00'	--	--
63	4°15'	3°45'	--	--
64	4°00'	3°30'	--	--
65	3°45'	3°30'	--	--
66	3°45'	3°15'	--	--
67	3°30'	3°15'	--	--
68	3°30'	3°00'	--	--
69	3°15'	3°00'	--	--
70	3°15'	2°45'	--	--
71	3°00'	2°45'	--	--
72	3°00'	2°30'	--	--
73	2°45'	2°30'	--	--
74	2°45'	2°15'	--	--
75	2°30'	2°15'	--	--

(A) See Superelevation Tables 202-7, 8, 9, and 10 for corresponding radii values.

MAXIMUM DEGREE OF CURVE WITHOUT SUPERELEVATION

202-3

REFERENCE SECTION 202.4.3

	DESIGN SPEED (mph)	DEGREE OF CURVE: RURAL HIGHWAYS		DEGREE OF CURVE: URBAN STREETS & HIGHWAYS	
LOW SPEED	20	--	FIGURE 202-7	54° 23'	FIGURE 202-9
	25	2° 35'		29° 20'	
	30	1° 53'		17° 30'	
	35	1° 26'		11° 28'	
	40	1° 08'		7° 42'	
	45	0° 55'		5° 40'	
HIGH SPEED	50	0° 45'		0° 47'	FIGURE 202-8
	55	0° 38'		0° 39'	
	60	0° 32'		0° 33'	
	65	0° 28'		0° 29'	
	70	0° 25'		0° 26'	
	75	0° 23'		0° 23'	

SUPERELEVATION TRANSITIONS

202-4

REFERENCE SECTION 202.4.5 & 202.4.8

Maximum Relative Gradient for Profiles Between the Edge of Traveled Way and the Centerline or Reference Line (Axis of Rotation)		
Design Speed (mph)	Maximum Relative Gradient (Percent) "Δ"	Equivalent Maximum Relative Slope "G"
20	0.74	135:1
25	0.70	143:1
30	0.66	152:1
35	0.62	161:1
40	0.58	172:1
45	0.54	185:1
50	0.50	200:1
55	0.47	213:1
60	0.45	222:1
65	0.43	233:1
70	0.40	250:1
75	0.38	263:1

Adjustment Factors, b_w		
Number of Lanes, Rotated n_1	Divided * Roadways b_w	Undivided Roadways b_w
1	1.00	1.00
1.5	1.00	0.83
2	1.00	0.75
2.5	1.00	0.70
3	1.00	0.67
3.5	1.00	0.64

* Interstates, Freeways, Expressways and Ramps

In Figures 202-7, 202-8 and 202-10, the table values for the Minimum Length of Superelevation Runoff, L_r, were determined by the following equation:

$L_r = \frac{(w\ x\ n_1)e_d}{\Delta}(b_w)$ x 100 or $L_r = (w \text{ x } n_1)(e_d)(G)(b_w)$

The equation can also be used to determine L_r, when more than one lane is rotated about the centerline or the edge or if the lane width is other than 12 feet for Figures 202-7 and 202-8 or 16 feet for Figure 202-10.

Once L_r has been determined, the Minimum Length of Tangent Runout, L_t, should be determined by the following equation:

$L_t = (e_{NC} \div e_d)\ L_r$

The equation for L_t can be used by Figures 202-7, 202-8, 202-9 and 202-10.

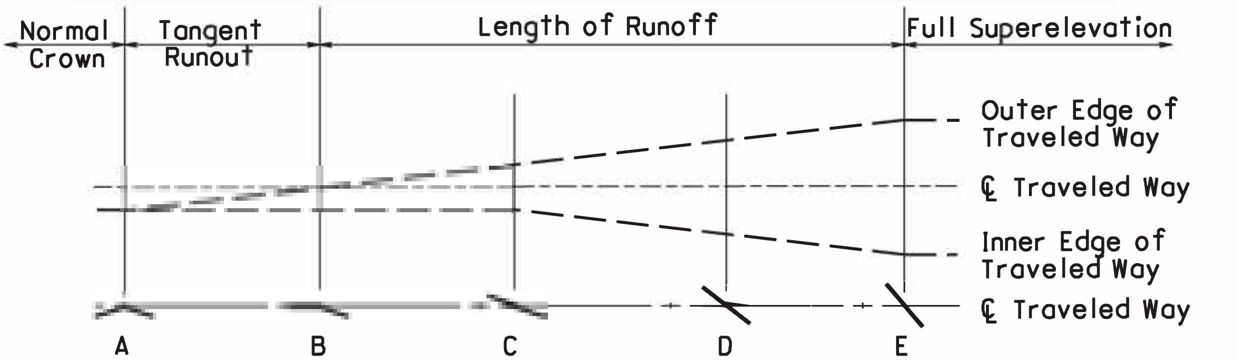
Where: L_r = minimum length of superelevation runoff, ft
L_t = minimum length of tangent runout, ft
Δ = maximum relative gradient, percent
n_1 = number of lanes rotated
w = width of one traffic lane, ft (typically 12 ft)
e_d = design superelevation rate
e_{NC} = normal cross slope rate, (0.016)
G = equivalent maximum relative slope, (the reciprocal of Δ)
b_w = adjustment factor for number of lanes rotated

METHODS OF SUPERELEVATION ROTATION

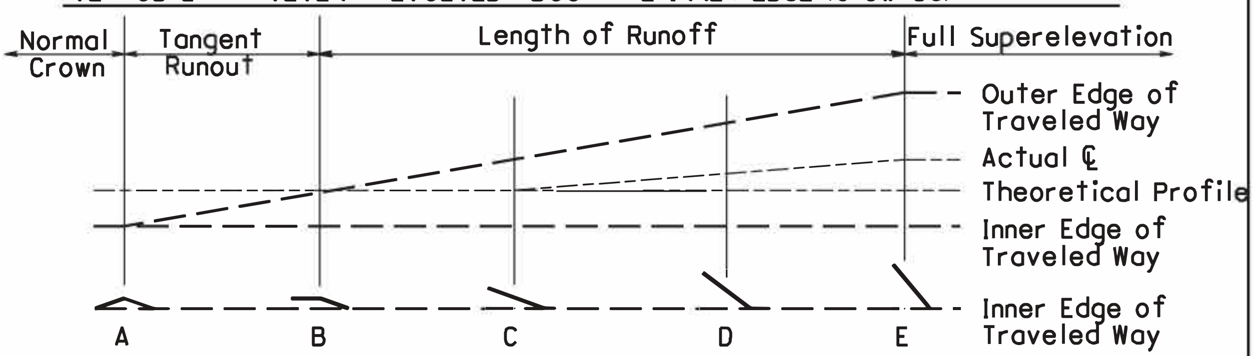
202-5

REFERENCE SECTION 202.4.4

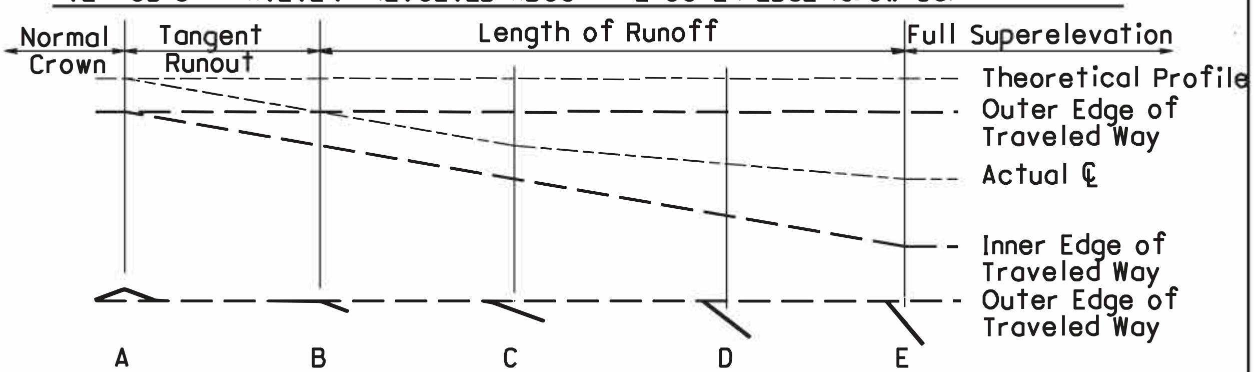
METHOD 1 - PAVEMENT REVOLVED ABOUT THE CENTERLINE (Crowned)



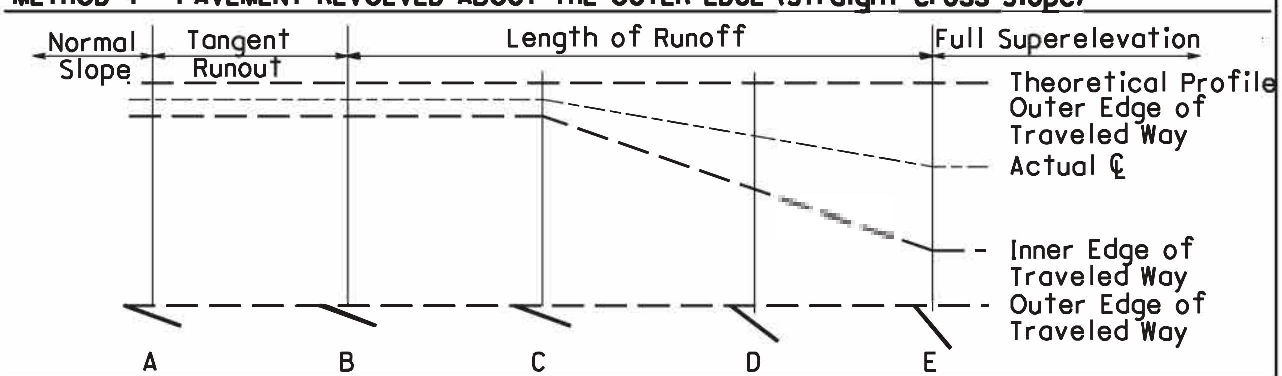
METHOD 2 - PAVEMENT REVOLVED ABOUT THE INNER EDGE (Crowned)



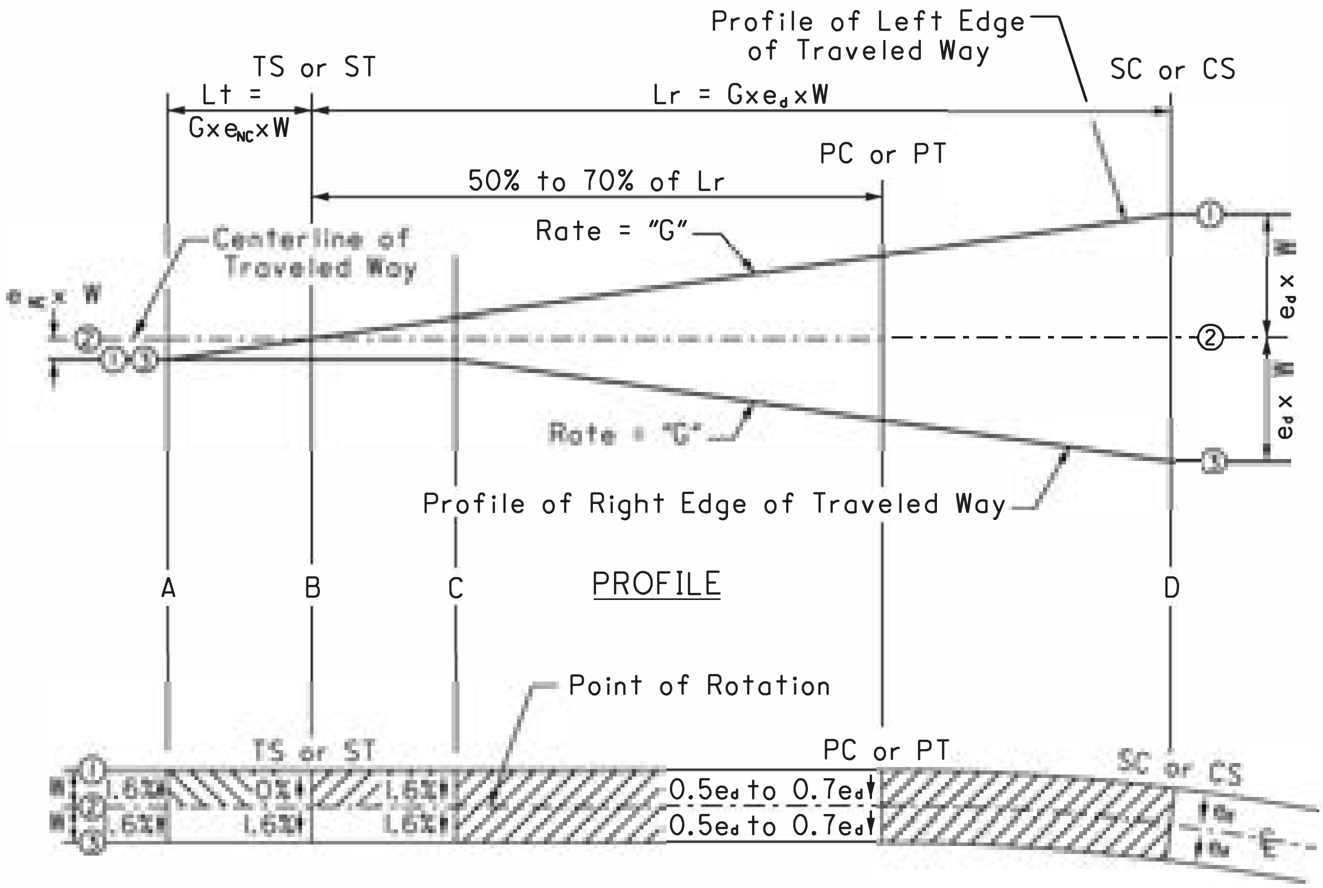
METHOD 3 - PAVEMENT REVOLVED ABOUT THE OUTER EDGE (Crowned)

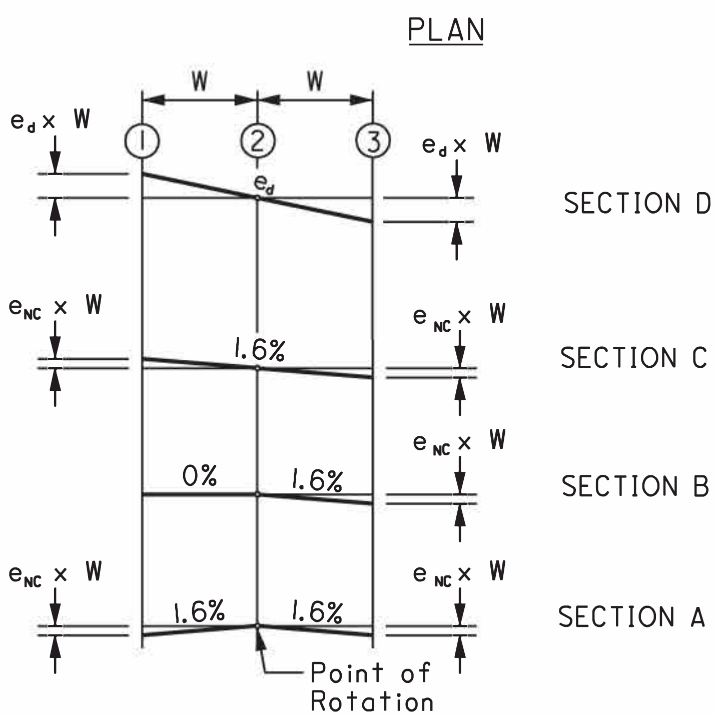


METHOD 4 - PAVEMENT REVOLVED ABOUT THE OUTER EDGE (Straight Cross Slope)



SUPERELEVATION DEVELOPMENT TWO-LANE UNDIVIDED	202-5a
	REFERENCE SECTION 202.4.4 - 202.4.6, 202.4.8



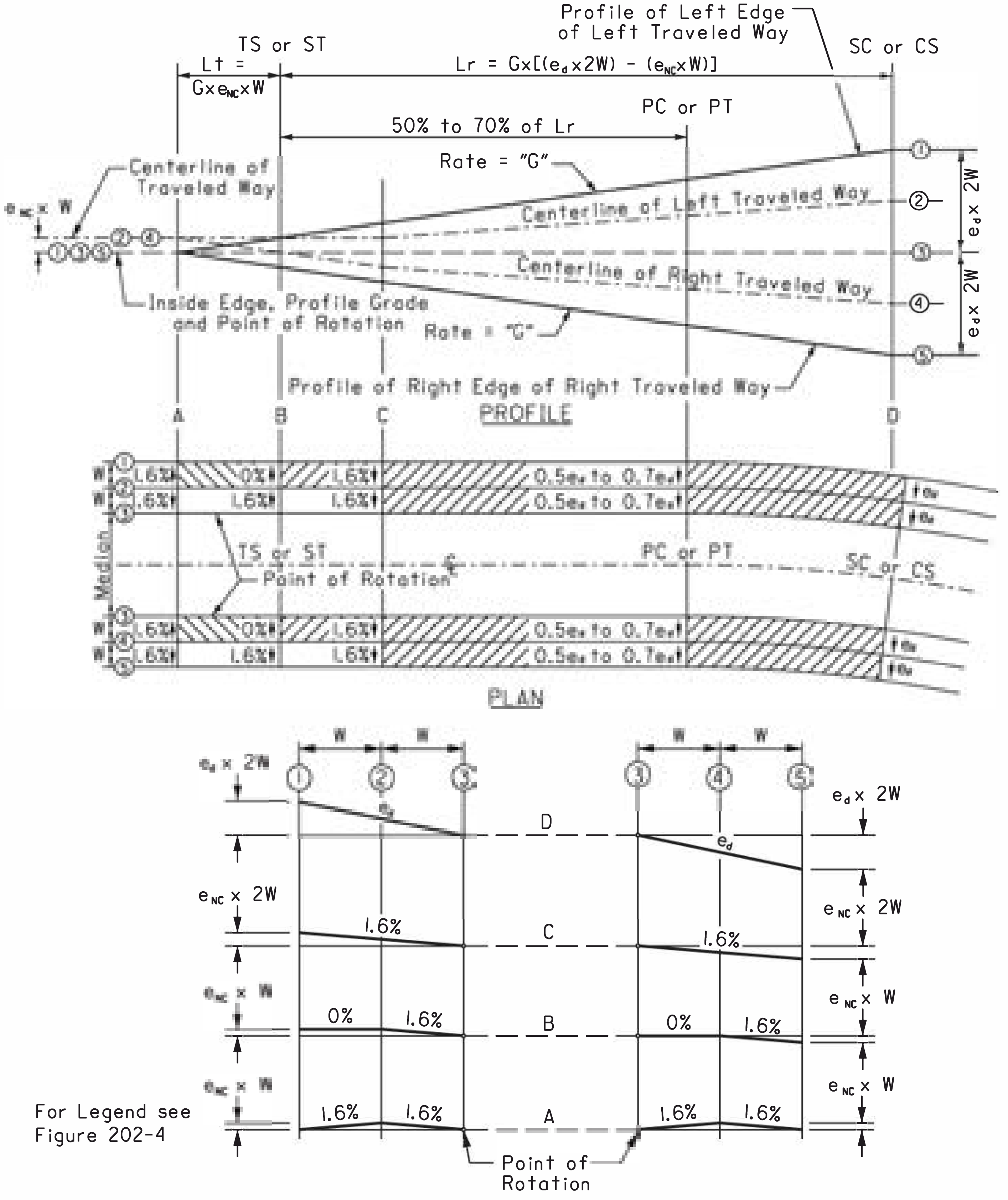


For Legend see Figure 202-4

SUPERELEVATION DEVELOPMENT FOUR-LANE DIVIDED

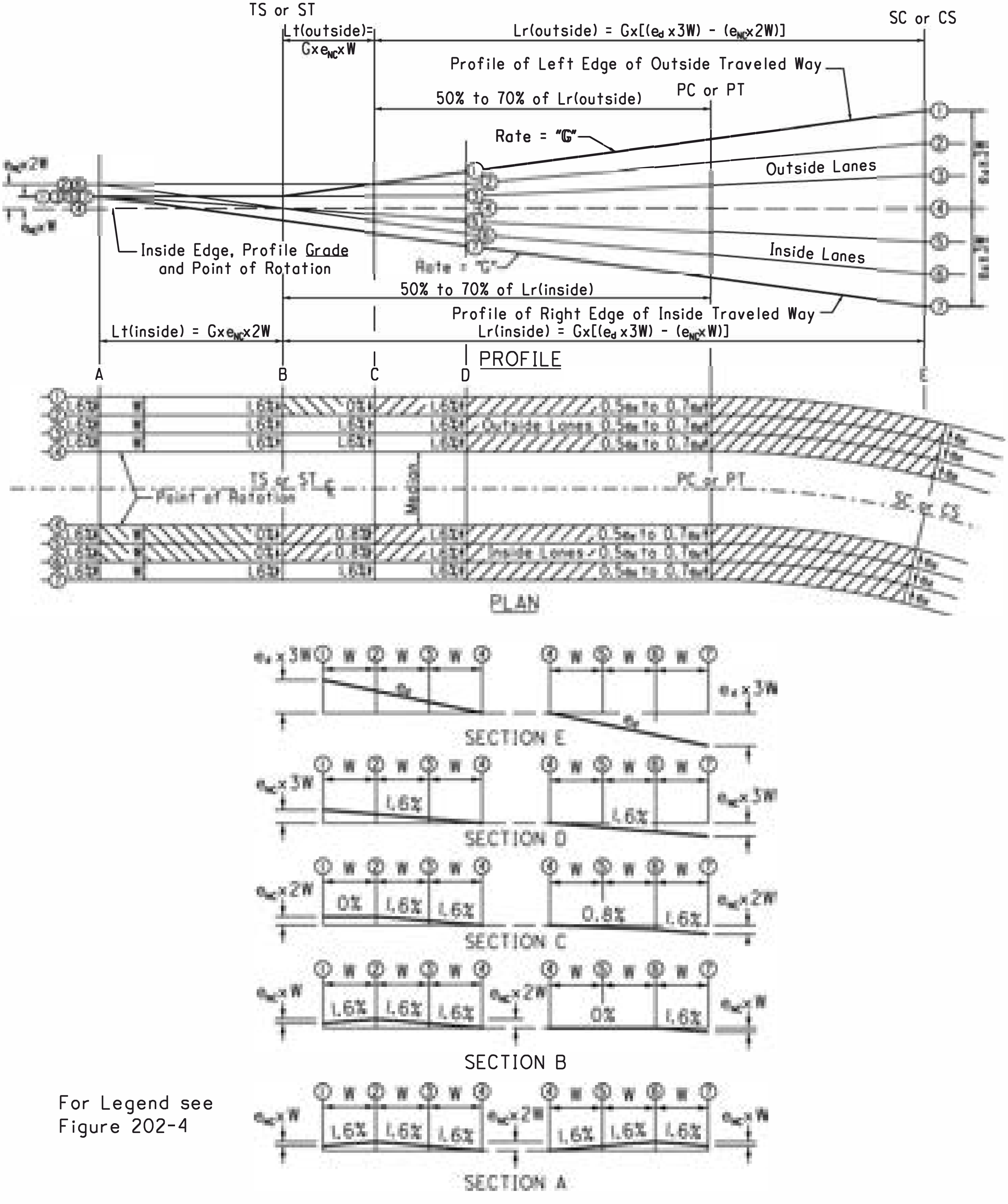
202-5b

REFERENCE SECTION 202.4.4 - 202.4.6, 202.4.8



For Legend see Figure 202-4

SUPERELEVATION DEVELOPMENT SIX-LANE OR MORE DIVIDED (OR FOUR-LANE DIVIDED WITH FUTURE MEDIAN LANES)	202-5c REFERENCE SECTION 202.4.4 - 202.4.6, 202.4.8

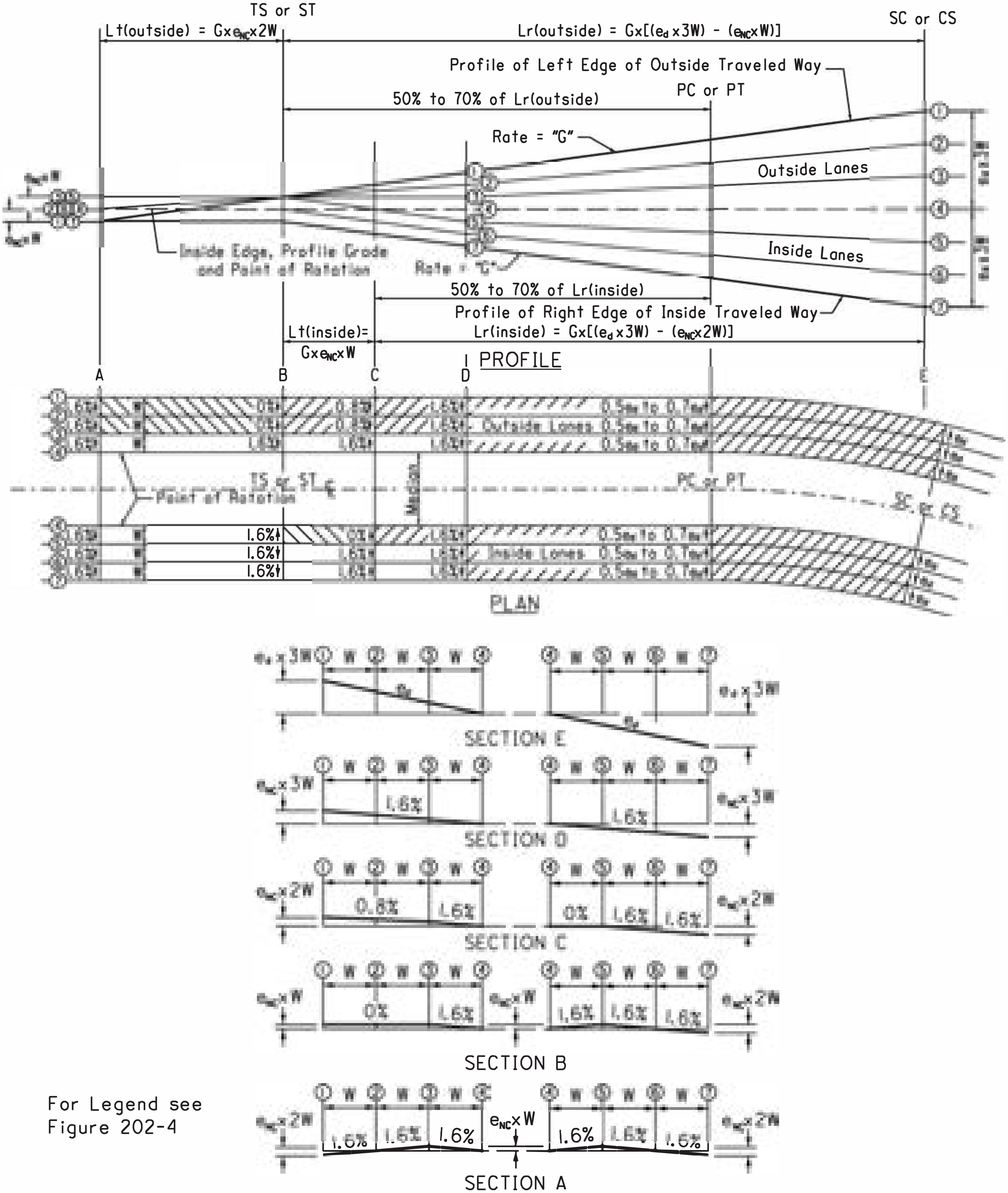


For Legend see Figure 202-4

SUPERELEVATION DEVELOPMENT SIX-LANE OR MORE DIVIDED (OR FOUR-LANE DIVIDED WITH FUTURE OUTSIDE LANES)

202-5d

REFERENCE SECTION 202.4.4 - 202.4.6, 202.4.8

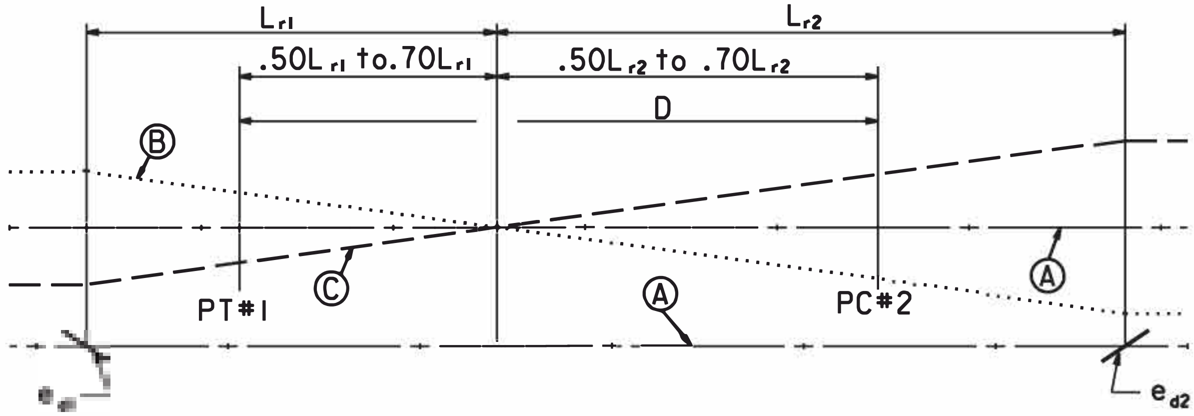


For Legend see Figure 202-4

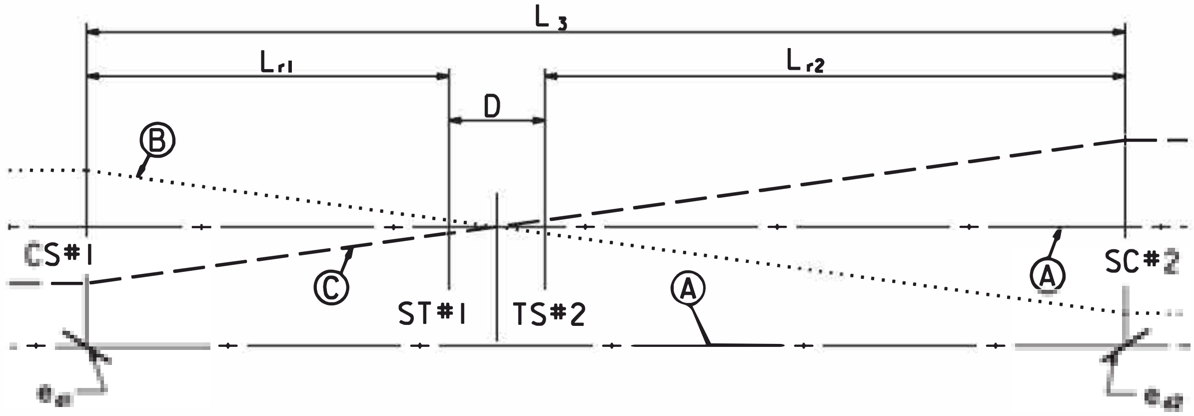
SUPERELEVATION TRANSITION BETWEEN REVERSE HORIZONTAL CURVES

202-6

REFERENCE SECTION
202.4.8



SIMPLE CURVES



SPIRAL CURVES

LEGEND:

- A - Centerline Pavement
- B - Outside E.P. Curve 1, Inside E.P. Curve 2
- C - Inside E.P. Curve 1, Outside E.P. Curve 2
- e_{d1}, e_{d2} = Design Superelevation Rates - Curves 1 & 2
- L_{r1}, L_{r2} = Superelevation Transition Lengths - Curves 1 & 2
- D = Distance Between Curves
- L_3 = Total Superelevation Transition Between Spiral Curves

SUPERELEVATION AND RUNOFF LENGTHS FOR HORIZONTAL CURVES ON RURAL HIGHWAYS - Based on Max. S.E. of 0.08 ft/ft -

202-7

REFERENCE SECTION
202.4.1, 202.4.3, & 202.4.5

Dc	RADIUS	DESIGN SPEED 25 e_d	25 L_r	30 e_d	30 L_r	35 e_d	35 L_r	40 e_d	40 L_r	45 e_d	45 L_r	50 e_d	50 L_r	55 e_d	55 L_r	60 e_d	60 L_r
0°15′	22918	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--
0°30′	11459	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--
0°45′	7639	NC	--	NC	--	NC	--	NC	--	NC	--	.016	39	.019	49	.022	59
1°00′	5730	NC	--	NC	--	NC	--	NC	--	.017	38	.021	51	.025	64	.029	78
1°30′	3820	NC	--	NC	--	.017	33	.021	44	.025	56	.030	72	.035	90	.041	110
2°00′	2865	NC	--	.017	32	.022	43	.027	56	.032	72	.038	92	.045	116	.051	136
2°30′	2292	NC	--	.021	39	.026	51	.033	69	.039	87	.046	111	.053	136	.061	163
3°00′	1910	.018	31	.024	44	.031	60	.038	79	.045	100	.053	128	.060	154	.068	182
3°30′	1637	.021	37	.028	52	.035	68	.043	89	.050	111	.058	140	.066	169	.074	198
4°00′	1432	.024	42	.031	57	.039	76	.047	98	.055	123	.063	152	.071	182	.078	208
4°30′	1273	.026	45	.034	63	.042	82	.051	106	.059	131	.068	164	.075	192	.080	214
5°00′	1146	.029	50	.037	68	.046	89	.055	114	.063	140	.071	171	.078	200	△=4°45′	
5°30′	1042	.031	54	.040	73	.049	95	.058	120	.066	147	.074	178	.080	205	▲=0°32′	
6°00′	955	.033	57	.042	77	.051	99	.061	126	.070	156	.077	185	.080	205		
6°30′	881	.035	61	.045	83	.054	105	.063	131	.072	160	.079	190	△=6°00′			
7°00′	819	.037	64	.047	86	.056	109	.066	137	.074	165	.080	192	▲=0°38′			
7°30′	764	.039	67	.049	90	.058	113	.068	141	.076	169	.080	192				
8°00′	716	.041	71	.051	94	.060	116	.070	145	.078	174	△=7°30′					
8°30′	674	.042	73	.052	95	.062	120	.072	149	.079	176	▲=0°45′					
9°00′	637	.044	76	.054	99	.064	124	.074	153	.080	178						
9°30′	603	.046	79	.055	101	.066	128	.075	155	.080	178						
10°00′	573	.047	81	.057	104	.067	130	.077	159	△=9°45′							
10°30′	546	.048	83	.058	106	.069	134	.078	161	▲=0°55′							
11°00′	521	.049	85	.059	108	.070	136	.079	164								
11°30′	498	.050	86	.061	112	.071	138	.079	164								
12°00′	477	.051	88	.062	114	.073	142	.080	166								
12°30′	458	.052	90	.063	115	.074	143	.080	166								
13°00′	441	.053	91	.064	117	.075	145	△=12°45′									
13°30′	424	.054	93	.066	121	.076	147	▲= 1°08′									
14°00′	409	.055	95	.067	123	.077	149										
14°30′	395	.056	97	.068	125	.077	149										
15°00′	382	.057	98	.069	126	.078	151										
16°30′	347	.059	102	.071	130	.079	153										
18°00′	318	.062	107	.074	135	.080	155										
20°00′	286	.064	110	.076	139	△=18°15′											
22°00′	260	.067	115	.078	143	▲= 1°26′											
23°00′	249	.068	117	.079	145												
25°00′	229	.071	122	.080	146												
26°30′	216	.072	124	.080	146												
28°00′	205	.074	127	△=26°45′													
31°00′	185	.076	131	▲= 1°53′													
34°00′	169	.078	134														
36°00′	159	.079	136														
38°00′	151	.079	136														
40°00′	143	.080	138	△=42°30′													
42°00′	136	.080	138	▲= 2°35′													

NC = Normal Crown

△ = Max. Dc for the Design Speed
▲ = Max. Dc Without Superelevation

Dc	RADIUS	DESIGN SPEED 65 e_d	65 L_r	70 e_d	70 L_r	75 e_d	75 L_r
0°15′	22918	NC	--	NC	--	NC	--
0°30′	11459	.017	48	.019	57	.021	67
0°45′	7639	.025	70	.028	84	.031	98
1°00′	5730	.032	90	.036	108	.040	127
1°15′	4584	.039	110	.044	132	.049	155
1°30′	3820	.046	129	.051	153	.058	184
1°45′	3274	.052	146	.058	174	.066	209
2°00′	2865	.058	163	.065	195	.073	231
2°15′	2546	.063	177	.071	213	.078	247
2°30′	2292	.068	191	.075	225	.080	253
2°45′	2083	.072	202	.078	234	△=2°30′	
3°00′	1910	.075	210	.080	240	▲=0°23′	
3°15′	1763	.078	219	△=3°15′			
3°30′	1637	.079	221	▲=0°25′			
3°45′	1528	.080	224				
		△=3°45′					
		▲=0°28′					

e_d= Design Superelevation Rate
L_r= Min. Runoff Length, 2-Lane Highway Rotated About the Centerline, Lane Width of 12 feet

SUPERELEVATION AND RUNOFF LENGTHS FOR HORIZONTAL CURVES ON HIGH-SPEED* URBAN HIGHWAYS

- Based on a Maximum Superelevation of 0.06 ft/ft -

202-8
REFERENCE SECTION 202.4.1, 202.4.3, & 202.4.5

* 50 mph or greater

		DESIGN SPEED											
		50		55		60		65		70		75	
Dc	RADIUS	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r
0°15′	22918	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--
0°30′	11459	NC	--	NC	--	NC	--	.016	45	.018	54	.020	64
0°45′	7639	NC	--	.018	47	.021	56	.024	68	.026	78	.029	92
1°00′	5730	.020	48	.023	59	.027	72	.030	84	.033	99	.037	117
1°15′	4584	.024	58	.028	72	.032	86	.036	101	.040	120	.045	143
1°30′	3820	.028	68	.032	82	.037	99	.041	115	.046	138	.051	161
1°45′	3274	.032	77	.036	93	.041	110	.046	129	.051	153	.056	177
2°00′	2865	.035	84	.040	103	.045	120	.050	140	.055	165	.059	187
2°15′	2546	.038	92	.043	110	.048	128	.053	149	.057	171	.060	190
2°30′	2292	.040	96	.045	116	.051	136	.056	157	.059	177	△=2°15′	
2°45′	2083	.042	101	.048	123	.053	142	.058	163	.060	180	▲=0°23′	
3°00′	1910	.045	108	.050	128	.055	147	.059	165	△=2°45′			
3°30′	1637	.048	116	.054	139	.058	155	.060	168	▲=0°26′			
4°00′	1432	.052	125	.057	146	.060	160	△=3°30′					
4°30′	1273	.054	130	.059	151	△=4°15′		▲=0°29′					
5°00′	1146	.056	135	.060	154	▲=0°33′							
5°30′	1042	.058	140	.060	154								
6°00′	955	.059	142	△=5°30′									
6°30′	881	.060	144	▲=0°39′									
		△=6°45′											
		▲=0°47′											

NC = Normal Crown

△ = Max. Dc for the Design Speed
▲ = Max. Dc Without Superelevation

e_d = Design Superelevation Rate
L_r = Min. Runoff Length, 2-Lane Highway Rotated About the Centerline, Lane Width of 12 feet

See Figure 202-4 for the calculation of Lr and adjustments to Lr when more than one lane is rotated about the centerline or the lane width is other than 12 feet.

SUPERELEVATION AND RUNOFF LENGTHS FOR HORIZONTAL CURVES ON LOW-SPEED URBAN STREETS* - Based on Max. S.E. of 0.04 ft/ft

202-9

REFERENCE SECTION
202.4.1, 202.4.3, & 202.4.5

Dc	RADIUS	DESIGN SPEED 20		25		30		35		40		45	
		e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r
5°00′	1146	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--
5°30′	1042	NC	--	NC	--	NC	--	NC	--	NC	--	NC	--
6°00′	955	NC	--	NC	--	NC	--	NC	--	NC	--	RC	36
7°00′	819	NC	--	NC	--	NC	--	NC	--	NC	--	RC	36
7°30′	764	NC	--	NC	--	NC	--	NC	--	NC	--	.027	61
8°00′	716	NC	--	NC	--	NC	--	NC	--	RC	34	.039	87
9°00′	637	NC	--	NC	--	NC	--	NC	--	RC	34	△=8°00′	
9°30′	603	NC	--	NC	--	NC	--	NC	--	.017	37	▲=5°40′	
10°00′	573	NC	--	NC	--	NC	--	NC	--	.027	56		
10°30	546	NC	--	NC	--	NC	--	NC	--	.036	75		
11°00′	521	NC	--	NC	--	NC	--	NC	--	△=10°45′			
11°30′	498	NC	--	NC	--	NC	--	RC	31	▲=7°42′			
12°00′	477	NC	--	NC	--	NC	--	RC	31				
13°00′	441	NC	--	NC	--	NC	--	RC	31				
14°00′	409	NC	--	NC	--	NC	--	.020	40				
15°00′	382	NC	--	NC	--	NC	--	.035	67				
15°30′	370	NC	--	NC	--	NC	--	.040	78				
17°00′	337	NC	--	NC	--	NC	--	△=15°30′					
17°30′	327	NC	--	NC	--	NC	--	▲=11°28′					
18°00′	318	NC	--	NC	--	RC	30						
20°00′	286	NC	--	NC	--	RC	30						
20°30′	279	NC	--	NC	--	RC	30						
21°00′	273	NC	--	NC	--	.021	38						
22°00′	260	NC	--	NC	--	.031	57						
22°45′	252	NC	--	NC	--	.039	72						
25°00′	229	NC	--	NC	--	△=22°45′							
29°00′	198	NC	--	NC	--	▲=17°30′							
29°30′	194	NC	--	RC	28								
30°00′	191	NC	--	RC	28								
33°00′	174	NC	--	RC	28								
33°45′	170	NC	--	.016	28								
35°00′	164	NC	--	.025	44								
36°00′	159	NC	--	.033	57								
37°00′	155	NC	--	.040	69								
40°00′	143	NC	--	△=37°00′									
50°00′	116	NC	--	▲=29°19′									
54°30′	105	RC	26										
56°00′	102	RC	26										
58°00′	99	RC	26										
61°00′	94	RC	26										
61°15′	93	.016	26										
62°00′	92	.020	32										
63°00′	91	.024	40										
64°00′	90	.029	47										
65°00′	88	.034	55										
66°00′	87	.038	62	△=66°30′									
66°30′	86	.040	65	▲=54°23′									

* 45 mph or less

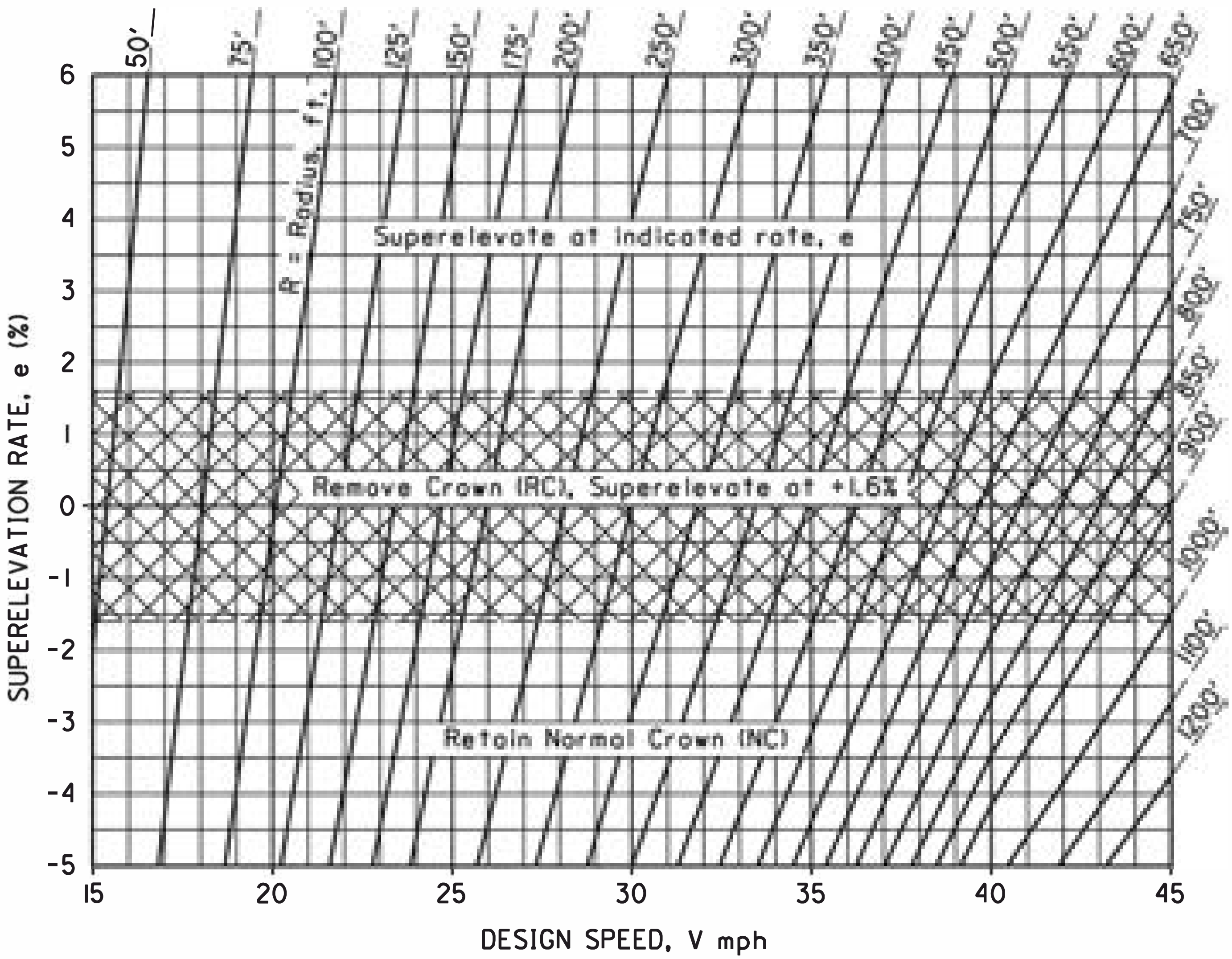
NC = Normal Crown
RC = Remove Crown

e_d= Design Superelevation Rate
L_r= Minimum Runoff Length, 12-foot Wide Lane Rotated About the Centerline

△ = Maximum Degree of Curve for the Design Speed
▲ = Maximum Degree of Curve for Normal Crown

See Figure 202-4 for the calculation of L_r and adjustments to L_r when more than one lane is rotated about the centerline or the lane width is other than 12 feet.

SUPERELEVATION RATES FOR HORIZONTAL CURVES ON LOW-SPEED URBAN STREETS	202-9a REFERENCE SECTION 202.4.1, 202.4.3, & 202.4.5



NOTES:

1. The Figure provides a range of curves and superelevation rates which apply to a selected design speed for low-speed urban streets. AASHTO Method 2 was used to distribute superelevation and side-friction.

2. For curves that fall within the shaded area, it is desirable to remove the crown and superelevate the roadway at a uniform slope of +1.6%.

3. Dc = 5729.58 / R

SUPERELEVATION AND RUNOFF LENGTHS FOR HORIZONTAL CURVES ON LOW-SPEED* URBAN RAMPS AND OTHER INTERCHANGE ROADWAYS - Based on Max. S.E. of 0.06 ft/ft -

202-10

REFERENCE SECTION
202.4.1, 202.4.3, & 202.4.5

* = 45 m.p.h. or less

		DESIGN SPEED									
		25		30		35		40		45	
Dc	RADIUS	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r	e_d	L_r
0°15′	22918	NC	--	NC	--	NC	--	NC	--	NC	--
0°30′	11459	NC	--	NC	--	NC	--	NC	--	NC	--
0°45′	7639	NC	--	NC	--	NC	--	NC	--	NC	--
1°00′	5730	NC	--	NC	--	NC	--	NC	--	.017	51
1°30′	3820	NC	--	NC	--	.016	42	.020	56	.024	72
2°00′	2865	NC	--	.016	39	.021	54	.025	69	.030	89
2°30′	2292	NC	--	.020	49	.025	65	.030	83	.035	104
3°00′	1910	.018	42	.023	56	.028	73	.034	94	.039	116
3°30′	1637	.020	46	.026	64	.032	83	.037	102	.043	128
4°00′	1432	.022	51	.028	69	.034	88	.040	111	.046	137
4°30′	1273	.024	55	.031	76	.037	96	.043	119	.049	146
5°00′	1146	.026	60	.033	81	.039	101	.045	124	.051	151
5°30′	1042	.028	65	.035	86	.041	106	.047	130	.053	157
6°00′	955	.030	69	.036	88	.042	109	.049	135	.055	163
6°30′	881	.031	71	.038	93	.044	114	.051	141	.057	169
7°00′	819	.033	76	.039	95	.045	116	.053	146	.058	172
7°30′	764	.034	78	.040	98	.047	122	.054	149	.059	175
8°00′	716	.035	81	.041	100	.048	124	.055	152	.060	178
8°30′	674	.036	83	.042	103	.050	129	.056	155	.060	178
9°00′	637	.037	85	.043	105	.051	132	.057	157	.060	178
9°30′	603	.038	87	.045	110	.052	135	.058	160		
10°00′	573	.038	87	.046	112	.053	137	.059	163		
10°30′	546	.039	90	.046	112	.054	140	.059	163		
11°00′	521	.040	92	.047	115	.055	142	.060	166		
11°30′	498	.040	92	.048	117	.056	145	.060	166		
12°00′	477	.041	94	.049	120	.057	147				
12°30′	458	.042	97	.050	122	.057	147				
13°00′	441	.042	97	.051	125	.058	150				
13°30′	424	.043	99	.052	127	.058	150				
14°00′	409	.044	101	.052	127	.059	152				
14°30′	395	.044	101	.053	129	.059	152				
15°00′	382	.045	103	.054	132	.060	155				
16°30′	347	.047	108	.055	134	.060	155				
18°00′	318	.048	110	.057	139						
20°00′	286	.050	115	.058	142						
22°00′	260	.052	119	.059	144						
23°00′	249	.053	122	.060	146						
25°00′	229	.055	126								
28°00′	205	.057	131								
31°00′	185	.058	133								
34°00′	169	.059	135								
36°00′	159	.060	138								
38°00′	151	.060	138								
39°30′	145	.060	138								

45 m.p.h.: △=9°00′ ▲=0°57′

40 m.p.h.: △=11°45′ ▲=1°10′

35 m.p.h.: △=16°45′ ▲=1°29′

30 m.p.h.: △=24°45′ ▲=1°58′

25 m.p.h.: △=39°45′ ▲=2°42′

NC = Normal Crown

e_d = Design Superelevation Rate

L_r = Minimum Runoff Length, 16-foot Wide Lane Rotated About the Edge

△ = Maximum Degree of Curve for the Design Speed

▲ = Maximum Degree of Curve Without Superelevation

See Figure 202-4 for the calculation of Lr and adjustments to Lr when more than one lane is rotated about the edge or the lane width is other than 16 feet.

# MAXIMUM CURVE WITHOUT A SPIRAL	202-11
	REFERENCE SECTION 202.5

DESIGN SPEED (mph)	MAX. DEGREE OF CURVE	MIN. RADIUS (feet)
50	4° 30’	1273
55	3° 45’	1528
60	3° 00’	1910
65	2° 30’	2292
70	2° 15’	2546
75	2° 00'	2865

MAXIMUM GRADES (PERCENT)

203-1

REFERENCE SECTION 203.2.3

	FUNCTIONAL CLASSIFICATION	TERRAIN	DESIGN SPEED (mph)										
			25	30	35	40	45	50	55	60	65	70	75
URBAN	Interstate, Other (A) Freeways and Expressways	LEVEL						4	4	3	3	3	3
		ROLLING						5	5	4	4	4	4
		HILLY						6	6	6	5	5	
	ARTERIAL STREET	LEVEL	7	7	7	7	6	6	5	5			
		ROLLING	10	9	8	8	7	7	6	6			
		HILLY	12	11	10	10	9	9	8	8			
	COLLECTOR STREETS	LEVEL	9	9	9	9	8	7	7	6			
		ROLLING	12	11	10	10	9	8	8	7			
		HILLY	13	12	12	12	11	10	10	9			
	LOCAL STREETS	LEVEL	10	9	9	9	9	8	8	7			
		ROLLING	13	12	12	11	11	10	10	8			
		HILLY	15	15	15	14	14	12	12	10			
RURAL	Interstate, Other (A) Freeways and Expressways	LEVEL						4	4	3	3	3	3
		ROLLING						5	5	4	4	4	4
		HILLY						6	6	6	5	5	
	ARTERIAL STREET	LEVEL	5	5	5	5	5	4	4	3	3	3	
		ROLLING	8	7	7	6	6	5	5	4	4	4	
		HILLY	9	8	8	8	7	7	6	6	5	5	
	COLLECTOR (B) STREETS	LEVEL	7	7	7	7	7	6	6	5			
		ROLLING	10	9	9	8	8	7	7	6			
		HILLY	11	10	10	10	10	9	9	8			
	LOCAL STREETS	LEVEL	7	7	7	7	7	6	6	5			
		ROLLING	11	10	10	10	9	8	7	6			
		HILLY	15	14	14	13	12	10	10				

(A) Grades 1% steeper may be used where development in urban areas precludes the use of flatter grades. Grades 1% steeper may also be used for one-way down-grades, except in hilly terrain.

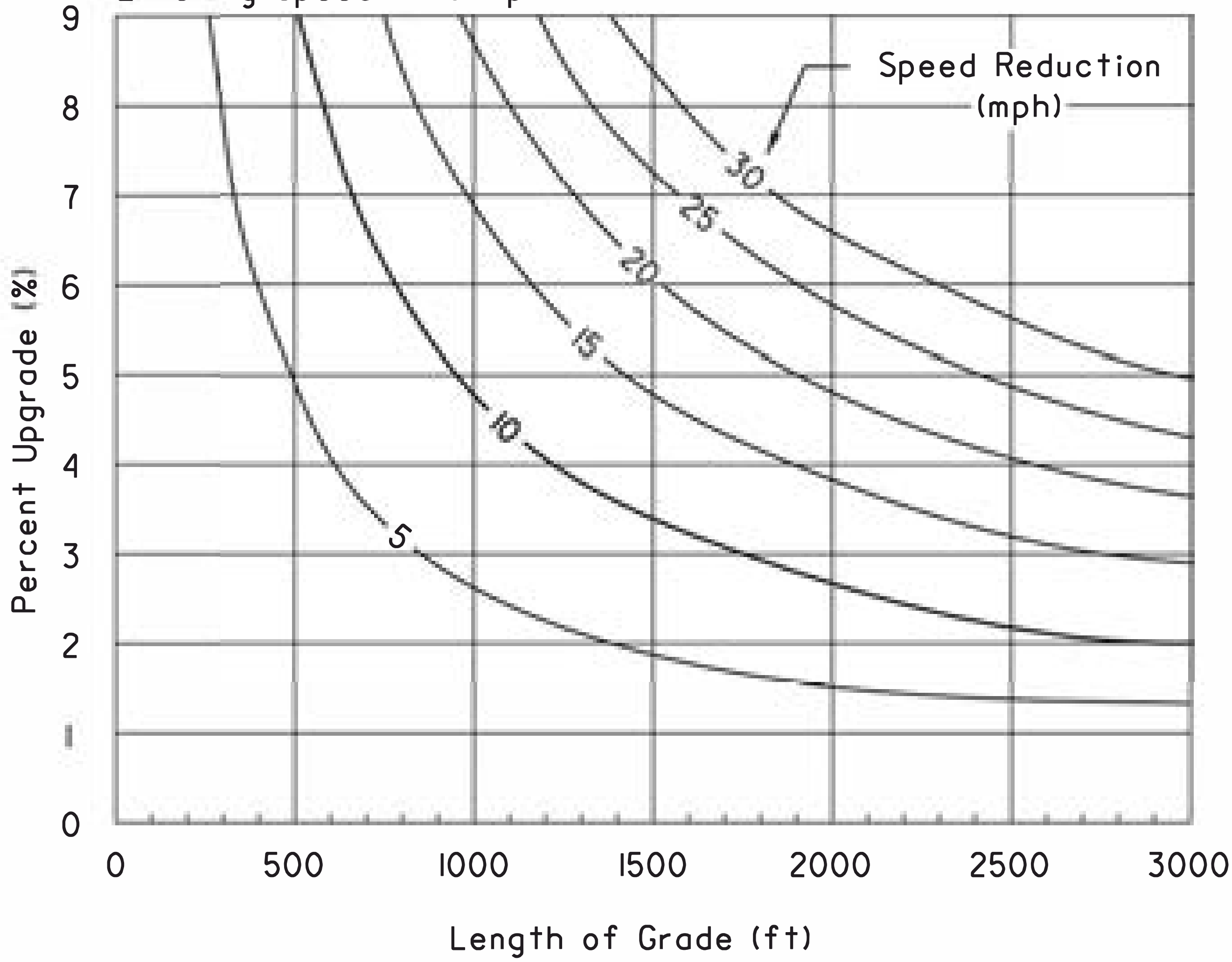
(B) Grades 2% steeper may be used for short lengths (less than 500 ft.), on one-way down-grades, and on low-volume rural collectors with current AADT less than 2,000 veh/day.

CRITICAL LENGTHS OF GRADE

203-1a

REFERENCE SECTION 203.2.3

Graph based on Typical Heavy Truck of 200 lb/hp, Entering Speed = 70 mph



The above figure can also be used to compute the critical length of grade for grade combinations. For example, find the critical length of grade for a 4% upgrade preceded by 2000 feet of 2% upgrade and a tolerable speed reduction of 15 mph. From the figure, 2000 feet of 2% upgrade results in a speed reduction of 7 mph. Subtracting 7 mph from the tolerable speed reduction of 15 mph gives the remaining tolerable speed reduction of 8 mph. The figure shows that the remaining tolerable speed reduction of 8 mph would occur on 1000 feet of the 4 % upgrade.

The critical length of grade is the length of tangent grade. When a vertical curve is part of the critical length of grade, an approximate equivalent tangent grade should be used. Where A <= 3%, then the vertical tangent lengths can be used (VPI to VPI). Where A > 3%, then about one-quarter of the vertical curve length should be used as part of the tangent grade.

MAXIMUM CHANGE IN VERTICAL ALIGNMENT WITHOUT VERTICAL CURVE

203-2

REFERENCE SECTION 203.3.2

DESIGN SPEED (mph)	MAX. GRADE CHANGE Δ
25	1.85%
30	1.30%
35	0.95%
40	0.75%
45	0.55%
50	0.45%
55	0.40%
60	0.30%
65	0.30%
70	0.25%
75	0.20%

Based on the AASHTO comfort formula for sag vertical curves:

$$A = 46.5\ L / V^2 = 1162.5 / V^2$$

Where:

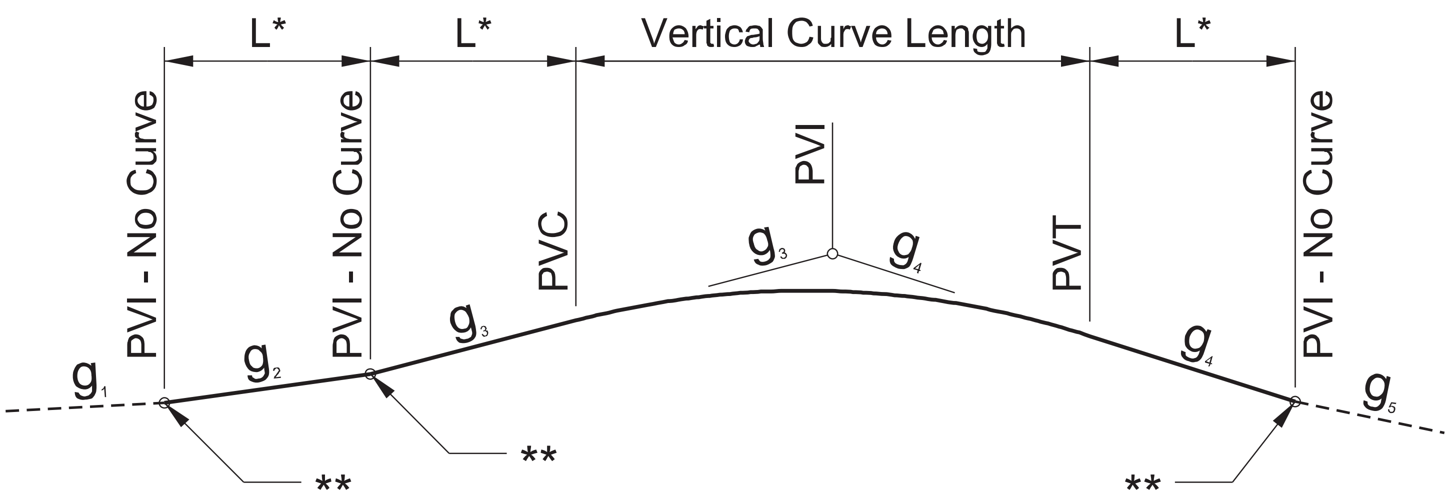
A = Maximum Grade Change (%)

L = Length of Vertical Curve (assume 25')

V = Design Speed (mph)

Δ ROUNDED TO NEAREST 0.05%

RELATIONSHIP BETWEEN VERTICAL CURVES AND GRADE BREAKS



* The minimum distance between consecutive deflections is:
100' where design speed is 50 mph or greater
50' where design speed is less than 50 mph

** Allowable grade break location.

CREST VERTICAL CURVES

HEIGHT OF EYE 3.50’ - HEIGHT OF OBJECT 2.00’

DESIGN SPEED	DESIGN SSD	DESIGN K
20	115	7
21	120	7
22	130	8
23	140	10
24	145	10
25	155	12
26	165	13
27	170	14
28	180	15
29	190	17
30	200	19
31	210	21
32	220	23
33	230	25
34	240	27
35	250	29
36	260	32
37	270	34
38	280	37
39	290	39
40	305	44
41	315	46
42	325	49
43	340	54
44	350	57
45	360	61
46	375	66
47	385	69

DESIGN SPEED	DESIGN SSD	DESIGN K
48	400	75
49	415	80
50	425	84
51	440	90
52	455	96
53	465	101
54	480	107
55	495	114
56	510	121
57	525	128
58	540	136
59	555	143
60	570	151
61	585	159
62	600	167
63	615	176
64	630	184
65	645	193
66	665	205
67	680	215
68	695	224
69	715	237
70	730	247
71	745	257
72	765	271
73	780	282
74	800	297
75	820	312

Using: S = Stopping Sight Distance, ft.
L = Length of Crest Vertical Curve, ft.
A = Algebraic Difference in Grades (%), Absolute Value
K = Rate of Vertical Curvature

- For a given design speed and an “A” value, the calculated length “L” = K x A
- To determine “S” with a given “L” and “A”, use the following:

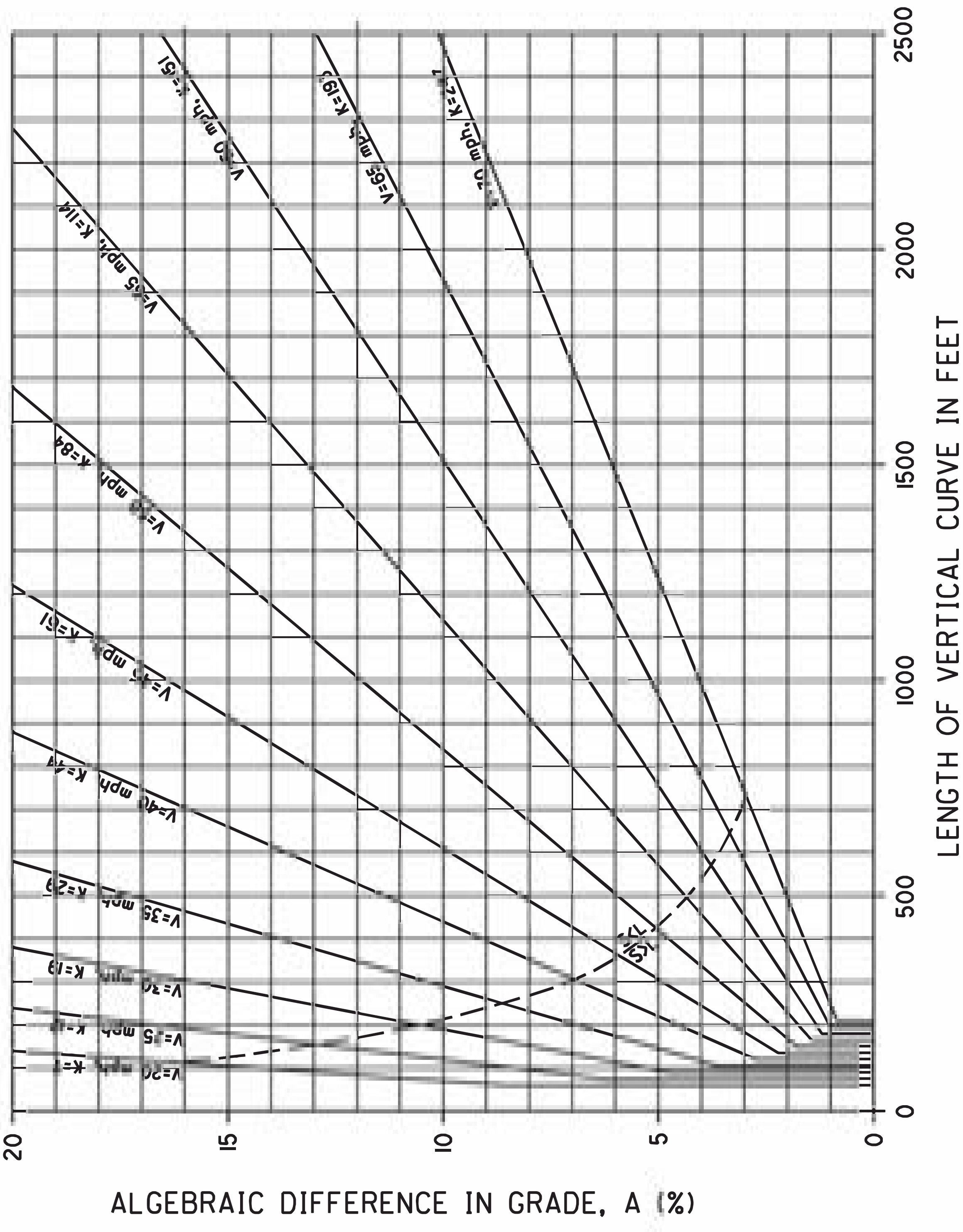
For S<L: $S = 46.45\sqrt{K}$, where K = L/A

For S>L: S = 1079/A + L/2

DESIGN CONTROLS STOPPING SIGHT DISTANCE CREST VERTICAL CURVES

203-4

REFERENCE SECTION 201.2.2 & 203.3.3



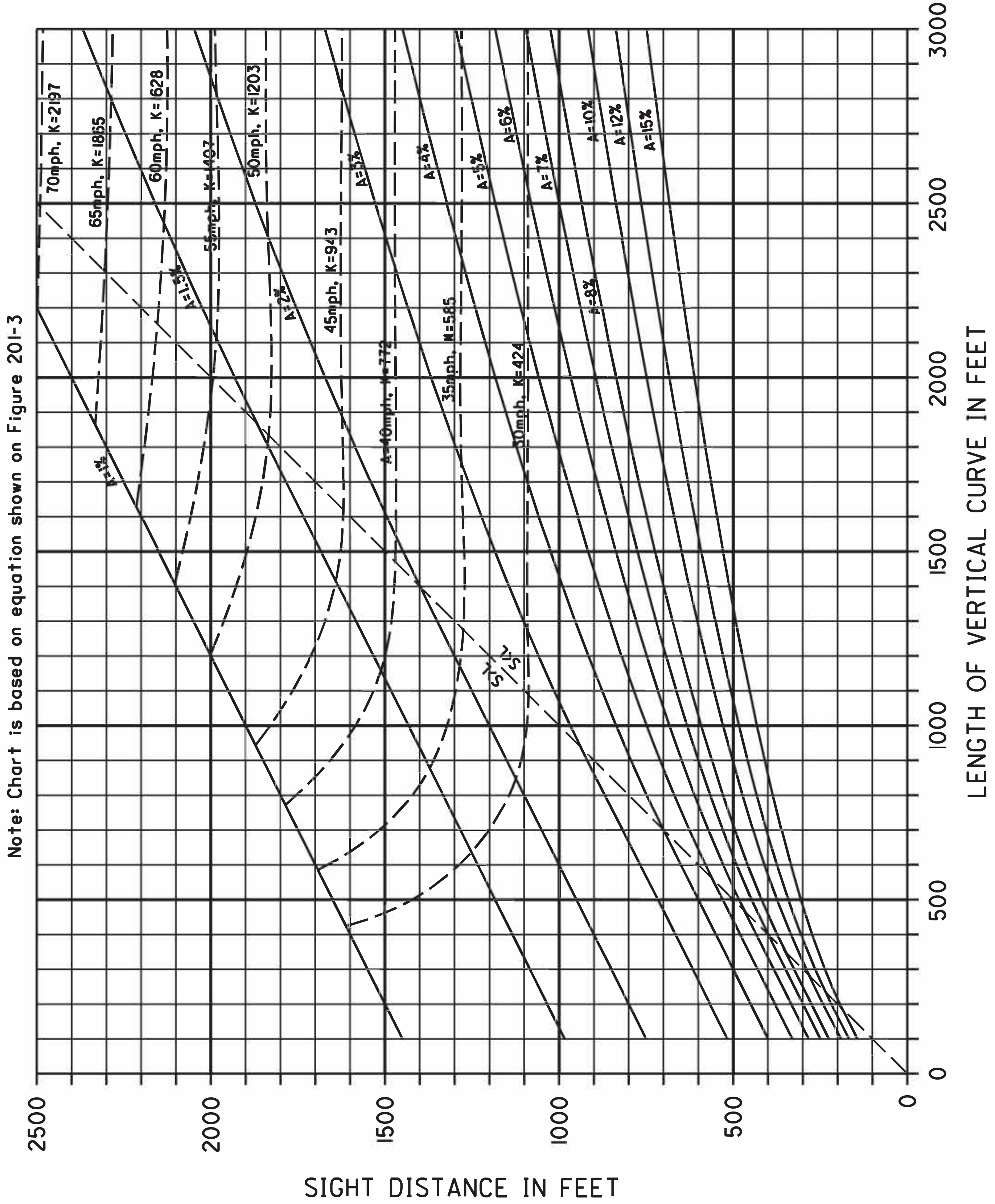
Note 1. Chart is based on equations shown on Figure 203-3 and Lmin = 3xV, where "V" is the design speed in mph and L is in feet.

2. To determine the Design Speed for a given "A" and "L" when S>L, use equation for S>L on Figure 203-3.

DESIGN CONTROLS PASSING SIGHT DISTANCE CREST VERTICAL CURVES

203-5

REFERENCE SECTION 203.3.3



SAG VERTICAL CURVES	203-6
	REFERENCE SECTION 201.2.2 & 203.3.4

HEIGHT OF HEADLIGHT = 2.00'

UPWARD LIGHT BEAM DIVERGENCE = 1°00'

DESIGN SPEED	DESIGN SSD	DESIGN K
20	115	17
21	120	18
22	130	20
23	140	22
24	145	24
25	155	26
26	165	28
27	170	29
28	180	32
29	190	34
30	200	37
31	210	39
32	220	42
33	230	44
34	240	47
35	250	49
36	260	52
37	270	55
38	280	57
39	290	60
40	305	64
41	315	66
42	325	69
43	340	73
44	350	76
45	360	79
46	375	83
47	385	85

DESIGN SPEED	DESIGN SSD	DESIGN K
48	400	89
49	415	93
50	425	96
51	440	100
52	455	104
53	465	107
54	480	111
55	495	115
56	510	119
57	525	123
58	540	128
59	555	132
60	570	136
61	585	140
62	600	144
63	615	148
64	630	153
65	645	157
66	665	162
67	680	167
68	695	171
69	715	176
70	730	181
71	745	185
72	765	190
73	780	194
74	800	200
75	820	206

Using: S = Stopping Sight Distance
L = Length of Sag Vertical Curve
A = Algebraic Difference in Grades (%), Absolute Value
K = Rate of Vertical Curvature

- For a given design speed and an "A" value, the calculated length "L"= K x A
- To determine "S" with a given "L" and "A", use the following:

For S<L: $S = \frac{3.5L + \sqrt{12.25L^2 + 1600AL}}{2A}$

For S>L: $S = (AL + 400)/(2A - 3.5)$

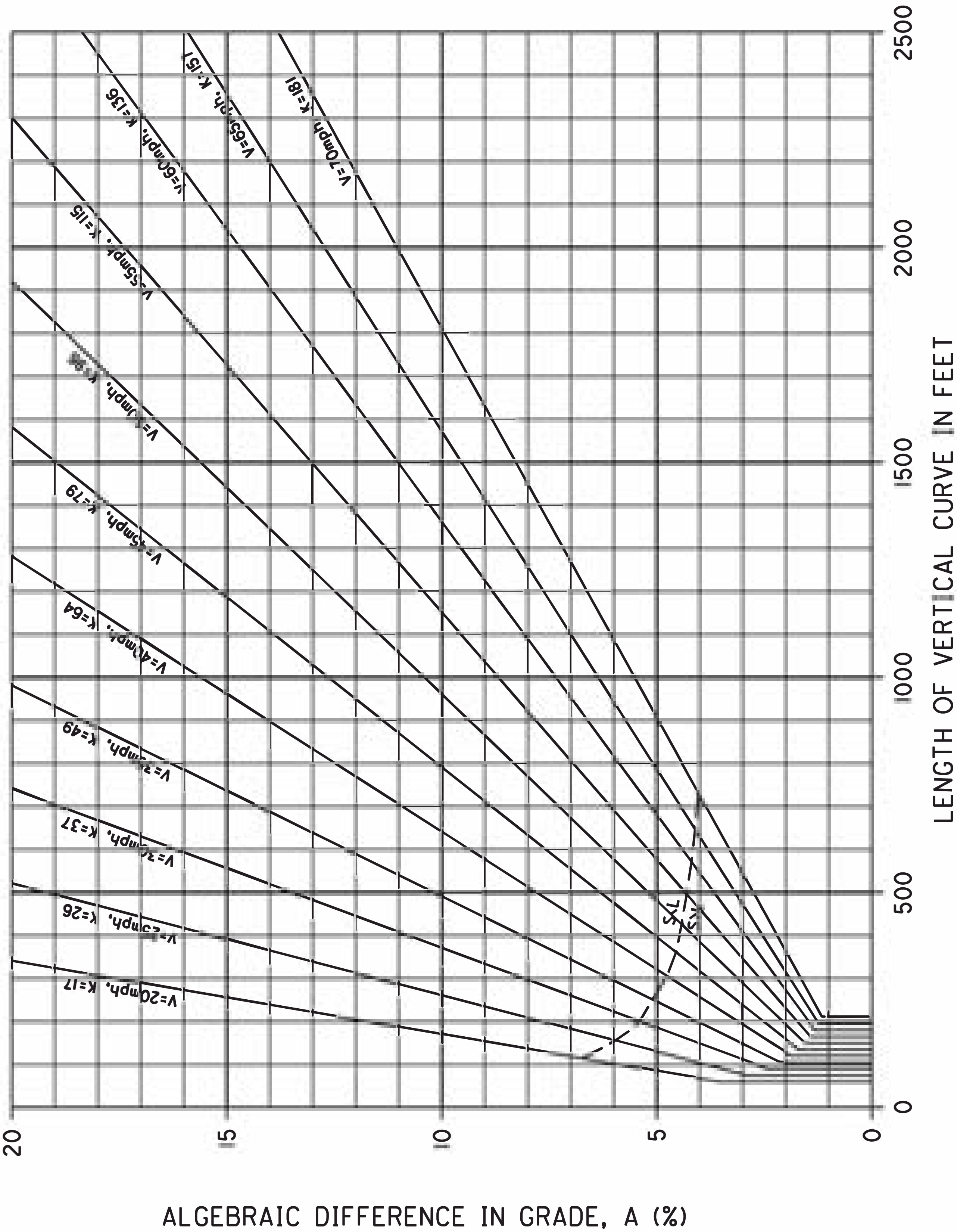
Note: When the Algebraic difference, A, is 1.75% or less, SSD is not restricted by the vertical curve.

DESIGN CONTROLS SSD DISTANCE SAG VERTICAL CURVES

203-7

REFERENCE SECTION

201.2.2 & 203.3.4

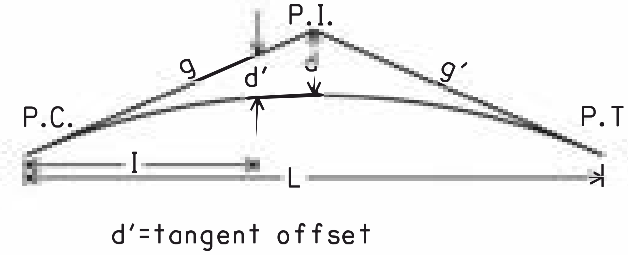


Note: Chart is based on equations shown on Figure 203-6 and Lmin = 3xV, where V is the design speed in mph and L is in feet.

TANGENT OFFSETS FOR VERTICAL CURVES

Basic formulae are: $d = \frac{L}{800}(g-g')$

and: $d' = \left[\frac{I}{\left(\frac{L}{2}\right)}\right]^2 \times d$



d'=tangent offset

To find offset from tanget for any 25' interval from PC or PT, use factor from table for I and multiply by g-g'

(Example) Given; L=1200'
I=225'
g=+2%
g'=-4%
g-g'= 6

Then the factor for 225' (L=1200'), is 0.2109 and offset = 0.2109 x 6 or d'=1.265'

LENGTH OF VERTICAL CURVE IN FEET	L/800																														
200	2500	.0156	.0625	.1406	.2500																										
250	.3125	.0125	.0500	.1125	.2000	.3125																									
300	3750	.0104	.0417	.0938	.1667	.2604	.3750																								
350	4375	.0089	.0357	.0803	.1429	.2232	.3214	.4375																							
400	5000	.0078	.0313	.0703	.1250	.1953	.2812	.3828	.5000																						
450	5625	.0069	.0278	.0625	.1111	.1736	.2500	.3403	.4444	.5625																					
500	6250	.0063	.0250	.0563	.1000	.1563	.2250	.3063	.4000	.5063	.6250																				
600	7500	.0052	.0208	.0469	.0833	.1302	.1875	.2552	.3333	.4219	.5208	.6302	.7500																		
700	8750	.0045	.0719	.0402	.0714	.1116	.1607	.2188	.2857	.3616	.4464	.5402	.6429	.7545	.8750																
800	1.000	.0039	.0516	.0352	.0625	.0977	.1406	.1914	.2500	.3164	.3906	.4727	.5625	.6602	.7656	.8789	1.0000														
900	1.125	.0035	.0139	.0313	0556	0868	.1250	.1701	.2222	.2813	.3472	.4201	.5000	.5868	.6806	.7813	.8889	1.0035	1.1250												
1000	1.250	.0031	.0125	.0281	0500	.0781	.1125	.1531	.2000	.2531	.3125	.3781	.4500	.5281	.6125	.7031	.8000	.9031	1.0125	1.1281	1.2500										
1100	1.375	.0028	.0114	.0256	.0455	.0710	.1023	.1392	.1818	.2301	.2841	.3438	.4091	.4801	.5568	.6392	.7273	.8210	.9205	1.0256	1.1365	1.2528	1.3750								
1200	1.500	.0026	.0104	.0234	.0417	.0651	.0938	.1276	.667	.2109	.2604	.3151	.3750	.4401	.5104	.5859	.6667	.7526	.8438	.9401	1.0417	1.1484	1.2604	1.3776	1.5000						
1300	1.625	.0024	.0096	.0216	.0385	.0601	.0865	.1178	.1538	.1947	.2404	.2909	.3462	.4063	.4712	.5409	.6154	.6947	.7788	.8678	.9615	1.0601	1.1635	1.2716	1.3846	1.5024	1.6250				
1400	1.750	.0022	.0089	.0201	.0357	.0558	.0804	.1094	.1429	.1808	.2232	.2701	.3214	.3772	.4375	.5022	.5714	.6451	.7232	.8058	.8929	.9844	1.0804	1.1808	1.2857	1.3951	1.5089	1.6272	1.7500		
1500	1.875	.0021	.0083	.0188	.0333	.0521	.0750	.1021	.1333	.1688	.2083	.2521	.3000	.3521	.4083	.4688	.5333	.6021	.6750	.7521	.8333	.9188	1.0083	1.1021	1.2000	1.3021	1.4083	1.5188	1.6333	1.7521	1.8750
	I=	25	50	75	100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	500	525	550	575	600	625	650	675	700	725	750

INTERMEDIATE DISTANCES FROM P.C. OR P.T. IN FEET

TABLE OF CONTENTS

300 Cross Section Design

301 ROADWAY CRITERIA

301.1 Pavement

301.1.1 General

This section will assist the designer in determining lane width and pavement cross slope. The number of lanes required is determined through the use of a capacity analysis. This process is explained in detail in the current Highway Capacity Manual, published by the Transportation Research Board. ***Figure 301-1*** should be used as a guide for selection of design levels of service.

Pavement type is determined by the volume and composition of traffic, soil characteristics, performance of pavements in the area, availability of materials, initial cost, annual maintenance cost and service life cost. The determination of pavement type and structural design is included in the Pavement Design & Rehabilitation Manual published by the Office of Pavement Engineering.

301.1.1.1 Disposition of Pavement Required Due to Maintenance of Traffic

ODOT Policy 21-008(P) (and related Standard Procedure 123-001(SP)), Traffic Management in Work Zones, establishes criteria intended to eliminate or reduce traffic delays caused by work zones. Application of this policy to major rehabilitation projects typically results in the need for additional pavement to satisfy the policy. In some situations, this has caused debate on whether the additional pavement should be permanent, full depth design that would be opened to traffic upon completion of the project or a temporary, thinner design that would be removed after construction. The following is intended to provide guidance in making these decisions. This guidance should be considered during the earliest steps of the Project Development Process, and should not supersede any planning or PDP requirements.

The Districts should first request 20 year design traffic for the project and run capacity analyses to determine the need for future permanent lanes. According to ***Figure 301-1***, the goal is to achieve level-of-service C or better depending on the terrain and locale. Typically, level-of-service C is considered satisfactory for all roadways due to budgetary constraints. However, this guideline has been slightly modified in urbanized areas to permit level-of-service D, if approved by the MPO. If the analyses indicate additional lanes will be needed within the next 20 years, the District should proceed with the environmental documentation required, including a Major Investment Study (MIS) in Metropolitan Planning Organization (MPO) areas. This may result in changing the original project classification from a minor project (major rehabilitation with no additional lanes) to a major project. If stream coordination, noise impacts, air quality and planning requirements can be addressed satisfactorily, FHWA will support further development of a project which includes additional permanent, full-depth pavement. Gap closure will not be accepted as the principal Purpose and Need for adding permanent lanes in the future if the capacity analysis does not indicate a need. Median treatments must also be analyzed to determine barrier requirements.

Projects which are approved for additional pavement that will be opened to traffic upon completion must be submitted to TRAC for their concurrence even if Major New funds are not being requested.

Due to the 1 mile signing requirements on freeways to warn motorists of a lane drop ahead, the additional lane/s which were permanently added should not be opened to traffic unless their total length is 5 miles or greater if the adjoining pavement sections at either end of the project have not been widened to match.

Where the need for additional lanes has been determined, the Districts should also look at the need for modifying any existing interchanges which are in the project boundaries of the additional through lanes. Since approval for Interchange Justification Studies are based on not degrading the level-of-service of the Interstate or freeways from the no-build alternative to the build alternative, adding additional capacity to the roadway almost always permits existing interchanges to be expanded to handle additional traffic. Even if funding is not readily available for modifying interchanges at the time the additional through lanes are being added to the freeway, the District should still check the capacity need for modifying the existing interchanges. Many times auxiliary lanes are needed on the freeway from one interchange to another, and these lanes, along with bridge widening to accommodate these lanes could be performed at the same time the additional lanes are constructed.

If Districts determine additional capacity will not be needed within the next 20 years, the additional widening required to maintain traffic should be a temporary buildup sufficient for the duration of the construction project and then be removed upon completion of the project. It is not cost effective to construct full-depth pavement and open it to traffic at the conclusion of the project if the capacity is not required within a 20-year planning horizon.

301.1.2 Lane Width

Lane width in rural areas is dependent upon functional classification, traffic volumes and design speed and is shown in ***Figure 301-2***. ***Figure 301-4*** shows lane widths in urban areas based on functional classification and locale. See ***Section 105.3*** for the National Network lane width requirements.

301.1.3 Traveled Way Widening on Highway Curves

Additional widening may be necessary on curves depending on the design speed, curvature and traveled way width. The Traveled Way Widening values ***Figure 301-5c*** are based on the WB-62 [WB-19] vehicle and are applicable to either one-way or two-way, two-lane traveled ways, and other similar type facilities. A WB-62 [WB-19] design vehicle is to be used on state maintained roadways. The design vehicle for other than state maintained roadways should be determined by the maintaining authority. Note that widening less than 2.0 ft. is not required.

Curve widening should be placed on the inside edge of the curve. Where spirals are used, the widening should begin at the TS and reach maximum width at the SC. On alignments without spirals, the widening should be developed over the same distance as the superelevation transition. See ***Section 202.4*** and ***Figure 301-5a***. The transition ends should be rounded to avoid an angular break at the traveled way edge and intermediate points should be widened proportionately. The longitudinal center joint and the centerline marking should be placed equidistant from the traveled way edges.

301.1.4 Pavement Transition/ Taper Rates

Where traveled way widths decrease, the length of transition should be calculated using the following:

Design Speed of 50 mph or more:

$L=WS$

Design Speed of less than 50 mph:

$L=WS^2/60$

Where: L = Taper length in feet
W = Offset width in feet
S = Design speed

The transition length for increases in traveled way width (diverging tapers) may be more abrupt. The diverging taper for turn lanes should be 50’. Ramps should have a 100’ taper length.

301.1.5 Pavement Cross Slope

Normal crowned pavements in Ohio are sloped at the rate of 0.016. There are occasions when, because of drainage or pavement type, this rate may be increased to 0.02. An increase in the 0.016 slope rate normally takes place on facilities maintained by local governmental agencies and usually at design speeds less than 50 mph.

Cross slope arrangements for normal crowned sections vary based on features such as the number of lanes, whether or not the highway is divided or undivided, the type and width of the median, and drainage. ***Figure 301-6*** shows examples normally used in Ohio. Generally the following are applicable on normal crowned pavements:

1. Crowns are to be located between lanes.
2. For three or four lane roadways, no more than two lanes should slope in the same direction.
3. When 3 or more lanes are sloped in the same direction on a high speed roadway (50 mph or greater), the first two lanes from the crown point should have the normal cross slope of 0.016 and any adjacent outside lanes may have an increased maximum cross slope of 0.02.
4. Undivided pavement sections are to be crowned at the middle when the number of lanes are even and at the edge of the center lane when there is an odd number of lanes. When possible, the majority of the pavement should slope to the side which will best accommodate the drainage.
5. Narrow raised median sections are crowned such that the majority of the pavement will drain toward the outside.
6. Pavement sections on either side of wide, depressed medians are to be treated similar to undivided pavement sections (See Item 3 above), with the majority of the pavement sloped to the outside.

Special conditions on individual projects may result in deviations from the above and from those examples shown in ***Figure 301-6***.

301.2 Shoulders

301.2.1 General

Shoulders are used to provide an area adjacent to the pavement to accommodate stopped vehicles, for emergency use, for use while maintaining traffic through construction work zones, for the lateral support of the pavement and to generally improve the safety of a highway. They are also available for the use of pedestrians and bicyclists. When discussing shoulders in this manual, the following meanings are applicable. (See ***Figure 301-7***.)

Traveled Way - The portion of roadway used for the movement of vehicles, exclusive of shoulders and bicycle lanes.

Graded Shoulder Width - The width measured from the edge of the traveled way to the intersection of the shoulder slope and foreslope.

Treated Shoulder Width - The width of that portion of the graded shoulder improved with stabilized aggregate or better.

301.2.2 Shoulder Type

Four basic types of shoulders are used. These include paved, bituminous surface treated, stabilized aggregate, and turf. Structural design of shoulders and shoulder typical sections are covered in the Pavement Design & Rehabilitation Manual published by the Office of Pavement Engineering. ***Figures 301-3*** and ***301-4*** show the type shoulder to use based on functional classification and traffic or locale.

301.2.3 Shoulder Width

Graded and treated shoulder widths vary depending on functional classification and traffic or locale. The criteria for graded and treated shoulder widths are shown in ***Figures 301-3 and 301-4***. Consideration should be given to providing paved shoulders of sufficient width and strength to accommodate temporary traffic on Interstates, other freeways and expressways. Paved shoulder width reductions of less than 2’ at sign or luminaire foundations or bridge piers will not require a design exception. The 4’ minimum lateral clearance must still be provided.

301.2.3.1 Right Turn Lane Shoulder Width

Under normal roadway conditions, it is desirable to maintain the required mainline shoulder width throughout the length of the right turn lane. Where the roadway has constrained R/W limits and a low volume truck traffic, the width of the shoulder adjacent to the turn lane may be reduced, but to no less than 4 ft. paved and 6 ft. graded. The normal mainline shoulder width should still be maintained in advance of the diverging taper for the turn lane. The transition between the mainline shoulder width and the reduced shoulder width should take place during the span of the right turn taper. The reduced shoulder width may then be carried out throughout the length of the right turn lane. It should be noted that any shoulders or auxiliary lanes (i.e., right turn lanes) are considered part of the mainline clear zone.

301.2.3.2 Shoulder Taper Rate

A 25:1 taper should be used to transition to a reduced shoulder width. The transition length for increases in shoulder width (diverging tapers) may be more abrupt, i.e. 5:1 ratio.

301.2.4 Shoulder Cross Slope

Figures 301-8, 301-9 and 301-10 show cross slopes to be used depending on the shoulder type and pavement cross slope.

If the bridge shoulder cross slope is different than the approach roadway shoulder cross slope, then the shoulder cross slope shall be transitioned on the roadway section, off the bridge and approach slabs, within a distance of 100 feet.

301.2.5 Lateral Clearance

In general, roadside objects and barriers should be placed as far away from the traveled way as conditions permit. Proper lateral placement enhances a driver's comfort level of the roadway, allows for a greater chance of recovery for errant vehicles, and provides for improved sight distance.

The distance from the edge of the traveled way, beyond which a roadside object will not be perceived as an obstacle and result in a motorist reducing speed or changing vehicle position on the roadway is called the shy line offset. As a minimum, the designer should provide a shy line offset of at least 4 ft. When an obstacle is placed too closely to the traveled way, it may interfere with the sight distance of the roadway.

302 BRIDGE CRITERIA

302.1 General

This section provides overall physical bridge dimensions such as width, lateral clearance at underpasses and vertical clearance over roadways. This information is given for New and Reconstructed Bridges in ***Figure 302-1*** and for Existing Interstate and Other Freeway Bridges to Remain in ***Figure 302-2***. Similar information for existing non-freeway bridges that are to be left in place and not reconstructed is shown in ***Figure 302-3***. For additional design information, including Minimum Design Loading, refer to the Bridge Design Manual, published by the Office of Structural Engineering.

303 INTERCHANGE ELEMENTS

303.1 General

An interchange is a system of interconnecting roadways, with one or more grade separations, used to efficiently manage traffic between different types and levels of highways. Interchanges are composed of various elements such as Acceleration-Deceleration Lanes, One and Two-lane Directional Roadways, and Ramps. ***Figure 303-1*** shows information relating to the design of interchange elements including pavement and shoulder dimensions, as well as medians between adjacent ramps.

304 MEDIANS

304.1 General

A median is the portion of the highway separating opposing directions of the traveled way. Medians are highly desirable elements on all streets or roads with four or more lanes. This is especially true on rural arterials. All rural arterials, on new locations requiring four or more lanes, should be designed with a median.

The principal functions of a median are to prevent interference of opposing traffic, to provide a recovery area for out-of-control vehicles, to provide areas for emergency stopping and left turn lanes, to minimize headlight glare and to provide width for future lanes. A median should be highly visible both day and night and in definite contrast to the roadway.

304.2 Width

The width of a median is the distance between the inside edges of the traveled way. See ***Figure 304-1***. Width depends upon the type of facility, cost, topography and right-of-way.

304.2.1 Rural

In flat or rolling terrain, the desirable median width for rural freeways is 60 to 84 ft. The 84 foot wide median allows for a future 12 foot wide lane in each direction of travel, and the 60 ft. median. The minimum median width is normally 40 ft. However, in rugged terrain, narrower medians ranging from 10 to 30 ft. may be used. A constant width median is not necessary and independent profiles may be used for the two roadways. For narrower medians, see ***Section 601.2*** for Median Barrier warrants.

304.2.2 Urban

Barrier medians are normally used in urban areas. The median width is dependent upon the width of the barrier and the shoulder width required in ***Figure 301-4***. The minimum median width for a four-lane urban freeway should be 10 ft. which provides for two 4 ft shoulders and a 2 ft median barrier. For freeways with six or more lanes, the minimum width should be 22 ft. The minimum median widths noted above do not take into account the extra width required if median piers are encountered. Where median piers are encountered either widen the median throughout or apply for a design exception. Preferably, use a 26 ft. wide median when the DDHV for truck traffic exceeds 250 vehicles per hour to provide a wider median shoulder to accommodate a truck.

304.3 Types

Medians are divided into types depending upon width and treatment of the median area and drainage arrangement. In general, raised or barrier medians are applicable to urban areas, while wide, depressed medians apply to rural areas. See ***Figure 304-1***.

304.3.1 Rural

Medians in rural areas are normally depressed to a swale in the center and constructed without curbs.

304.3.2 Urban

There are various types of medians applicable to urban areas. The type selected depends upon the traffic volume, speed, degree of access and available right-of-way.

On major streets with numerous business drives, a median consisting of an additional lane, striped as a continuous two-way left turn lane is desirable.

The solid 6-inch high concrete median, at a minimum width of 4ft. (See **Standard Construction Drawing RM-3.1**) may be used where the design speed is less than 50 mph and where an all-paved section is appropriate and a wider median cannot be justified. Barrier medians, described in ***Section 601.2***, are normally recommended for urban facilities where the design speed is 50 mph or greater. However, care must be exercised when barrier medians are used on expressways with unsignalized at-grade intersections because of sight distance limitations.

304.4 U-turn Median Openings

304.4.1 Purpose

U-turn median openings may be provided on expressways, freeways or interstate highways with non-barrier medians where space permits as outlined below and when needed for proper operation of police and emergency vehicles, as well as equipment engaged in physical maintenance, traffic service, and snow and ice control.

304.4.2 Location

U-turn crossings should not be constructed in barrier-type medians.

When U-turn median openings are required, they should be spaced as close to 3-mile intervals as possible.

Crossings should be located at points approximately 1,000 ft. beyond the end of each interchange speed change lane. Additional crossings may be constructed at maintenance borders, District borders, State lines and other desired locations in accordance with the 3-mile spacing interval requirement. Examples of the allowable number of crossings between interchanges, in addition to crossings provided at interchange speed change lanes, are shown below:

Interchange Spacing	Number of Crossings
3 miles or less	None
3 to 6 miles	One
6 to 9 miles	Two
9 to 12 miles	Three

U-turn median crossings should be located to fit the median drainage pattern. Each should be placed either immediately downstream from a catch basin or on a crest. They should be located so that visibility is not restricted by structures, vertical curves or horizontal curves.

304.4.3 Design Details

Median crossings should be constructed as shown on ***Figure 304-2*** which indicates geometric features applicable to the design of crossings located in medians of widths ranging from 40 to 84 ft. Tapers should be 200 ft. in length for all median widths. The profile grade line should normally be an extension of the cross slope of the shoulder paving, rounded at the lowest point.

305 CURBS

305.1 General

The type of curb and its location affect driver behavior patterns which, in turn, affect the safety and utility of a road or street. Curbs, or curbs and gutters, are used mainly in low speed urban areas (See ***Section 305.3***). Following are various reasons for justifying the use of curbs, or curbs and gutters:

1. Where required for drainage.
2. Where needed for channelization, delineation, control of access or other means of improving traffic flow and safety.
3. To control parking where applicable.
4. To reduce right-of-way requirements.

Conventional concrete or bituminous curbs offer little visible contrast to the pavement surface, particularly during fog or at night when the surface is wet. The visibility of the curbs can be greatly enhanced with the use of reflectorized paints. Curb markings should be placed in accordance with OMUTCD.

305.2 Types and Uses

There are two general types of curbs; vertical curbs and sloped curbs. Vertical curbs are relatively high (6 inches or more) and steep-faced. Sloped curbs are 6 inches or less in height and have flatter, sloping faces so that vehicles can cross them with varying degrees of ease.

The curb sections detailed on **Standard Construction Drawing BP-5.1** are approved types to be used as stated below:

Type 1 Curb (asphalt curb) is a sloping 6 inch curb used mostly for temporary situations, such as correcting special drainage problems.

Type 2, 2-A, and 2-B curbs are 6 inches high with a steep sloped face. They are widely used along the edges of traveled way in urban areas where design speeds are less than 50 mph. Type 2 curb is preferred to Type 6 curb to eliminate the joint between the curb and the gutter.

Types 3, 3-A, 3-B and Type 4, 4-A, 4-B and 4-C curbs are 4 inches high with a sloped face. They are used for channelizing islands and occasionally along medians and edges of traveled way. Type 3 is preferred for channelizing islands with the gutter sloped at the same rate as the adjacent pavement.
Type 6 Curb is a 6 inch high steep faced vertical curb. It is used in situations similar to Type 2 described above.

Type 7 Curb is a vertical type used in low speed areas (design speed of less than 50 mph) for protection at bridge approaches. It may also be used to control traffic in areas involving heavy trucks.

Type 9 Curb is a sloping 3 inch high curb used around the truck apron of a roundabout.

305.3 Position of Curb

305.3.1 Urban Areas (Design Speed less than 50 mph)

Curbs are normally used at the edge of traveled way on urban streets where the design speed is less than 50 mph. Curbs at the edge of traveled way have an effect on the lateral placement of moving vehicles. Drivers tend to shy away from them. Therefore, all curbs should be offset at least 1 foot and preferably 2 ft. from the edge of the traffic lane. Where curb and gutter is used, the standard gutter width is 2 ft.

305.3.2 Urban and Rural High Speed Areas

On roads where the design speed is 50 mph or greater, the use of curb should be avoided. Curbs should only be used in special cases. Special cases may include, but are not limited to, the use of curb to control surface drainage or to reduce right-of-way requirements in restricted areas. When it is necessary to use curbs on roads where the design speed is 50 mph or greater, they should not be closer to the traffic than 4 ft. or the edge of the treated shoulder, whichever is greater and their height should not exceed 4 inches.

305.3.3 Curb/Guardrail Relationship

Refer to ***Section 602.1.5***.

305.4 Curb Transitions

305.4.1 Curb Vertical Height Tapers

The approach and trailing ends of curb and raised medians should be tapered from the curb height to 0 in 10 ft.

305.4.2 Curbed to Uncurbed Transitions

When an urban type section with curbs at the edge of traveled way changes to a rural type section without curbs, the curb should be transitioned laterally at a 4:1 (longitudinal: lateral) rate to the outside edge of the treated shoulder or 3 ft., whichever is greater. See ***Figure 401-4b***, Option 2.

305.4.3 Curbed Approach to Uncurbed Mainline

When a curbed side road intersects a mainline that is not curbed, the curb should be terminated no closer to the mainline edge of traveled way than 8 ft. or the edge of the treated shoulder, whichever is greater. See ***Figure 401-4a***.

306 PEDESTRIAN FACILITIES

306.1 General

When pedestrians' facilities are to be constructed or reconstructed as part of a project, the facilities shall be designed to accommodate persons with disabilities. The pedestrian environment must be designed to accommodate the needs of all users, some of whom have a broad range of mobility, physical and cognitive skills.

Additional guidance in the design of pedestrian facilities may be found in The Access Board's Accessible Rights-of-Way - A Design Guide, and FHWA's Designing Sidewalks and Trails for Access, Part 2, Best Practices Design Guide and other publications.

306.2 Sidewalk Design

306.2.1 Sidewalk and Shoulder Installation

Sidewalks are the principal improvements used to accommodate pedestrians, but it is recognized that wide shoulders and unpaved walkable space may be acceptable in some instances. ***Figure 306-1*** provides a detailed listing of the recommended guidelines for the various roadway classifications for sidewalks/walkways.

While ***Figure 306-1*** recommends when and where to install sidewalks, sidewalks should be considered on projects with curb-and-gutter installations and in areas where there is obvious pedestrian use (such as worn footpaths).

While no sidewalk requirements are specifically recommended for certain rural roadways, some residential areas should have a pedestrian connection to the rest of the rural community. A paved or unpaved shoulder should be provided as a minimum where it is impractical to provide a sidewalk along a paved rural road.

306.2.2 Sidewalk Widths

Minimum and desirable sidewalk widths are shown in ***Figure 306-2***. The minimum recommended width is 5 ft. Under limited conditions, a 4 ft. sidewalk width can tolerated, although this width does not provide adequate clearance room or mobility for pedestrians passing in opposite directions. A 4 ft. width can be accepted if there are 5 ft. wide by at least 5 ft. long passing sections at least every 200 ft.

306.2.3 Obstacles and Protruding Objects

The sidewalk widths shown in ***Figure 306-2*** represent a clear or unobstructed pedestrian travel way. Still, be aware of the three dimensional corridor which makes up an accessible route and attempt to locate utilities, light poles, signs, fire hydrants, mail boxes, parking meters and street furniture (benches, shelters, bike racks, etc.) out of this sidewalk corridor. If unable to avoid keeping objects out of this space, then certain dimensional requirements must be maintained. See FHWA’s Designing Sidewalks and Trails for Access, Part 2, Best Practices Design Guide, Section 4.1.3, for information.

Placement of utility covers, gratings and other covers should be off of the sidewalk to the maximum extent feasible.

306.2.4 Buffer Widths

A buffer width, also known as a tree lawn or planting strip, is the distance between the sidewalk and the adjacent roadway. Providing a buffer can improve pedestrian safety. The buffer width in a commercial area will be different than the buffer needs of a residential area. Buffer widths as measured from the face of curb are shown in ***Figure 306-2***.

On-street parking or bike lanes can also act as a sidewalk buffer. In areas where there is no on-street parking or bike lane, the ideal width of a buffer is 6 ft.

If a buffer cannot be provided, then the curb-attached sidewalk width should be at least 7 ft. wide in residential areas. In commercial areas or along busy arterial streets, the minimum curb-attached sidewalk width should be 8 ft. to provide space for light poles and other street furniture.
All roadways with curb attached sidewalks or buffers should be constructed with vertical curbing.

306.2.5 Grade and Cross Slope

Wherever possible, sidewalks and walkways should be designed with maximum grades of 5 percent. When the topography of an area leaves no other choice than to be use a steeper grade, **Table 306-1** provides a series of specific recommendations for each situation. The only exception to the recommendations is when the adjacent road grade is steeper than 5 percent and there is no other alternative alignment for the sidewalk.

Sidewalks should be constructed with a maximum cross slope of 2 percent. The cross slope is the slope that is measured perpendicular to the direction of travel. A driveway crossing should maintain a level pedestrian zone (see ***Figure 803-3*** for sidewalk design at drives).

Table 306-1 SIDEWALK GRADE RECOMMENDATIONS	
Public Right of Way	
Maximum Sidewalk Grade Adjacent to Roadway	No limit if it follows the grade of the street
Roadway	
Maximum Cross slope	0.02
Accessible Routes Not Adjacent to Roadway	
Max. Allowable Running Grade w/o Railings	0.05
Max. Ramp Grade w/ handrails and Landings	0.083
Landing Spacing	
Landing intervals for Accessible Routes	If the slope of a ramp is between 1:12 and 1:16, the max. rise shall be 30 inches and the max. run shall be 30 ft. If the slope of the ramp is between 1:16 and 1:20, the max. rise shall be 30 inches and the max. run shall be 40 ft.
Landing Dimensions	5 ft. Length and Width

306.2.6 Surface Treatments

The sidewalk surface treatment can have a significant impact on the overall accessibility and comfort level of the facility. The requirement is that the surface be stable, firm and slip resistant. There shall be an unobstructed reduced vibration zone within a pedestrian access route, this minimum width being 48 inches.

Concrete, asphalt or gravel walks may be specified according to the location and the particular need:

1. Concrete walks are the most widely used type. They should normally be 4 inches thick. The exception is at driveway locations where the thickness is increased to 6 inches, or drive thickness, whichever is greater.

2. Asphalt and gravel walks are used mostly in parks, rest areas, or for shared-use paths. Asphalt walks should be constructed of 2 inches of asphalt and 5 inches of aggregate base. Gravel walks should be constructed of 4 inches of compacted aggregate base. Increased thicknesses may be needed if maintenance or emergency vehicles will routinely use paths.

Specialty surface treatments are often desired for aesthetic reasons. But a disadvantage of either bricks or stamped concrete/brick decorative sidewalks is the problem seemingly small surface irregularities pose for certain wheelchair users. Designers should provide a zone of reduced vibration. It is possible to enhance sidewalk aesthetics while still providing a smooth walking surface by combining a smooth concrete walking area with a decorative edging. See FHWA's Designing Sidewalks and Trails for Access, Part 2, Best Practices Design Guide for information.

306.3 Curb Ramps

306.3.1 Curb Ramp Locations

Section 729.12 of the Ohio Revised Code requires that all new or reconstructed curbs shall have curb ramps at each pedestrian crosswalk so that the sidewalk and street blend to a common level.

All newly constructed or modified curb ramps must be ADA compliant. Curb ramps shall be provided on all plans where curb and walks are being constructed, reconstructed or altered at intersections and other major points of pedestrian curb crossing such as mid-block crosswalks.

If a project has curbs and pedestrians are allowed, curb ramps need to be installed wherever sidewalks are present. In areas without sidewalks, pedestrian curb cuts as shown on ***Figure 306-4*** are required if no curb ramps are provided.

Curb ramps are also to be installed in resurfacing projects as outlined in ODOT Policy 21-003(P) Curb Ramps Required in Resurfacing Plans.

It is desirable to provide a continuous path for the persons with disabilities. When a curb ramp is built on one side of a street, a companion curb ramp is required on the opposite side of the street. Therefore, when normal project or work limits end within an intersection, the work limits must be extended to allow construction of companion ramps. The basic requirement is that a crosswalk must be accessible via curb ramps from both ends, not one end only. In most cases, curb ramps will be installed in all quadrants of an intersection.

306.3.2 Design Considerations

Curb ramps should be designed to the least slope consistent with the curb height, available corner area and underlying topography. A level landing is necessary for turning, maneuvering or bypassing the sloped surface. Proper curb ramp design is important to users either continuing along a sidewalk path or attempting to cross the street.

306.3.3 Curb Ramps Components

The basic components to the standard curb ramp design are explained here and depicted on ***Figure 306-3.***

1. Ramps - The grade of a ramp must not exceed 0.083. The cross slope must not be greater than 0.02. The recommended minimum width of a curb ramp is 4 ft.

2. Gutters - Gutters require a counter slope at the point at which the ramp meets the street for proper drainage. This counter slope may not exceed 0.05, and the change in angle must be flush, without a lip, raised joint or gap. Lips or gaps between the curb ramp slope and counter slope can arrest forward motion by catching caster wheels or crutch tips. The algebraic difference between the ramp slope and the gutter counter slope cannot exceed 11 percent, or a 24 inch level strip must be provided between the two slopes. See ***Figure 306-3***.

3. Landings - Landings provide a level area (less than 2 percent slope in any direction) for wheelchair users to maneuver into or out of the curb ramp, or to simply bypass it. A level landing 5 ft. square is preferred. Level landings are required at the top and bottom of each ramp.

4. Flares - Curb ramp flares are graded transitions from a curb ramp to the surrounding sidewalk. Flares are not intended to be wheelchair routes, and may be one of the cues used to identify the presence of a curb ramp. Flares are only needed in locations where the ramp edge abuts pavement. A curb edge is used as a visual cue where the ramp edge abuts grass or landscaping

306.3.4 Curb Ramp Types

Three types of ramps are currently used in street corner designs. In all cases curb ramps should be located entirely within the marked crosswalks (where they exist). Drainage grates or inlets should not be located within the crosswalk area, where wheelchair casters or canes tips may be caught. Nonetheless, curb ramps need to be adequately drained. See ***Figure 306-4*** for a sketch of these types, and for details see ***Standard Construction Drawing BP-7.1***.

Perpendicular Curb Ramps are generally perpendicular to the curb. Users will generally be traveling perpendicular to vehicular traffic when they enter the street at the bottom of the ramp. Advantages include providing a straight path of travel on tight radius corners at the expected crossing location for all pedestrians. Disadvantages are that they do not provide a straight path of travel on large radius corners and they require a level landing that takes up additional right-of-way. Perpendicular ramps are generally the best design for pedestrians, provided that a minimum 4 foot landing is available for each approach.

Parallel Curb Ramps have two ramps leading down towards a center level landing at the bottom between both ramps, with a level landing at the top of each ramp. They can be installed where the available space between the curb and property line is too tight to permit the installation of both a ramp and a landing, and are effective on steep terrain or at locations with high curbs. Unfortunately, sidewalk users have to negotiate two ramp grades. Since the landing is depressed and level, drainage of the ramp landing at the street must be carefully designed.

Diagonal Curb Ramps are a single curb ramp that is located at the apex of the corner. Diagonal Curb Ramps are not acceptable designs for access to new sidewalks, but may be applied in retrofit locations where a pair of perpendicular ramps is not feasible due to existing site constraints. This design directs a visually impaired person away from the crosswalk and into traffic. Therefore when designed the entire lower landing area must fall within the crosswalk that the ramp serves and cannot be located in the traveled lane of opposing traffic.

306.3.5 Detectable Warnings

Detectable warnings are standardized surface features on walking surfaces to warn visually impaired people of the transition between the sidewalk and the street.

Truncated domes are specified as the detectable warnings to be used and are to be included in all connections to all street crossings to mark the street edge, where a sidewalk crosses a vehicular way. This includes islands and medians that are cut through level with the roadway.

Detectable Warnings should be used at the following locations:

- At the edge of depressed corners,
- At the border of raised crosswalks and raised intersections,
- At the base of curb ramps,
- At the border of median and islands,
- At street crossing for shared-use paths, and
- Where sidewalks cross railroad tracks.

Detectable Warnings are not needed where a sidewalk crosses an unsignalized driveway, nor where the sidewalk crosses an alley.

Truncated dome dimensions and alignment can be found on **Standard Construction Drawing BP-7.1**.

306.3.6 Curb Ramp Evaluation

Resurfacing is an alteration that triggers the requirement to add curb ramps if it involves work on a street or roadway spanning from one intersection to another, and includes overlays of additional material to the road surface, with or without milling. Examples include, but are not limited to the following treatments or their equivalents: addition of a new layer of asphalt, reconstruction, concrete pavement rehabilitation and reconstruction, open-graded surface course, micro-surfacing and thin lift overlays, cape seals and in-place asphalt recycling.

Projects classified as maintenance do not trigger an evaluation of existing curb ramps and the addition of new curb ramps. Maintenance treatments are those that serve solely to seal and protect the road surface, improve friction, and control splash and spray are considered to be maintenance because they do not significantly affect the public's access to or usability of the road. Some examples of the types of treatments that would normally be considered maintenance are: painting or striping lanes, crack filling and sealing, surface sealing, chip seals, slurry seals, for seals, scrub sealing, joint crack seals, joint repairs, dowel bar retrofit, spot high-friction treatments, diamond grinding, and pavement patching. A curb ramp evaluation form is available on the Office of Roadway Engineering website.

306.3.7 Exception to Curb Ramp Replacement

Existing curb ramps in good repair (without spalling, cracking or uneven surfaces) may remain in place if they met the 1991 ADA standard at the time they were constructed. If no detectable warnings are present on an existing curb ramp constructed under the 1991 ADA standard, truncated domes should be added. The portion of the curb ramp not effected by the installation of the detectable warnings may remain in place.

306.4 Sidewalks for Highway Bridges/Underpasses

306.4.1 General

Provisions should always be made to include some type of walking facility as part of a vehicular bridge or underpass, if only as an emergency exit path. Wherever possible, sidewalk widths across bridges and through underpasses should be the same as the clear width of the existing connecting sidewalks.

306.4.2 Walks on Bridges

Walks should be provided on bridges located in urban or suburban areas having curbed typical sections under the following conditions:

1. Where there are existing walks on the bridge and/or bridge approaches, or

2. Where evidence can be shown through local planning processes, or similar justification, that walks will be required in the future (20 years). Anticipated pedestrian volumes of 50 per day would justify a walk on one side and 100 per day would justify walks on both sides.

Walks on bridges should preferably be 6 ft. in residential areas and 8 ft. in commercial areas measured from the face of curb to the face of parapet. The width should never be less than 5 ft.

In rural areas or other sites where flush shoulders approach a bridge and light pedestrian traffic is anticipated on the shoulders, the shoulder width should be continued across the bridge using the preferred lateral clearance from ***Figure 302-1***, or greater if deemed appropriate. A raised walkway should not be used in these areas. Where an existing bridge has a safety curb (used as a walkway) and removal is not economically justified, the ends of the walkway should be shielded with a traffic barrier or ramped into the approach shoulder at a vertical transition rate of approximately 20:1.

306.4.3 Walks under Bridges

The criteria for providing walks at underpasses are basically the same as described above for Walks on Bridges. An exception is in areas where there are no approach walks, space will be provided for future walks, but walks generally will not be constructed with the project unless there is concurrent approach walk construction.

Where the approach walks at underpasses include a tree lawn, the tree lawn width may be carried through the underpass wherever space permits.

306.5 Pedestrian Overpasses and Underpasses

306.5.1 General

Due to the high costs of constructing pedestrian- only structures, they should be considered only where other more standard and/or less costly solutions are not acceptable. Both pedestrian overpasses and underpasses need to meet ADA ramp criteria for maximum slopes (0.083), landings every 30 ft. of run, and handrails; or elevators.

Freeways should not have pedestrian crossings at-grade and may require the occasional use of separate pedestrian structures.

Underpasses that are below grade should provide clear sight distances to and through the underpass. A minimum width of 14-16 ft. is desirable, but longer tunnels need to be wider for security. Likewise, vertical clearance of 8 ft. is sufficient for short tunnels, but longer ones may need 10 ft. Heights of maintenance and emergency vehicles need to be addressed. Drainage must be carefully considered.

Both pedestrian overpasses and underpasses should be adequately illuminated.

306.5.2 Guidelines

Experience has shown that the primary location for pedestrian overpass/underpass is an urban area outside the central business district. Such a pedestrian crossing may be considered when the following conditions exist:

1. The community has expressed a strong desire for a pedestrian crossing.
2. A reasonable alternate route for pedestrian is not available.
3. There is no signal, stop intersection, or pedestrian crossing available within 660 ft. of the proposed location.
4. Pedestrians can be prevented from crossing at grade.
5. Physical conditions permit construction.
6. The traffic volume and pedestrian volume are above those required to warrant the installation of pedestrian signals as stated in the Ohio Manual of Uniform Traffic Control Devices for Streets and Highways (OMUTCD). This stipulation can be waived in special cases such as when sight distances are limited.
7. Where there are a large number of pedestrians who must regularly cross a high-speed, high volume roadway.

307 GRADING AND SIDESLOPES

307.1 General

This section is concerned with the design of slopes, ditches, parallel channels and interchange grading. It incorporates into the roadside design, the concepts of vehicular safety developed through dynamic testing. Designers are urged to consider flat foreslopes and backslopes, wide gentle ditch sections and elimination of barriers in their initial design approach.

307.2 Slopes

307.2.1 Roadside Grading

There are several combinations of slopes and ditch sections that may be used in the grading of a project. Details and use of these combinations are discussed in subsequent paragraphs. In general, slopes should be made as flat as possible to minimize the necessity for barrier protection and to maximize the opportunity for a driver to recover control of a vehicle after leaving the traveled way. Regardless of the type of grading used, projects should be examined in an effort to obtain flat slopes at low costs. For instance, fill slopes can be flattened with material which otherwise might be wasted and backslopes can be flattened to reduce borrow.

ODOT does not allow non-ODOT agencies to use ODOT right of way for the purpose of locating stormwater Best Management Practices (BMPs).

In order to more fully understand the discussions on the various types of grading, the designer should become familiar with the need for barrier protection and the clear zone concept covered in ***Section 600***.

Safety grading is the shaping of the roadside using 6:1 or flatter slopes within the clear zone area (***Section 600.2***) and 3:1 or flatter foreslopes and recoverable ditches extending beyond the clear zone. Safety grading is used on Interstate, other freeways and expressways. ***Figures 307-1*** and ***307-2*** show many of these details.

Clear zone grading is the shaping of the roadside using 4:1 or flatter foreslopes and traversable ditches within the clear zone area. Foreslopes of 3:1 may be used, but are not measured as part of the clear zone distance. Clear zone grading is recommended for undivided rural facilities where the design speed is 50 mph or greater, the design hourly volume is 100 or greater, and when at least one of the following conditions exists:

1. The wider cross section is consistent with present or future planning for the facility.
2. The project is new construction or major reconstruction involving significant length.
3. The wider cross section can be provided at little or no additional cost.

Figure 307-3 shows examples of clear zone grading and traversable ditches.

Common grading is the shaping of the roadside using 3:1 or flatter foreslopes and normal ditches. It is used on undivided facilities where the conditions for the use of safety grading or clear zone grading do not exist. The designer should ensure that all obstacles within the clear zone receive proper consideration. ***Figure 307-4*** shows examples of common grading and normal ditches.

Barrier grading is the shaping of the roadside when barrier is required for slope protection. Normally 2:1 foreslopes and normal ditch sections are used. ***Figure 307-4*** gives an example of barrier grading.

307.2.2 Slope Transitions

When clear zone grading is used to eliminate the need for barrier protection of a fixed object, the length of slope transition should be determined using the length of need concept described in ***Section 602.1***. The clear zone measurement should be used for the Lateral Extent of the Hazard ("LH"). Clear Zone grading should not be utilized unless the required lane and shoulder widths are present.

As shown in these conditions, Clear Zone grading is desirable throughout a roadway corridor. It does little to increase the safety of a roadway if Clear Zone grading is only done on spot projects such as culvert replacements, if the rest of the corridor will be maintained with common grading.

307.2.3 Rounding of Slopes

Slopes should be rounded at the break points and at the intersection with the existing ground line to reduce the chance of a vehicle becoming airborne and to harmonize with the existing topography. Recommended rounding at the edge of the graded shoulder is shown in ***Figure 301-3***. Rounding at other locations is shown in ***Figures 307-1, 307-3*** and ***307-4***.

307.2.4 Special Median Grading

Figure 307-5 shows some examples of median grading when separate roadway profiles are used.

307.2.5 Rock Slopes (See Figure 307-5)

In rock cuts, determine the cut slope angle(s) and necessary slope benches using design guidance presented in Geotechnical Bulletin 3, "Rock Cut Slope and Catchment Design". The designer should examine the project to ascertain whether flatter slopes could be used to the advantage of reducing borrow within a reasonable haul distance. Such a situation should also be discussed with the Office of Geotechnical Engineering.

307.2.6 Curbed Streets

The slope treatment adjacent to curbed streets is shown on ***Figure 307-6***.

307.2.7 Driveways and Cross Roads

At driveways or crossroads, where the roadside ditch is within the clear zone distance and where clear zone grading can be obtained, the ditch and pipe should be located as shown on ***Figure 307-7***.

Requirements for pipe location should be applied to all new construction, reconstruction, widening and resurfacing projects if regrading of the roadsides to safety or clear zone grading is included in the work. New driveways constructed by permit should also conform to the above if other such installations on the route conform, otherwise the new driveway pipe may be located in the existing roadside ditch.

307.3 Ditches

When the depth or velocity of the design discharge accumulating in a roadside or median ditch exceeds the desirable maximum established for the various highway classifications, a storm sewer will be required to intercept the flow and carry it to a satisfactory outlet. If right-of-way and earth work considerations are favorable, a deep parallel side ditch (see ***Figure 307-5***) may be more practical and should be considered instead of a storm sewer.

In some cases where large areas contribute flow to a highly erodible soil cut, an intercepting ditch may be considered near the top of the cut to intercept the flow from the outside and thereby relieve the roadside ditch.

Constant depth ditches (usually 18 inches deep) are desirable. Where used, the minimum pavement profile grades should be 0.24% to 0.48%. Where flatter pavement grades are necessary, separate ditch profiles are developed and the ditch flow line elevations are shown on each cross section.

307.4 Parallel Channels

Where it is desirable that a stream intercepted by the improvement be relocated parallel to the roadway, the channel should be located beyond the limited access line in a channel easement. This does not apply to conventional intercepting erosion control ditches located at the top of cut slopes in rolling terrain. This arrangement locates the channel beyond the right-of-way fence. See ***Figure 307-5***.

In areas of low fill and shallow cut, protection along a channel by a wide bench is usually provided. Fill slopes should not exceed 6:1 when this design is used and the maximum height from shoulder edge to bench should generally not exceed 10 ft. If it should become necessary to use slopes steeper than 6:1, guardrail may be necessary and fill slopes as steep as 2:1 may be used.

In cut sections 5 ft. or more in depth, earth barrier protection can be provided. Where very deep channels are constructed, this design probably affords greater protection and requires less excavation. See ***Figure 307-5***. Where the sections alternate between cut and fill and it is desired to use a single design, earth barrier protection would be less costly if waste is a problem. Likewise, bench protection would be less costly if borrow is needed.

Earth bench or earth barrier protection provided adjacent to parallel channels should not be breached for any reason other than to provide an opening for a natural or relocated stream requiring a drainage structure larger than 42 inches in rise. Outlet pipes from median drains or side ditches shall discharge directly into the parallel channel.

Channels and toe-of-slope ditches, used in connection with steep fill slopes, should be removed from the normal roadside section by benches. The designer shall establish control offsets to the center of each channel or ditch at appropriate points which will govern their alignment so they will flow in the best and most direct course to the outlet. Bench width shall be varied as necessary (See ***Figure 307-5***).

307.5 Interchange Grading

Interchange interiors should be contour graded so the least amount of guardrail is required and so maximum safety is provided with corresponding ease of maintenance. Sight distance is critical for passenger vehicles on ramps as they approach entrance or merge areas, especially if barrier is erected on the merging side of the vehicle. Therefore, sight distance shall be unobstructed by landscaping, earth mounds or other barriers.

307.5.1 Crossroads

At a road crossing within an interchange area, bridge spill-through slopes should be 2:1, unless otherwise required by structure design. They should be flattened to 3:1 or flatter in each corner cone and maintained at 3:1 or flatter if within the interior of an interchange. Elsewhere in interchange interiors, fill slopes should not exceed 3:1.

307.5.2 Ramps

Roadside design for ramps should be based on the mainline grading concept.

307.5.3 Gore Area (See Figure 307-8)

Gore areas of trumpets, diamonds and exteriors of loops adjacent to the exit point, should be graded to obtain slopes (6:1 or flatter) which will not endanger a vehicle which is unable to negotiate the curvature because of excessive speed.

307.5.4 Trumpet Interiors (See Figure 307-8)

Interior areas of trumpets should be graded to slopes not in excess of 8:1, sloping downward from each side of the triangle to a single rounded low point. Roadside ditches should not be used. Exteriors should be graded in accordance with the mainline or ramp standards.

307.5.5 Loop Interiors (See Figure 307-9)

In cut, the interior should be graded to form a normal ditch section adjacent to the lower part of the loop and the backslope should be extended to intersect the opposite shoulder of the upper part of the loop, unless the character and the amount of material or the adjacent earth work balances indicate that

the cost would be prohibitive. Roadside cleanup and landscaping should be provided in undisturbed areas of loop interiors.
If channels are permitted to cross the loop interior, slopes should not be steeper than 4:1.

307.5.6 Diamonds

If the location of the ramp intersection at the crossroad is relatively near to the main facility, a continuous slope between the upper roadway shoulder and the lower roadway ditch will provide the best and most pleasing design.

If the ramp intersection at the crossroad is located a considerable distance from the main facility, then both ramp and mainline roadsides should have independent designs, until the slopes merge near the gore. If the quadrant is entirely, or nearly so, in cut, it is suggested that the combination of a 3:1 backslope at the low roadway ditch and a gentle downslope from the high roadway shoulder will provide the best design in the wide portion of the quadrant. Approaching the gore, the slopes should transition to continuous 4:1 and 6:1 or flatter slopes.

Quadrants located entirely in fill areas should have independently designed roadways for ramp, mainline and crossroad. Each should be provided with normal slopes not greater than 3:1, with the otherwise ungraded areas sloped to drain without using ditches.

If the quadrant is located partially in cut and partially in fill, the best design would feature a gentle fill slope at the upper roadway and a gentle backslope at the lower roadway joined to a bench at the existing ground level which is sloped to drain.

The combination of a long diamond ramp having gentle alignment with a loop ramp in the same interchange quadrant is not to be treated as a trumpet. Each ramp should be designed independently of the other in accordance with the suggested details set forth above.

307.6 Disposal of Construction Debris and Waste Material within ODOT R/W

All projects with pavement removal, particularly non-recyclable concrete pavement, or an excess of excavation should be evaluated for acceptable disposal areas within the state right-of-way. This material cannot be arbitrarily dumped within the limits of state right-of-way. If improperly placed, the material may interfere with adequate sight distance and may create an unnecessary hazard.

Acceptable disposal areas would preferably enhance highway operations and should not in any way reduce safety. Instead of hauling the material offsite or improperly placing the material, the excess fill may be used throughout the state right-of-way limits to improve grading and general roadside safety. For example, all interstate and interstate look-alike systems should use safety grading. If safety grading currently exists, consider extending it to the right-of-way limit. If clear zone grading currently exists, consider using safety grading or extend the clear zone grading to the right-of-way limit. Each barrier location should be evaluated to see if the application of safety grading, or at a minimum clear zone grading, would eliminate the need for barrier. Adjustments to drainage or drainage structures may also be required.

The determination as to whether or not to allow the disposal of waste material within the right-of-way of a project should be made as soon as possible in the project development process. Possible waste areas within the project right-of-way limits should be identified during the field review prior to final scope preparation. Areas deemed acceptable should be identified accordingly in the construction plans. If none of the areas are considered acceptable, this should also be clearly noted in the construction plans in the form of a plan note.

For the full text of the guidelines see Guidelines for Identifying Acceptable Locations for the Disposal of Waste Material and Construction Debris or the Excavation of Borrow Material within ODOT Right-of-Way located in the Reference section of this manual.

307.6.1 Exit Ramps

Fill material may be placed in the infield areas of exit ramps as long as the decision sight distance is provided and 6:1 or flatter slopes are provided in the gore areas. Decision sight distances, Avoidance Maneuver A or B, as per ***Figure 201-6*** should be provided for the design speed of the ramp. Also note that with respect to a diamond interchange, the placement of the fill material in the infields should not be such that it interferes with the intersection sight distance at the intersection of the crossroad and the exit ramp.

307.6.2 Entrance Ramps

Excess or disposable fill material should not be placed adjacent to an entrance ramp such that it interferes with the available sight distance. Decision sight distance, Avoidance Maneuver C or E, as per ***Figure 201-6*** should be provided for the design speed of the ramp. The decision sight distance is measured from a point on the ramp where the driver, on the ramp, has an unobstructed view of the mainline to where the lane width becomes less than 10 ft. and the driver must merge. This is the distance that the driver merging from the ramp has to decide where he can safely merge into the mainline traffic. This distance should also be unobstructed for the mainline driver to react to the ramp vehicle by either a lane or speed change.

307.6.3 Loop Ramps

In general, the infields of loop ramps should not be filled unless it is to eliminate barrier or provide safety graded slopes. Filling these areas may decrease sight distance and diminish the driver=s ability to anticipate the sharpness and total path of the ramp. It is important to have an unobstructed view of the ramp in order that driver may have adequate time to react to possible obstructions and delays ahead. Loop ramps are more susceptible to run off the road accidents due to the sharp curvature and high speeds.

If a designer chooses to fill the infield, as a minimum, the decision sight distance, Avoidance Maneuver A or B, as per ***Figure 201-6***, using the ramp design speed, should be provided for the exit portion of the ramp. Likewise, Avoidance C or E, using the ramp design speed, should be provided for the entrance portion of the ramp. The fill height should not exceed 20 ft. in height or as determined by the Office of Geotechnical Engineering. Slopes should not exceed 4:1 for ease of maintenance.

307.6.4 Adjacent to Noise Sensitive Areas

Excess or disposable fill material may be placed adjacent to a noise sensitive area via the construction of a small height berm. Consult with the Office of Environmental Services-Policy Section-Noise Unit regarding opportunities. A minimum 3'-6" tall berm height is recommended. Consult with the Office of Geotechnical Engineering regarding taller berm heights. Designer must adhere to clear zone requirements in LDM Vol 1 Section 600 and grading requirements in LDM ***Section 307***. The designer must consider issues including but not limited to underground utilities, tower lighting, signage, landfills, floodplains, utility markers, valve boxes, manholes, hydrants, exposed conduits, drainage concerns, tree removal, ecological items, etc.

308 ON-ROAD BICYCLE FACILITIES

308.1 General

This section provides an overview of designs that facilitate safe, efficient and convenient travel for bicyclists on roadways. Bicyclists often have to share these roadways with motorized vehicles as they travel.

308.2 Design

Generally, the basic geometric design guidelines for motor vehicles will result in a facility that will provide a safe accommodation for on-street bicyclists. If properly designed for motor vehicles, roadway design elements such as stopping sight distance, horizontal and vertical alignment, grades and cross slopes will meet or exceed the minimum design standards applicable to bicyclists. See AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition for additional information regarding the design of On-Road Bicycle Facilities.

308.3 Shared Lanes

Bicycles may be operated on all roadways except where prohibited by statute or regulation. Shared lanes where bicyclists and motor vehicles share the same travel lanes exist everywhere; on local neighborhood streets, on city streets, and urban, suburban and rural highways. There are no bicycle-specific designs or dimensions for shared lanes or roadways, but various design features can make shared lanes more compatible with bicycling, such as adequate sight distance and roadway designs that encourage lower speeds.

308.3.1 Shared Lanes on Major Roadways (Wide Curb/Outside Lanes)

Motor vehicles will begin encroaching at least part way into the next lane for lane widths of 13 ft. or less to pass a bicyclist. Lane widths of 14 ft. or greater will allow motorists to pass bicyclists without encroaching into the adjacent lane. For additional information on shared lane widths see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.4 Paved Shoulders

Bicyclist accommodations on roadways with higher speeds or traffic volumes can be greatly improved by adding, improving or expanding paved shoulders.

Paved shoulders are different from bicycle lanes, in that at intersection approaches paved shoulders are placed to the right of the right-turn lanes and bike lanes are placed on the left side of right-turn lanes since they are intended to serve the through movements by bicyclists. Through moving bicyclists should normally be to the left of right-turning motor vehicles to avoid conflicts. On roadways with paved shoulders that approach right-turn lanes, some jurisdictions introduce a bike lane only at the intersections, and then transition back to a paved shoulder. For more information on paved shoulders see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

For uncurbed roadways with no vertical obstructions immediately adjacent to the roadway, paved shoulders should be at least 4 ft. wide to accommodate bicycle travel. A shoulder width of at least 5 ft. is recommended from the face of any vertical obstruction such as guardrail, curb, or other roadside barrier since bicyclists generally shy away from a vertical face. It is desirable to increase the width of shoulders where any of the following conditions exist: high bicycle usage is expected, motor vehicle speeds exceed 50 mph, use by heavy trucks, buses, or recreational vehicles is considerable or static obstructions exist at the right side of the roadway.

On two-way roads it is preferable to provide paved shoulders on both sides; however, in constrained locations where pavement width is limited, it may be preferable to provide a wider shoulder on only one side of the roadway, rather than to provide a narrow shoulder on both sides. This approach may prove beneficial in the following situations:

1. On uphill roadway sections, a shoulder may be provided to give slow-moving bicyclists additional maneuvering space, thereby reducing conflicts with faster moving motor vehicle traffic.

2. On roadway sections with vertical or horizontal curves that limit sight distance, it can be helpful to provide shoulders over the crest and on the downgrade of a vertical curve, and on the inside of a horizontal curve.

For more information on paved shoulders see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.5 Bicycle Lanes

308.5.1 General Considerations

Bicycle lanes are one-way facilities designated for preferential use by bicyclists that typically carry bicycle traffic in the same direction as adjacent motor vehicle traffic. Bike lanes are the appropriate and preferred bicycle facility for thoroughfares in both urban and suburban areas. Where there is a high potential for bicycle use, bike lanes may be provided on rural roadways near urban areas. Paved shoulders may be designated as bike lanes by installing bike lane symbol markings.

308.5.2 Bicycle Lanes on Two-Way Streets

Bike lanes should be provided on both sides of two-way streets since a bike lane provided on only one side may invite wrong-way use. For additional information on bicycle lanes on two-way streets see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.5.3 Bicycle Lanes on One-Way Streets

On one-way streets the bike lane should be on the right-hand side of the roadway. If there are a significant number of left turning bicyclists or if a left-side bike lane decreases conflicts resulting from heavy bus traffic, heavy right-turn movements (including double right-turn lanes), deliveries, or on-street parking a bike lane may be placed on the left side of the roadway.

Bike lanes should typically be provided on both streets of a one-way couplet in order to provide facilities in both directions and discourage wrong-way riding. If width constraints or other conditions make it impractical to provide bike lanes on both streets, shared-lane markings should be considered on the constrained street. This provides a more complete network and encourages bicyclists to travel with the flow of the other traffic.

308.5.4 Bicycle Lane Widths

Bicycle lane widths should be determined based on the speed, volume, and type of vehicles in adjacent lanes since these factors significantly affect bicyclists' comfort and desire for lateral separation from other vehicles. Also, the appropriate width should take into account design features at the right edge of the bicycle lane, such as the curb, gutter, on-street parking lane, guardrail or other roadside barrier.

The preferred operating bicycle lane width is 5 ft. Wider bicycle lanes may be desirable under the following conditions:

- Adjacent to a parking lane (7 ft.) with a high turnover (such as those servicing restaurants, shops, or entertainment venues), a wider bicycle lane (6-7 ft.) provides more operating space for bicyclists to ride out of the area of opening vehicle doors.

- In areas with high bicycle use and without on-street parking, a bicycle lane width of 6 to 8 ft. makes it possible for bicyclists to ride side-by-side or pass each other without leaving the lane.

- On high-speed (greater than 45 mph) and high-volume roadways, or where there is a substantial volume of heavy vehicles, a wide bicycle lane provides additional lateral separation between motor vehicles and bicycles to minimize wind blast and other effects.

The minimum width of a bicycle lane is 4 ft. for roadways with no curb and gutter and no on-street parking. For roadways where the bike lane is immediately adjacent to the curb, guardrails or other vertical surface, the minimum bike lane width is 5 ft., measured from the face of a curb or vertical surface to the center of the bike lane line. There are two exceptions to this:

- Where a 2-ft. wide gutter is used, the preferred bike lane width is 6 to 7 ft. inclusive of the gutter. Gutters with longitudinal seams running parallel to bicycle travel can pose a potential crash hazard due to deterioration between the asphalt and concrete surfaces or any unevenness at the edges due to resurfacing.

- On extremely constrained, low-speed roadways with curbs but no gutter, where the preferred bike lane width cannot be achieved despite narrowing all other travel lanes to their minimum widths, a 4-ft. wide bike lane can be used.

For additional information or design considerations concerning bicycle lanes widths see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.5.5 Bicycle Lanes and On-Street Parking

Where on-street parking is permitted, the bike lane should be located between the parking lane and the travel lane. The recommended bike lane width in these locations is 6 ft. and the minimum width is 5 ft.

Bike lanes should not be placed between the parking lane and the curb. Such placement reduces visibility at driveways and intersections, increases conflicts with opening car doors, complicates maintenance, and prevents bike lane users from making convenient left turns.

Parallel Parking

Where bike lanes are installed adjacent to parallel parking, the recommended width of a marked parking lane is 8 ft., and the minimum width is 7 ft. Where parallel parking is permitted but a parking lane line or stall markings are not utilized, the recommended width of the shared bicycle and parking lane is 13 ft. If parking usage is low and turnover is infrequent a minimum width of 12 ft. may be satisfactory.

Diagonal Parking

Bike lanes should normally not be placed adjacent to conventional front-in diagonal parking, since drivers backing out of parking spaces have poor visibility of bicyclists in the bike lane.

The use of back-in diagonal parking can help mitigate the conflicts normally associated with bike lanes adjacent to angled parking. For additional information on the benefits of back-in diagonal parking see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.5.6 Bicycle Lanes at Intersections

Intersections and driveways present the increased likelihood for conflicts between bicyclists and motor vehicles.

For additional information regarding turning considerations for bicycle lanes see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.6 Retrofitting Bicycle Facilities on Existing Streets and Highways

Existing streets and highways can be retrofitted to improve bicycle accommodations by either widening the roadway or by reconfiguring the existing roadway. Paved shoulders can be added to improve mobility and comfort for bicyclists and reduce bicycle related crashes on busier or higher-speed rural roads. It may be possible to accommodate bike lanes on urban (curbed) roadways by reconfiguring travel lanes or make other adjustments that better accommodate bicyclists where reconfiguration of the lanes is not practical.

When retrofitting roads for bicycle facilities, the width guidelines for bike lanes and paved shoulders (see ***Sections 308.4*** and ***308.5.4***) should be applied. For additional information on retrofitting bicycle facilities on existing streets and highways see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

Retrofitting bicycle facilities on bridges presents special challenges because it may be impractical to widen an existing bridge. The guidance in ***Section 308.6.2*** for retrofitting bicycle facilities without roadway widening is applicable to existing bridges. Further guidance on accommodating bicyclists on bridges is presented in ***Section 308.8.2***.

308.6.1 Retrofitting Bicycle Facilities by Widening the Roadway

Where right-of-way is adequate, or where additional right-of-way can be obtained, roads can be widened to provide wide outside lanes, paved shoulders, or bike lanes. Widening must be weighed against the possibility that vehicle speeds will increase, which may adversely impact bicyclists and pedestrians.

308.6.2 Retrofitting Bicycle Facilities without Roadway Widening

In many areas, especially built-out urban and suburban areas, physical widening is impractical, and bicycle facility retrofits have to be done within the existing paved width. There are three methods of modifying the allocation of roadway space to improve bicyclist accommodation:

1. Reduce or reallocate the width used by travel lanes.
2. Reduce the number of travel lanes.
3. Reconfigure or reduce on-street parking.

In most cases, travel lane widths can be reduced without any significant changes in levels of service for motorists. Before a reduction or reallocation in the number of travel lanes or their widths shall be considered, an operational study should be performed to evaluate the impact of the proposed changes on the level of service of the facility. One benefit is that bicycle LOS will be improved. Creating shoulders or bike lanes on roadways can improve pedestrian conditions as well by providing a buffer between the sidewalk and the roadway.

Reducing Travel Lane Width

In some cases, the width needed for bike lanes or paved shoulders can be obtained by narrowing travel lanes. Lane widths on many roads are greater than the minimum values shown in ***Figures 301-2a*** and **301-4a** and, depending on condition, may be candidates for narrowing.

For additional information concerning the reduction of the travel lane widths see AASHTO's Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

Reducing the Number of Travel Lanes

One method that can be used to integrate bike lanes on existing roadways is reducing the number of travel lanes which is often referred to as a “road diet”. This strategy can be used on streets with excess capacity (more travel lanes than needed to accommodate the existing or projected traffic volumes), especially between intersections.

A traffic study should be conducted to evaluate potential reductions in crash frequency and severity, to evaluate motor vehicle capacity and level of service, to evaluate bicycle LOS, and to identify appropriate signalization modifications and lane assignment at intersections before implementing a road diet.

Road diets have many benefits, often reducing crashes; improving operations; and improving livability for pedestrians, bicyclists, adjacent residents, businesses, and motorists.

For additional information concerning the reduction in the number of travel lanes see AASHTO’s Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

Reconfiguring or Reducing On-Street Parking

For additional information concerning reconfiguring or reducing on-street parking see AASHTO’s Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

308.8 Other Roadway Design Considerations

Bicycle travel should be safely accommodated at railroad crossings, drainage grates, bridges, viaducts, tunnels, traffic signals, interchanges and roundabouts.

Exclusive bicycle lanes should not be provided through the roundabout but rather should be terminated upstream of the entrance line. Bicyclists have the option to either merge into the travel lanes navigating the roundabout in the same fashion as a motorized vehicle since bicycle speeds are in the range of the motorized vehicles or the bicyclist can dismount and walk their bicycle on the sidewalks.

For additional information concerning design features see AASHTO’s Guide for the Development of Bicycle Facilities 2012 Fourth Edition.

300 Cross Section Design

LIST OF FIGURES

* Note: For the design criteria pertaining to Collectors and Local Roads with ADT < 400 or less, refer to the AASHTO Publication - Guidelines for Geometric Design of Very Low-Volume Local Roads ADT < 400)

GUIDE FOR SELECTION OF DESIGN LEVELS OF SERVICE*

301-1
REFERENCE SECTIONS 201.4.1 & 301.1.1

Functional Classification	Locale and Terrain			
	Rural			Urban & Urbanized
	Level	Rolling	Hilly	
Interstate, Other Freeways and Expressways	B	B	C	C or D
Arterial	B	B	C	C or D
Collector	C	C	D	D
Local	D	D	D	D

LEVELS OF SERVICE

A - Free flow, with low volumes and high speeds.

B - Stable flow, speeds beginning to be restricted by traffic conditions.

C - In stable flow zone, but most drivers are restricted in freedom to select own speed.

D - Approaching unstable flow; drivers have little freedom to maneuver.

E - Unstable flow, may be short stoppages.

F - Forced or breakdown flow.

* This table should be used as guidance. The designer should use judgment to choose a design level of service that is practical for each location.

RURAL LANE WIDTHS

301-2

REFERENCE SECTION 301.1.2

Functional Classification	Traffic Design Year ADT	Minimum Lane Widths (ft.) (B)(D) Design Speed (mph) 20	25	30	35	40	45	50	55	60	65	70 or >
Interstate, Other Freeways and Expressways	ALL	--	--	--	--	--	--	12	12	12	12	12
Arterial	>2000	--	--	--	--	12	12	12	12	12	12	12
	400 to 2000	--	--	--	--	11	11	11	12	12	12	12
	<400	--	--	--	--	10	10	11	11	11	11	11
Collector	>2000	11	11	11	11	11	11	11	11 (C)	11 (C)	--	--
	400 to 2000	10	10	10	11	11	11	11	11	11	11	--
	<400	10	10	10	10	10	10	10	11	11	11	--
Local	>2000	11	11	11	11	11	11	11	11 (C)	11 (C)	--	--
	400 to 2000	10	10	10	10	10	11	11	11	11	--	--
	<400	9	9	9	9	9	10	10	11	11	--	--

NOTES:

(A) There may be some rural locations that are urban in character. An example would be a village where adjacent development and other conditions resemble an urban area. In such cases, urban design criteria (Figure 301 -4) may be used.

(B) The number of lanes should be determined by a capacity analysis.

(C) Consider using a12' lane width where substantial truck volumes are present or agricultural equipment frequently use the road

(D) For National Network lane width requirements, see Section105.3.

Note: For the design criteria pertaining to Collectors and Local Roads with ADT's of 400 or less, refer to the MSHTO Publication - Guidelines for Geometric Design of Very Low-Volume Local Roads ADT < 400.

# RURAL SHOULDER CRITERIA (A)	301-3
	REFERENCE SECTIONS 301.2.2, 301.2.3, 307.2.3 & 602.1.1

Functional Classification	Traffic	Graded Width			Treated Width	Type (I)	Rounding (K)		Guardrail Offset (From Traveled Way)
	Design Year ADT	With Barrier	Without Barrier and Foreslope Steeper than 6:1	Without Barrier 6:1 or Flatter Foreslope			Design Speed		
							≥50	<50	
Interstate, Other Freeways & Expressways (Q)	All	15' Rt. 9' Med. (B)	15' Rt. 9' Med. (B)	10' Rt. 4' Med. (D)	10' Rt. (C) 4' Med. (D)	Paved	10'	--	(L) (G)
Arterial (N)	> 2000	13' (P)	12'	8'	8'	PVD (O)	8'	4'	10' (G)
	1501 to 2000	11' (P)	10'	6'	6'	PVD (O)	8'	4'	8' (G)
	400 to 1500	11' (P)	10'	6'	6'	PVD (O)	4'	4'	8' (G)
	< 400	9' (P)	8'	4'	4'	PVD (O)	4'	4'	6' (G)
Collector (N)	> 2000	11' (P)	10'	8'	4'	BIT. SRF. TRT. (J)	8'	4'	8' (M)
	1501 to 2000	9' (P)	8'	6' (E)	4'	STBL. AGG.	8'	4'	6' (M)
	400 to 1500	7' (P)	6'	5'	4'	STBL. AGG.	4'	4'	4'
	< 400	7' (P)	6'	(F)	(F)	STBL. AGG.	4'	4'	4'
Local	> 2000	11' (P)(H)	10' (H)	8' (H)	4'	BIT. SRF. TRT. (J)	8'	4'	8' (M)
	1501 to 2000	9' (P)	8''	6' (E)	4'	STBL. AGG.	8'	4'	6' (M)
	400 to 1500	7' (P)	6'	5'	4'	STBL. AGG.	4'	4'	4'
	< 400	7' (P)	6'	(F)	(F)	STBL. AGG.	4'	4'	4'

See following sheet for corresponding notes.
Note: For the design criteria pertaining to Collectors and Local Roads with ADT≤400, refer to the AASHTO Publication - Guidelines for Geometric Design of Very Low-Volume Local Roads ADT≤400).

Notes to Figure 301-3: Rural Shoulder Criteria

(A) There may be rural locations that are urban in character. An example would be a village where adjacent development and other conditions resemble an urban area. In such cases, urban design criteria (**Figure 301-4**) may be used.

(B) If 6 or more lanes, use 15 ft.

(C) Where truck traffic exceeds 250 DDHV, additional shoulder width may be beneficial. If the treated shoulder width is increased then the graded shoulder width should be increased by the same amount.

(D) If 6 or more lanes, use 10 ft.

(E) A 6 ft. turf shoulder may be used with a 4:1 or flatter foreslope.

(F) See AASHTO=S Guidelines for Geometric Design for Very Low-Volume Local Roads for values.

(G) Concrete barrier may be placed at the edge of treated shoulder when used in lieu of guardrail.

(H) An 8 ft. graded shoulder may be used with a 4:1 or flatter foreslope.

(I) Turf shoulders may be used on non-state maintained roads at option of local government if current year ADT includes less than 250 B and C trucks. Turf shoulders are not to be used on State maintained roads.

(J) Stabilized aggregate may be used on State maintained roads if the design year ADT includes less than 250 B and C truck units. Paved shoulders are recommended if the design year ADT includes over 1000 B and C truck units.

(K) Rounding should be 4 ft. where the foreslope begins beyond the clear zone or where guardrail is installed and foreslope is steeper than 6:1. No rounding is required when the foreslope is 6:1 or flatter.

(L) Guardrail offset is treated width plus 2 ft.

(M) Whenever a design exception is approved for graded shoulder width, the guardrail offset may be reduced but shall not be less than 4 ft.

(N) The median and right shoulder width criteria for Interstates, other freeways and expressways shall apply to the shoulders of divided arterials and divided collectors.

(O) A fully paved shoulder is preferred, but may not be economically feasible. Therefore, a minimum 2 ft. of the treated shoulder should be paved. The remainder of the treated shoulder may be either stabilized aggregate or bituminous surface treated material according to the criteria stipulated in Note (J).

(P) Total Graded Width may be reduced as much as 3 ft. where MGS guardrail with the longer posts is used. See Section 603.1.2 and SCD MGS-1.1 for post length and position details.

(Q) Paved shoulder width reductions of less than 2’ at sign or luminaire foundations or bridge piers will not require a design exception. The 4’ minimum lateral clearance must still be provided.

# URBAN ROADWAY CRITERIA LANE & SHOULDER WIDTHS (A)	301-4
	REFERENCE SECTIONS 301.1.2, 301.2.2, 301.2.3 & 304.2.2

Functional Classification	Locale	Minimum Lane Width (ft.)	Minimum Curbed Shoulder Width (ft.) (F)	
			w/o Parking	with Parking (E)
Interstate, Other Freeways and Expressways (J)	**All**	12	10 Rt. Paved (H) 4 Med. Paved (D)	
Arterial Streets	**50 mph or more**	12	8 Each Side Paved (G)	
	Less than 50 mph	11 (B)	1-2 Paved	7-10 Paved
Collector Streets (I)	**Commercial / Industrial**	11	1-2 Paved	8 - 11 Paved
	Residential	10	1-2 Paved	7 - 8 Paved
Local Streets (I)	**Commercial / Industrial**	11	1-2 Paved	8 Paved
	Residential	10 (C)	1-2 Paved	7 Paved

NOTES:

(A) Use rural criteria (**Figure 301-3**) for uncurbed shoulders. Rural functional classification should be determined after checking the urban route extension into a rural area.

(B) On all Federal Aid Primary (FAP) roadways at least one 12 ft. lane in each direction is required. FAP listings may be obtained from Office of Technical Services' Roadway inventory reports.

See Section 105.3 for more information on the lane width requirements for the FAP and National Network.

(C) Lane width may be 9 ft. where right-of-way is limited and current ADT is less than 250.

(D) Use 10 ft. median shoulder on facilities with 6 or more lanes.

(E) Use minimum lane width if, in the foreseeable future, the parking lane will be used for through traffic during peak hours or continuously.

(F) See **Sections 305.3.2 and 305.3.3** for use of curbs and **Section 602.1.5** for curb/guardrail relationships.

(G) The median and right shoulder width for divided arterials shall follow the shoulder criteria for Interstates, other Freeways and Expressways.

(H) Where truck traffic exceeds 250 DDHV, additional shoulder width may be beneficial.

(I) The AASHTO Publication – Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT≤ 400) may be used for the design criteria of Collector and Local Streets with ADT's of 400 or less.

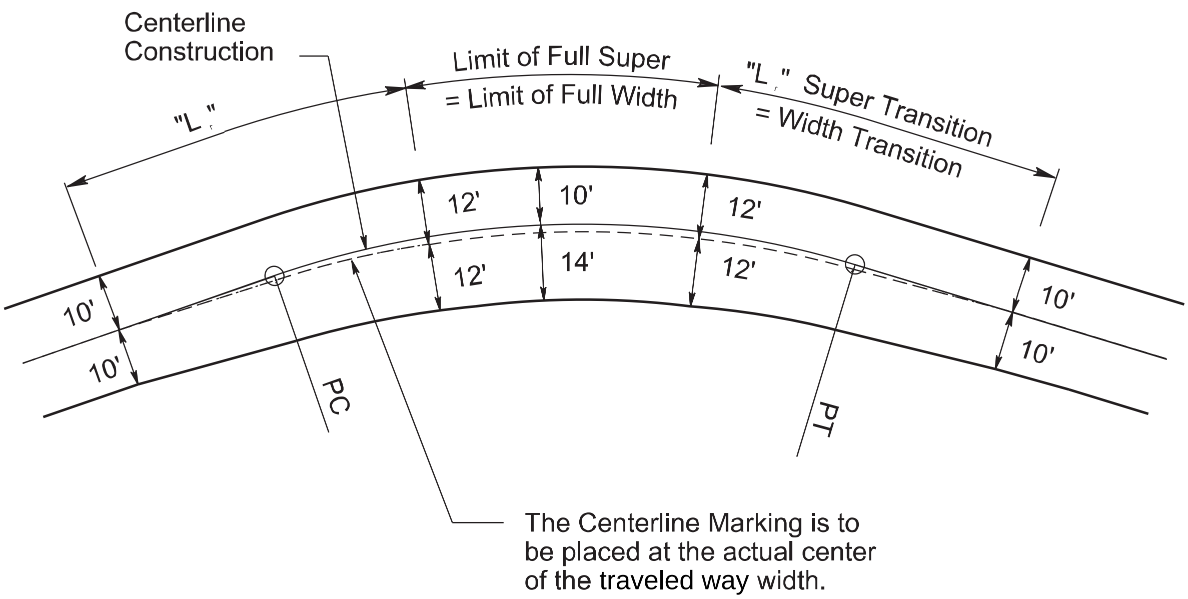
(J) Paved shoulder width reductions of less than 2' will not require a design exception at sign or luminaire foundations or bridge piers. The minimum 4' lateral clearance must still be provided.

TRAVELED WAY WIDENING AT CURVES

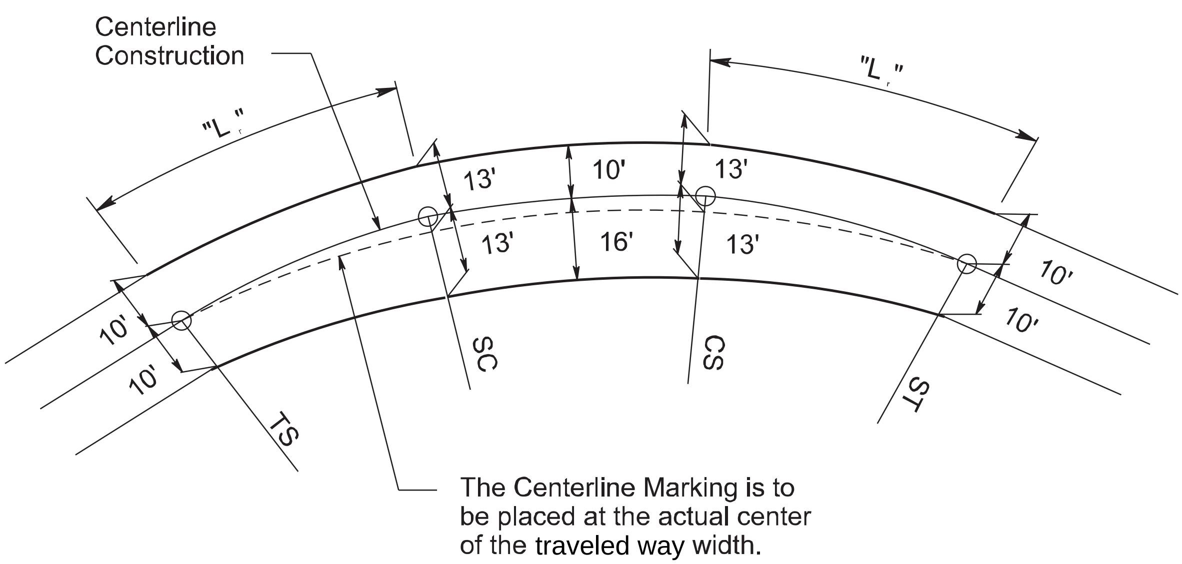
301-5a
REFERENCE SECTIONS 301.1.3

LOCATION OF TRAVELED WAY TRANSITION IN RELATIONSHIP TO THE SUPERELEVATION TRANSITION

Traveled Way Widening for a Simple Curve:



Traveled Way Widening for a Spiral Curve:



TRAVELED WAY WIDENING ON OPEN HIGHWAY CURVES FOR WB-62 DESIGN VEHICLES

301-5c

REFERENCE SECTIONS
301.1.3

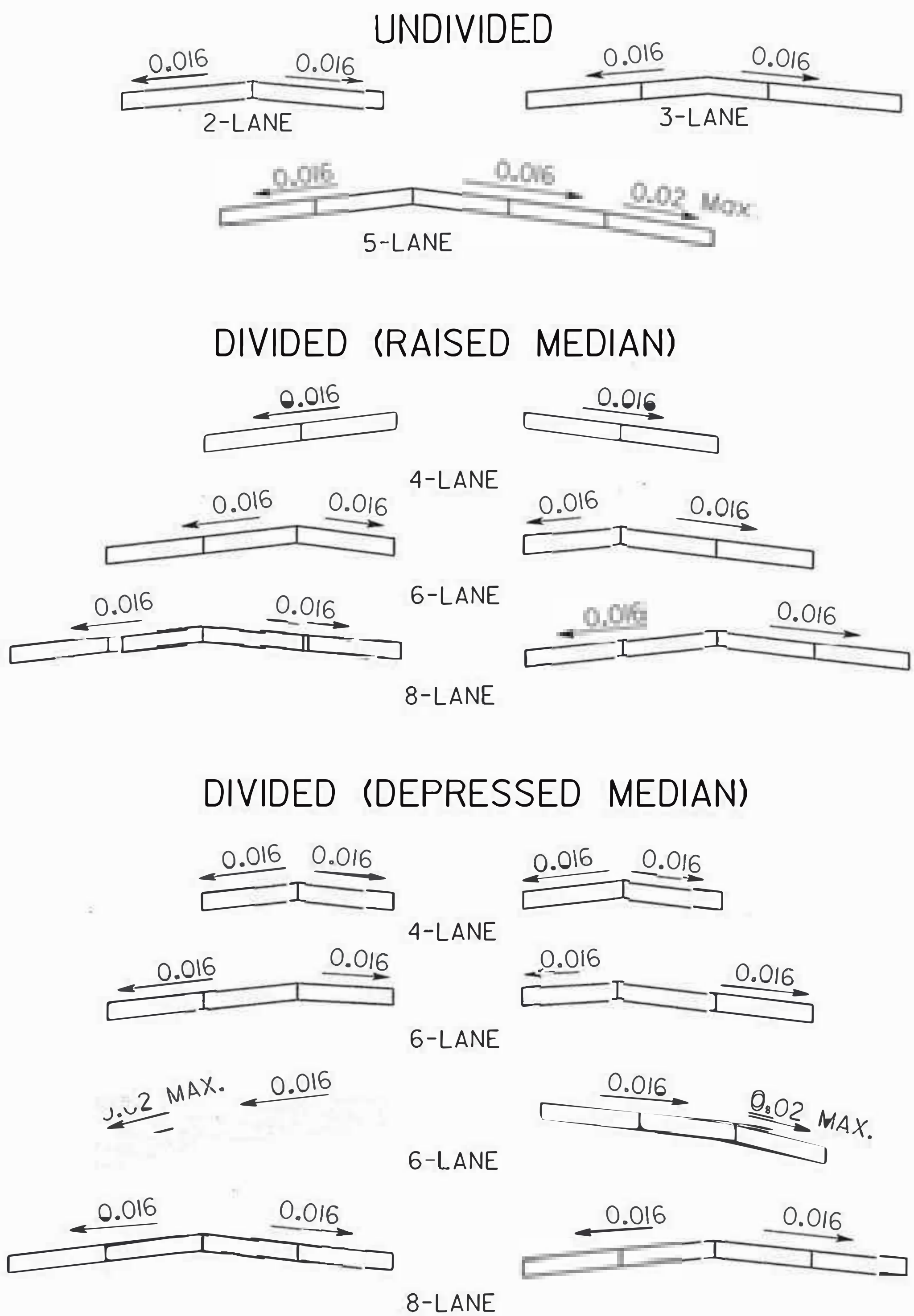
D_C	RADIUS	TRAVELED WAY WIDTH ON TANGENT (ft.)												
		24 ft.					22 ft.				20 ft.			
		Design Speed (mph)					Design Speed (mph)				Design Speed (mph)			
		30 to 39	40 to 49	50 to 59	60 to 69	≥70	30 to 39	40 to 49	50 to 59	≥60	30 to 39	40 to 49	50 to 59	≥60
1º00'	5730'	0	0	0	0	0	1.0	1.0	1.0	1.0	1.5	2.0	2.0	2.0
2º00'	2865'	0	1.0	1.0	1.0	1.0	1.5	1.5	1.5	1.5	2.5	3.0	3.0	3.0
3º00'	1910'	1.0	1.5	1.5	1.5	1.5	2.0	2.5	2.5	2.5	3.0	3.5	3.5	3.5
4º00'	1432'	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0	3.0	3.5	3.5	4.0	4.0
5º00'	1146'	2.0	2.0	2.5	3.0		3.0	3.5	3.5	3.5	4.0	4.0	4.5	4.5
6º00'	955'	2.5	2.5	3.0	3.5		4.0	4.0	4.5	4.5	4.5	5.0	5.5	*5.5*
7º00'	819'	3.0	3.0	4.0			4.0	4.5	5.0		5.0	5.5	6.0	
8º00'	716'	3.5	4.0	4.0			4.5	5.0	5.0		5.5	6.0	6.0	
9º00'	637'	4.0	4.0	4.5			5.0	5.0	5.5		6.0	6.0	6.5	
10º00'	573'	4.0	4.5				5.0	5.5			6.5	6.5		
11º00'	521'	5.0	5.5				6.0	6.5			7.0	7.5		
12º00	477'	5.5	5.5				6.5	6.5			7.5	7.5		
13º00'	441'	5.5	5.5				6.5	6.5			7.5	7.5		
14º00'	409'	6.0	6.0				7.0	7.0			8.0	8.0		
14º30'	395'	6.0	6.5				7.0	7.0			8.0	8.5		
15º00'	382'	6.5					7.5				9.0			
18º00'	318'	7.5					9.0				9.5			
19º00'	300'	8.0					9.0				10.5			
21º00'	265'	9.0					10.0				11.0			
22º00'	260'	9.0					10.0				11.0			
25º00'	229'	10.5					11.5				12.5			
26º00'	223'	10.5					11.5				12.5			
26º30'	219'	11.0					12.0				13.0			

NOTE:
Values are for two-lane highways, one-way or two-way. Values less than 1.0 ft. per lane may be disregarded. Multiply table values by 1.5 for 3-lanes and by 2.0 for 4-lanes.
For traveled way widening for other design vehicles, refer to the AASHTO “Green Book”.

NORMAL CROSS SLOPE ARRANGEMENTS

301-6

REFERENCE SECTIONS
301.1.5



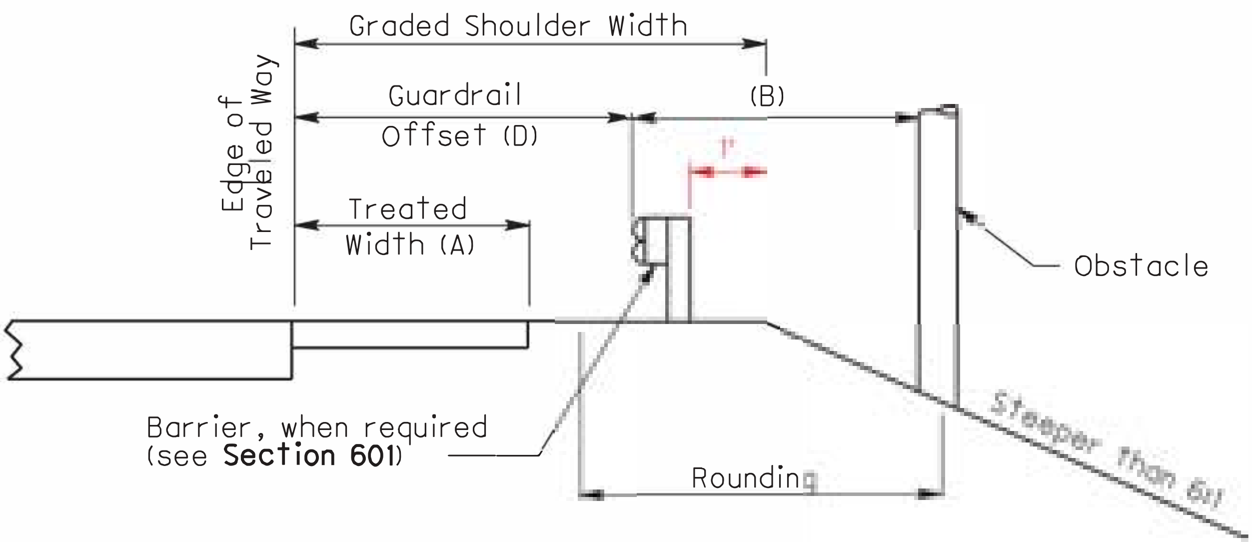
Note: All grade breaks should not exceed 0.032.

GRADED AND TREATED SHOULDER WIDTHS

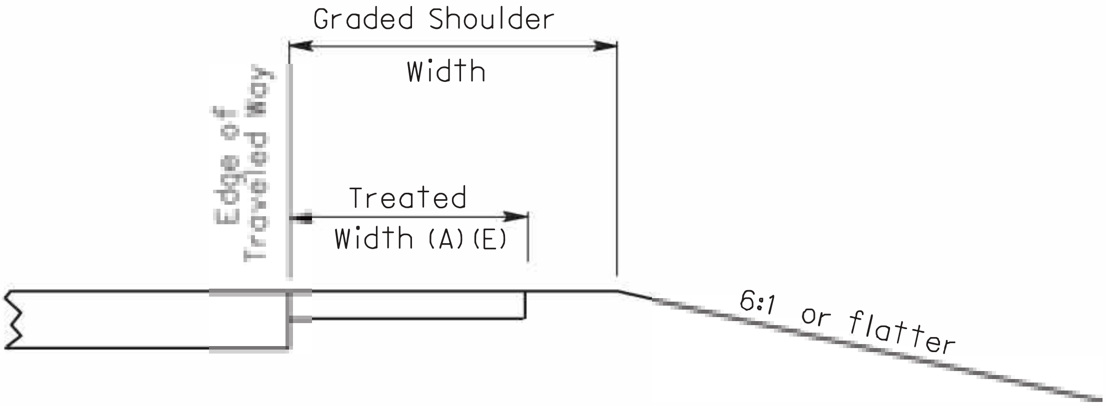
301-7

REFERENCE SECTIONS 301.2.1

See **Figure 301-3** for recommended dimensions



WITH BARRIER OR FORESLOPE STEEPER THAN 6:1



WITHOUT BARRIER AND FORESLOPE 6:1 OR FLATTER

NOTES:

(A) The "Treated Width" is that portion of the shoulder improved with stabilized aggregate or better.

(B) See **Figure 603-2** for minimum barrier clearance.

(D) Concrete barrier may be placed at the edge of treated shoulder when used in lieu of guardrail.

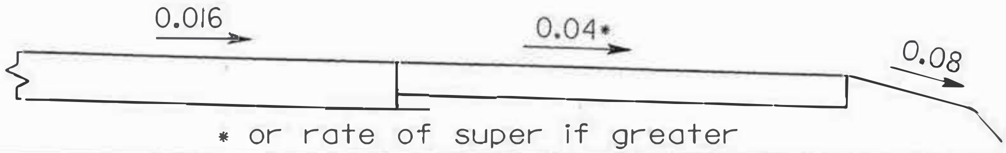
(E) Treated shoulder width may equal graded shoulder width in some cases.

PAVED SHOULDER CROSS SLOPES

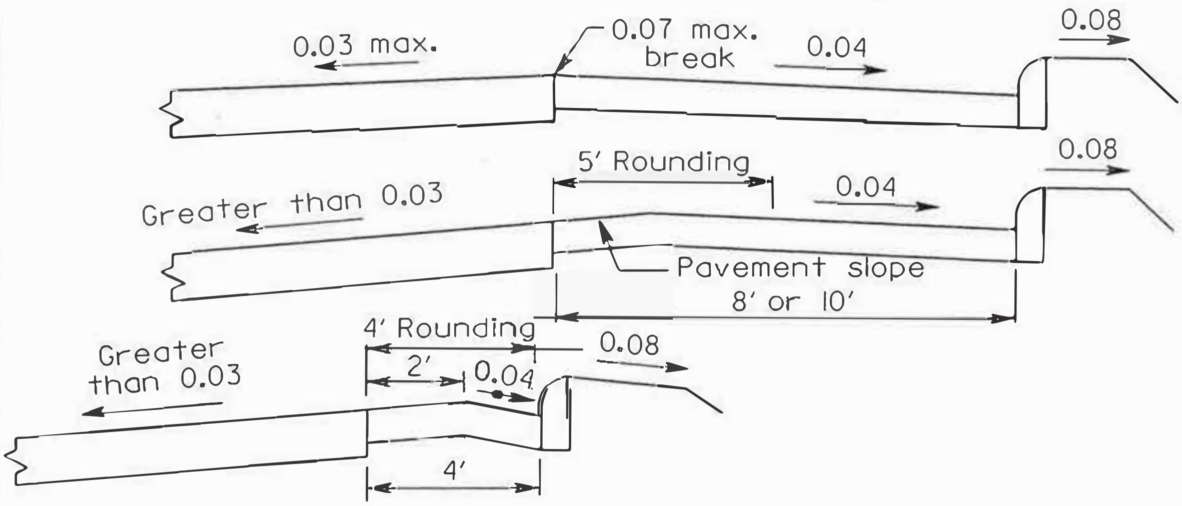
301-8

REFERENCE SECTIONS
301.2.4

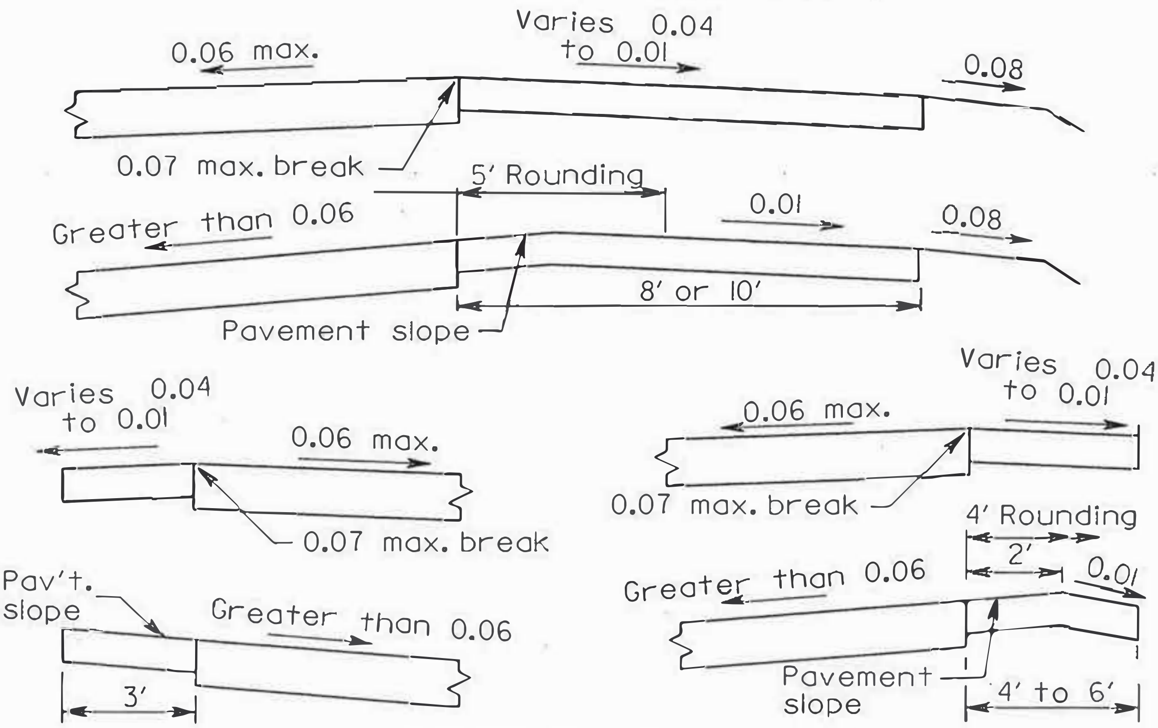
NORMAL AND LOW SIDE OF SUPERELEVATED SECTIONS



CURBED-HIGH SIDE OF SUPERELEVATED SECTIONS



UNCURBED-HIGH SIDE OF SUPERELEVATED SECTIONS

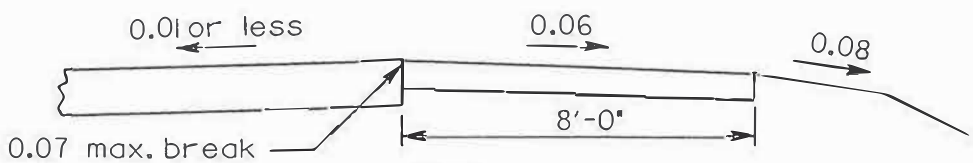


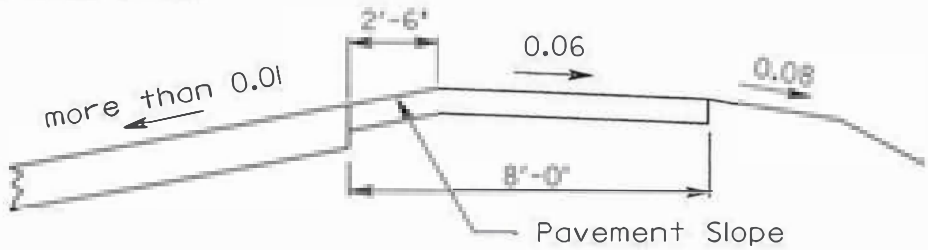
BITUMINOUS SURFACE TREATED OR STABILIZED AGGREGATED SHOULDER CROSS SLOPES	301-9
	REFERENCE SECTIONS 301.2.4

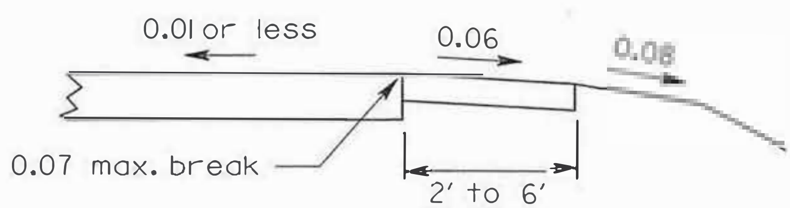
NORMAL AND LOW SIDE OF SUPERELEVATED SECTIONS

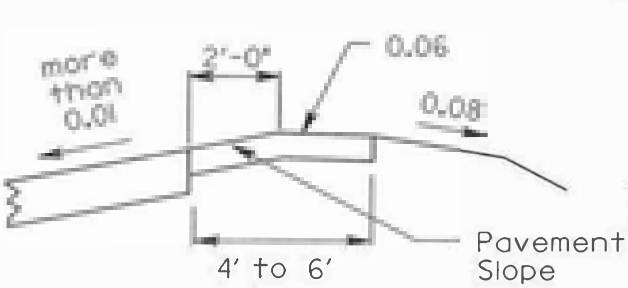
* or rate of super if greater

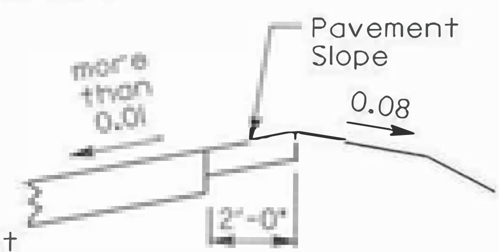
HIGH SIDE OF SUPERELEVATED SECTIONS







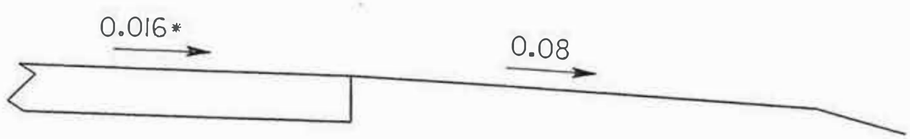




TURF SHOULDER CROSS SLOPES

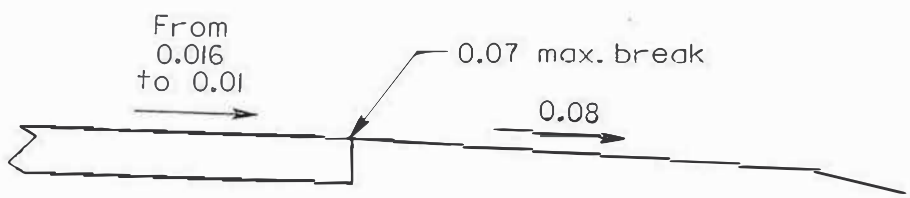
REFERENCE SECTIONS
301.2.4

NORMAL AND LOW SIDE (INNER SIDE) SUPERELEVATED SECTIONS

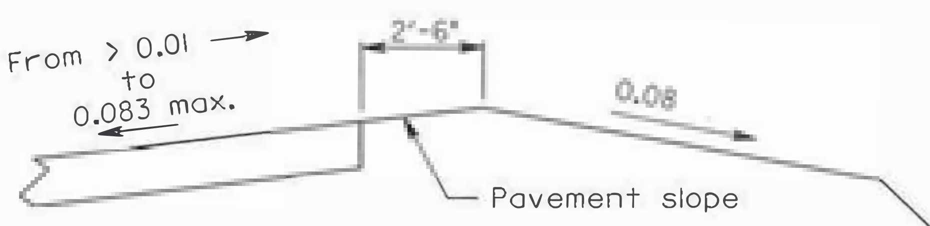


* or rate of super

RISING SIDE (OUTER SIDE) OF SUPERELEVATED SECTIONS IN TRANSITION



HIGH SIDE OF SUPERELEVATED SECTIONS



The break at the edge of the traveled way shall not exceed 0.07.

DESIGN CRITERIA NEW AND RECONSTRUCTED (J) BRIDGES

302-1

REFERENCE SECTIONS
302.1

Functional Classification	Traffic	Lateral Clearance			Vertical Clearance Over Roadway (H)
	Design Year ADT	On Bridge (A)		Under Bridge (F)	
		Rural	Urban		
		Min.	Min.	Minimum	Minimum
Interstates, Other Freeways & Expressways	All	10' Rt. (B)(D) 4' Lt. (E)(B)	For curbed shoulders, use shoulder widths from Figure 301-4 For uncurbed shoulders, use rural criteria at left	Curbed or treated shoulder widths, see Figures 301-3 & 301-4 plus barrier clearance from Figure 603-2	16.5' (I)
Arterial	> 2000	8' (B)		(G)	16.5' (I)
	1501 – 2000	6' (B)			
	400 – 1500	6' (B)			
	< 400	4'			
Collector	> 2000	4'		(G)	14.5'
	1501 – 2000	4'			
	400 – 1500	4'			
	< 400	(C)			
Local	> 2000	4'		(G)	14.5'
	1501-2000	4'			
	400 – 1500	4'			
	< 400	(C)			

SEE THE FOLLOWING SHEET FOR CORRESPONDING NOTES.

For structure design criteria not contained in this table such as minimum design loading, refer to the Bridge Design Manual from the Office of Structural Engineering.

WHERE THE APPROACH ROADWAY WIDTH (TRAVELED WAY PLUS SHOULDERS) IS SURFACED, AT A MINIMUM, THAT SURFACE WIDTH SHOULD BE CARRIED ACROSS THE STRUCTURE.

Notes to Figure 302-1: Design Criteria - New & Reconstructed Bridges

A. Lateral Clearance is the distance measured from the edge of the traveled lane to the face of curb (or railing if no curb is present).

B. If bridge is considered to be a major structure having a length of 200 ft. or more, the width may be reduced, subject to economic studies, but not less than a lateral clearance of 4 ft.

C. See AASHTO's Guidelines for Geometric Design for Very Low-Volume Local Roads (ADT ≤ 400) for values.

D. Where truck traffic exceeds 250 DDHV, additional shoulder width may be beneficial.

E. If 6 or more lanes, provide 10 ft. width.

F. Distance measured from the edge of the traveled lane to the face of walls of abutments and piers.

G. May be reduced to a clearance of 2 ft. plus barrier clearance (Figure 603-2) on urban streets with restricted right-of-way and a design speed less than 50 mph.

H. The minimum vertical clearance includes an allowance for future resurfacing equal to 0.5 ft. Sign supports and pedestrian structures shall have a 1 ft. additional clearance. Clearances shown shall be over paved shoulder as well as traveled way width.

I. A 15.5 ft. minimum clearance may be used in highly developed urban areas if attainment of 16.5 ft. clearance would be unreasonably costly **and** if there is an alternate freeway route or bypass which provides a minimum 16.5 ft. vertical clearance.

J. A reconstructed bridge is any improvement to an existing bridge involving the replacement of the bridge deck or more.

CRITERIA FOR EXISTING INTERSTATE AND OTHER FREEWAY BRIDGES TO REMAIN

302-2

REFERENCE SECTIONS 302.1

Functional Classification	Design Year ADT	Minimum Lateral Clearance		Minimum Vertical Clearance (E)
		On Bridge (A)	Under Bridge (C)	
Urban Interstate	All	10' Rt. (B) 3.5' Lt.	Curbed or Treated Shoulder Width Plus Barrier Clearance (D)	14.5' (F)
Rural Interstate	All	10' Rt. (B) 3.5' Lt.		16.0'
Other Freeways	All	10' Rt. (B) 3.5' Lt.		14.5'

This table is applicable to all bridges except those classified as new or reconstructed. (See **Figure 302-1**.)

For structural criteria not contained in this table, including Minimum Design Loading, see **Structural Engineering's Bridge Design Manual**.

NOTES:

(A) Distance measured to curb or railing, whichever is less, in no case shall the minimum width be less than the approach roadway (traveled way plus shoulders).

(B) On mainline bridges that are 200 ft. long or longer, the minimum may be reduced to 3.5 ft. for Interstate and 3 ft. for other freeways.

(C) Distance measured to face of walls, abutments or piers.

(D) See **Figure 603-2** for minimum barrier clearance.

(E) Includes height over shoulders

(F) Minimum vertical clearance is 16 ft. if there is no alternative Interstate routing with the minimum 16 ft. vertical clearance.

CRITERIA FOR EXISTING NON-FREEWAY BRIDGES TO REMAIN

302-3

REFERENCE SECTIONS 302.1

Functional Classification	Design Year ADT	Minimum Lateral Clearance (A)		Minimum Vertical Clearance
		On Bridge (B)	Under Bridge (E)	
Expressways and Arterials	> 4000	6 ft. (C)	Curbed or Treated Shoulder Width Plus Barrier Clearance (F)	14 ft.
	≤ 4000	3 ft.		
Collector	> 4000	6 ft. (C)		14 ft.
	2001-4000	3 ft.		
	1001-2000	2 ft.		
	400-1000	2 ft.		
	< 400	0		
Local	> 4000	6 ft. (C)		14 ft.
	2001-4000	3 ft.		
	1001-2000	2 ft.		
	400-1000	2 ft. (D)		
	< 400	0 (D)		

This table is applicable to all non-freeway bridges except those classified as new or reconstructed.

For structural criteria not contained in this table, including Minimum Design Loading, see **Structural Engineering's Bridge Design Manual**.

NOTES:

(A) Divided facilities shall have a minimum of 3 ft. lateral clearance on the median side.
(B) Distance measured to curb or railing, whichever is less. In no case shall the minimum width be less than the approach roadway (traveled way plus shoulders).
(C) On mainline bridges having a length of 100 ft. or more, the minimum may be reduced to 3 ft.
(D) One lane bridges have a total minimum width of 18 ft.
(E) Distance measured to face of walls, abutments or piers.
(F) See **Figure 603-2** for minimum barrier clearance.

Note: For the design criteria pertaining to Collectors and Local Roads with ADT's of 400 or less, refer to the AASHTO Publication - Guidelines for Geometric Design of Very Low-Volume Local Roads ADT≤400).

INTERCHANGE ELEMENTS -TRAVELED WAY, SHOULDERS AND MEDIANS

303-1

REFERENCE SECTIONS
303.1

INTERCHANGE ELEMENTS	TOTAL TRAVELED WAY WIDTH	Graded Shoulder Width				Paved Shoulder Width		Normal Rounding (E)	Guardrail Offset (From Traveled Way) (G)	
		Left		Right						
		With Barrier or Foreslope steeper than 6:1	w/o Barrier slopes 6:1 or flatter	With Barrier or Foreslope steeper than 6:1	w/o Barrier slopes 6:1 or flatter	LT	RT		LT	RT
Ramp	16' (A)	9' (C)	6'	11' (C)	8'	3'	6'	10'	6'	8'
1-Lane Directional Roadway	16' (A)	9' (C)	6'	11' (C)	8'	4'	6'	10'	6'	8'
2-Lane Directional Roadway or Multilane Ramps	Var. (B)	9' (C)(H)	6' (H)	15' (D)	10' (D)	4' (H)	10'	10'	6' (H)	12' (D)
Accel/Decel Lane or Combined	Var.	NA	NA	13' (D)(F)	8' (D)(F)	NA	8' (F)	10'	NA	10' (D)(F)

NOTES:

(A) Use 18 ft. when inside traveled way edge radius is less than 200 ft.

(B) For 2-lane directional roadways and 2-lane multilane ramps, the traveled way width shall be 24 ft.

(C) May be reduced 1 ft. if the face of the mainline barrier is 2 ft. from the outside edge of the graded shoulder.

(D) Or match mainline dimension if lesser.

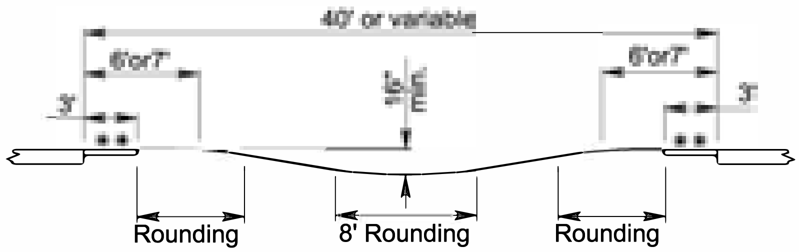
(E) Rounding is 4 ft. when barrier is used. No rounding is required when foreslope is 6:1 or flatter.

(F) Match Multilane Ramp dimensions when used with Multilane Ramps.

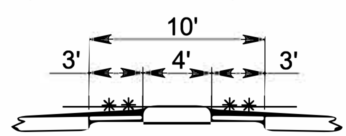
(G) Concrete barrier may be placed at the edge of the paved shoulder when used in lieu of guardrail, but no closer than 4'.

(H) For 3 or more lanes, use right side widths or dimensions.

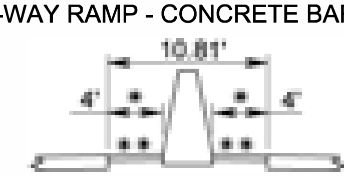
TWO-WAY RAMP MEDIAN



MINIMUM TWO-WAY RAMP - CONCRETE MEDIAN



MINIMUM TWO-WAY RAMP - CONCRETE BARRIER MEDIAN



* Check horizontal stopping sight distance

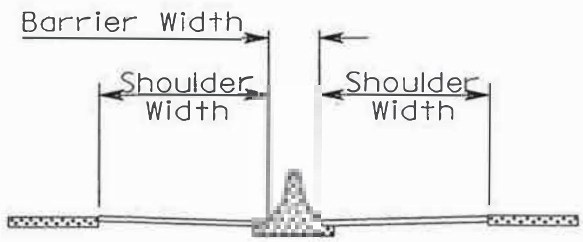
** See Figure 301-8 for shoulder cross slope

TYPICAL MEDIAN DESIGNS

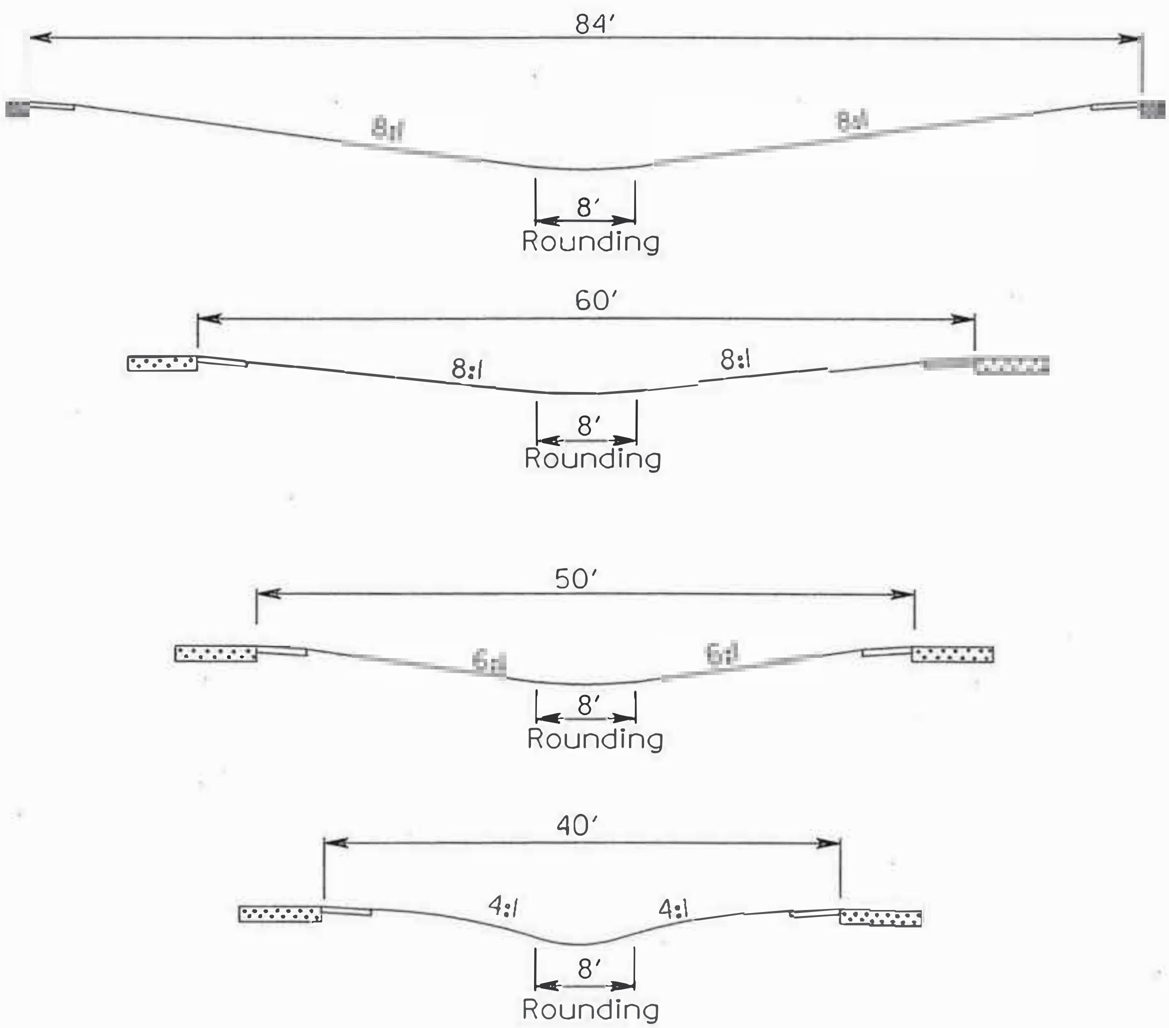
304-1

REFERENCE SECTIONS
304.3

BARRIER MEDIAN



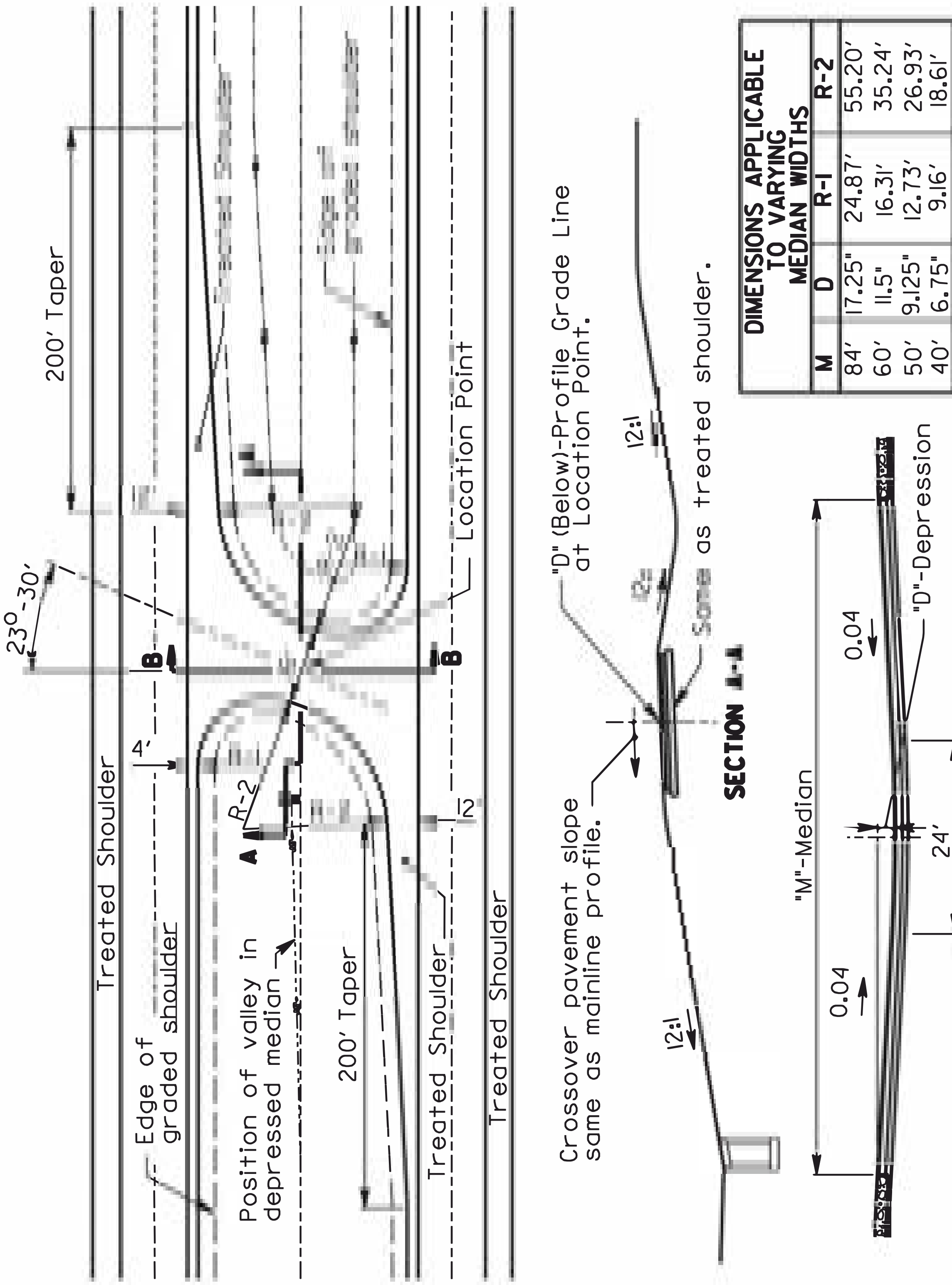
DEPRESSED MEDIANS



November 2002

STANDARD U-TURN MEDIAN OPENINGS

304-2
REFERENCE SECTIONS
304.4.3



DIMENSIONS APPLICABLE TO VARYING MEDIAN WIDTHS

M	D	R-1	R-2
84'	17.25"	24.87'	55.20'
60'	11.5"	16.31'	35.24'
50'	9.125"	12.73'	26.93'
40'	6.75"	9.16'	18.61'

The values shown do not reflect values needed for CAD accuracy

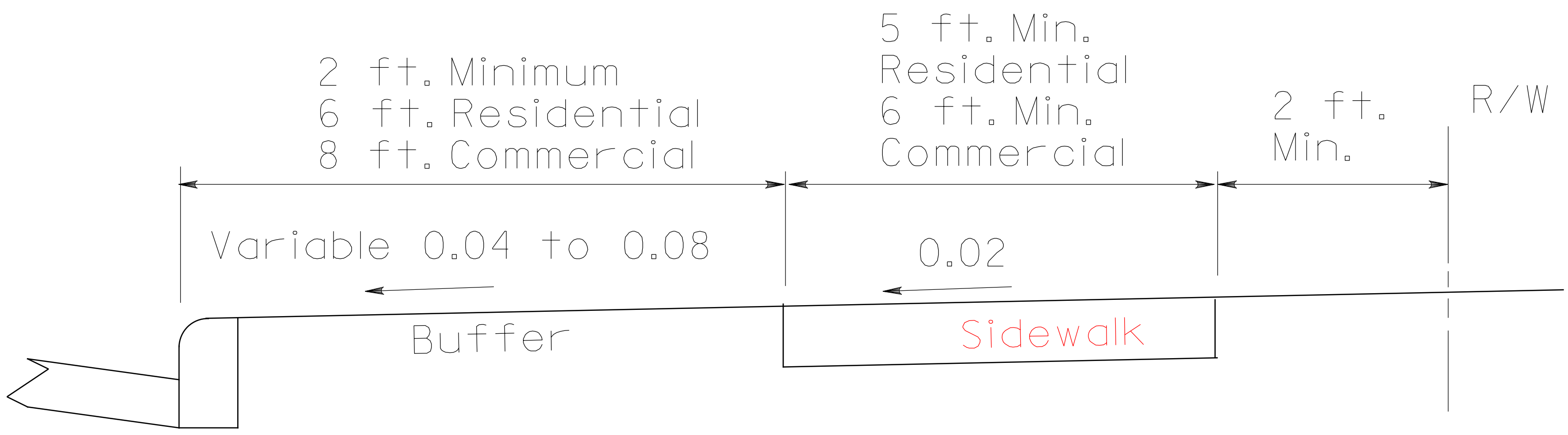
GUIDELINES FOR NEW SIDEWALK /WALKWAY INSTALLATIONS	306-1 REFERENCE SECTIONS 306.2.1

ROADWAY CLASSIFICATION & LAND USE	SIDEWALK/WALKWAY	FUTURE PHASING
Rural Highways (<2,000 ADT)	See AASHTO's A Policy on Geometric Design of Highway and Streets for combined traveled way and shoulder widths for local roads, collectors and arterials	----
Rural/suburban highways (ADT>2,000 and less than 1 dwelling unit per acre)	Minimum 8 ft. shoulders recommended	Secure/preserve ROW for future sidewalks
Suburban Highway (1 to 4 dwelling units per acre)	Sidewalks on both sides recommended	----
Local Urban Street (Residential - 1 to 4 dwelling units per acre)	Sidewalks on both sides preferred, min. of 8 ft. shoulders recommended	Secure/preserve ROW for future sidewalks
Local Urban Street (Residential - 1 to 4 dwelling units per acre)	Sidewalks on both sides recommended	----
Local Urban Street (Residential - more than 4 dwelling units per acre)	Sidewalks on both sides recommended	----
Urban Collector and Minor Arterial (residential)	Sidewalks on both sides recommended	----
Major Urban Arterial (residential)	Sidewalks on both sides recommended	----
All Commercial/Urban Streets	Sidewalks on both sides recommended	----
All Industrial Streets	Sidewalks on both sides preferred, sidewalk on one side and min. of 5 ft. shoulder recommended	----

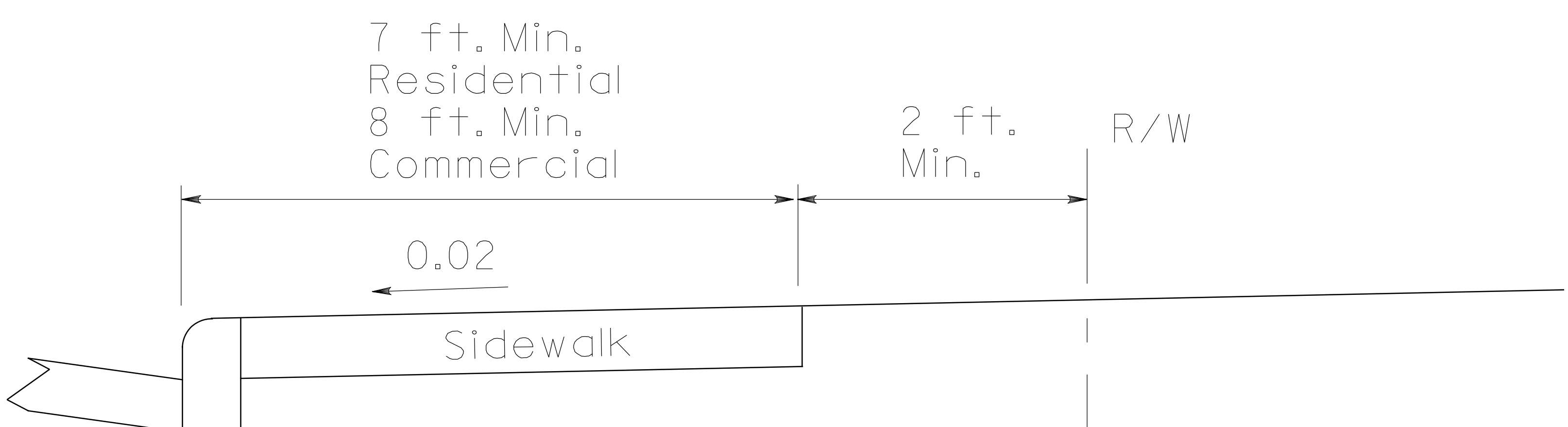
SIDEWALK DESIGNS

306-2E

REFERENCE SECTION
306.2.2, 306.2.4

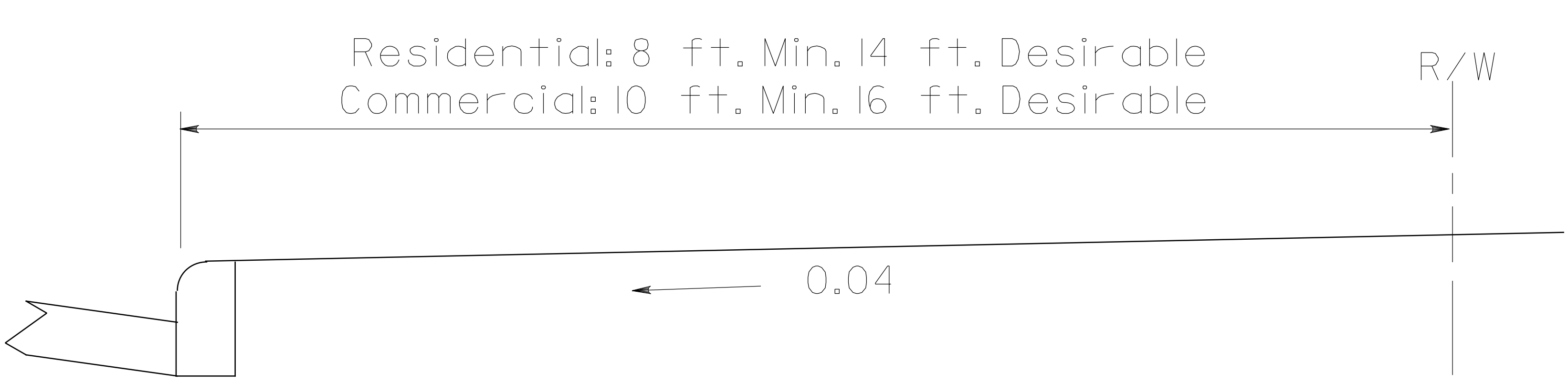


SIDEWALK WITH BUFFER



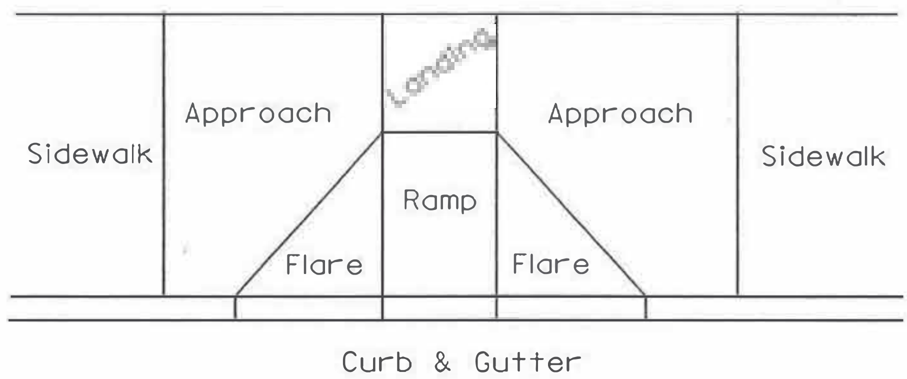
SIDEWALK-NO BUFFER

SIDEWALK-CENTRAL BUSINESS DISTRICT

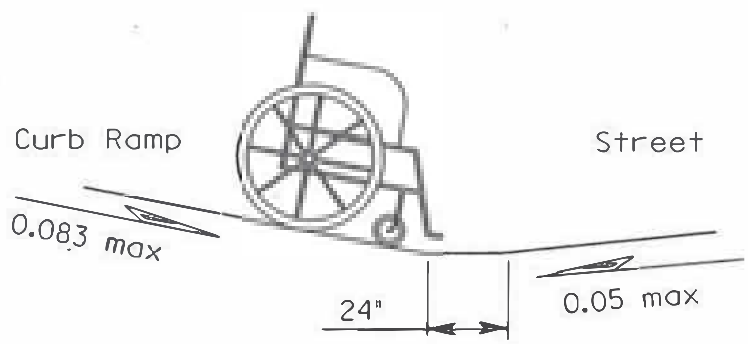


BORDER-NO SIDEWALK

CURB RAMP COMPONENTS	306-3
	REFERENCE SECTION 306.3.3



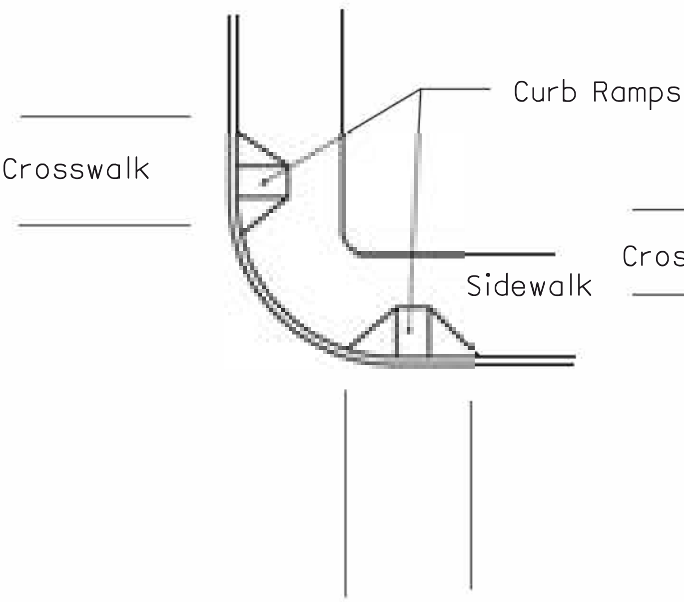
CURB RAMP TERMS



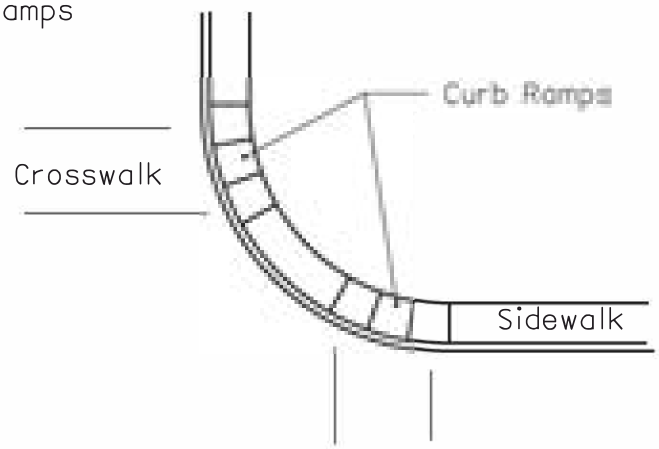
COUNTER SLOPE

November 2002

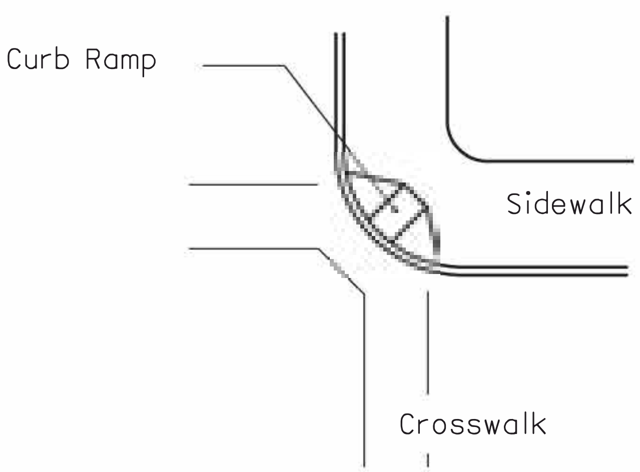
CURB RAMP TYPES AND PEDESTRIAN CURB CUTS	306-4
	REFERENCE SECTION 306.3.4



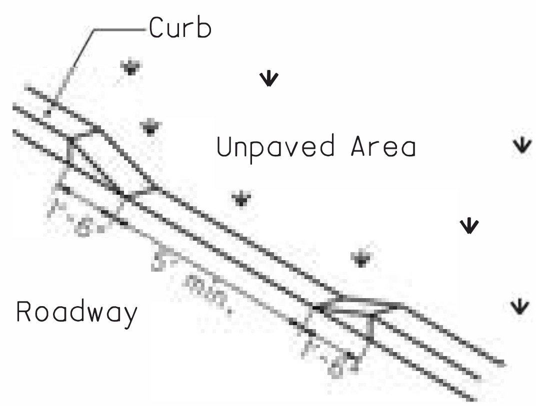
PERPENDICULAR



PARALLEL

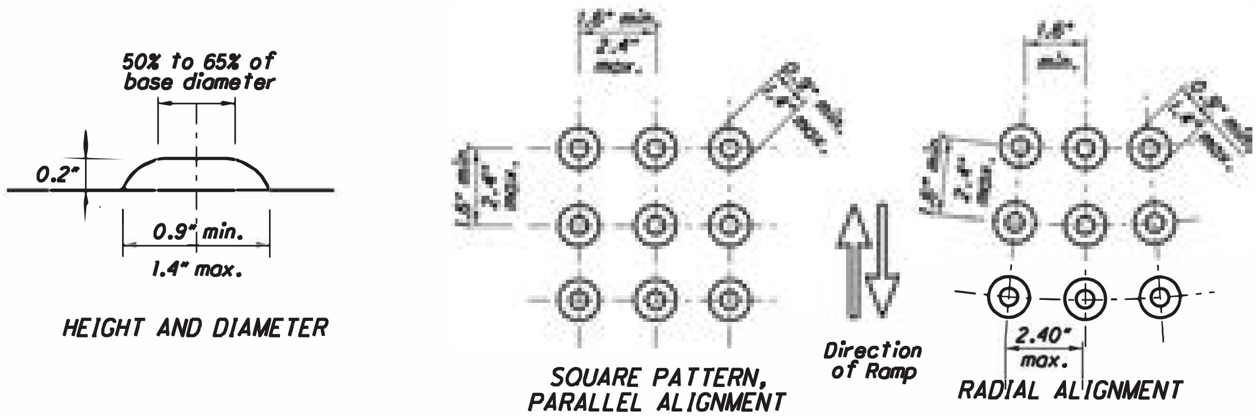


DIAGONAL

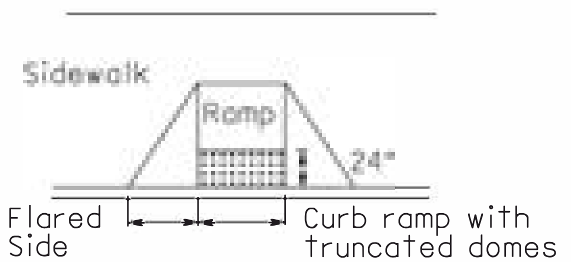


PEDESTRIAN CURB CUT

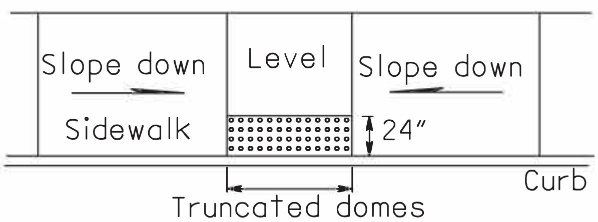
TRUNCATED DOMES	306-5 REFERENCE SECTION 306.3.5



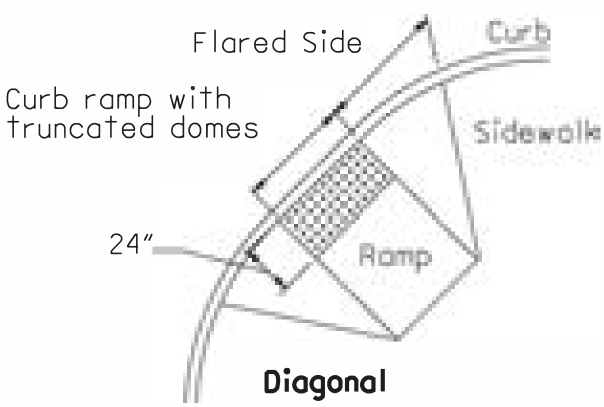
DOME HEIGHT AND SPACING



Perpendicular

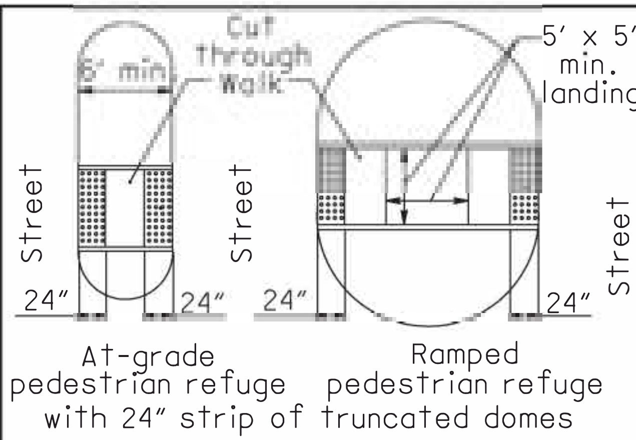


Parallel

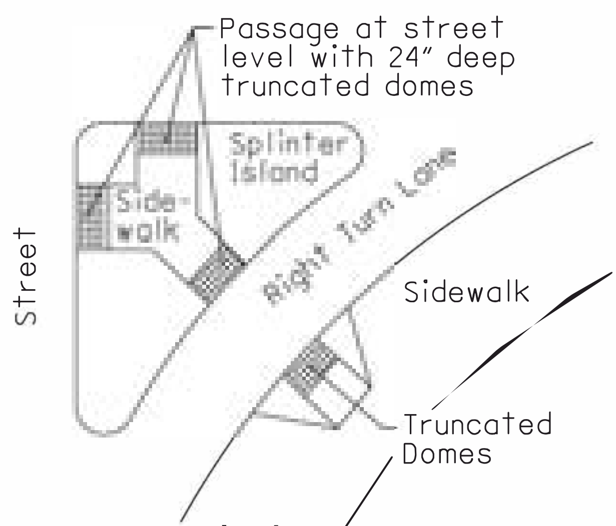


Diagonal

AT CURB RAMPS



Medians



Islands

AT MEDIANS AND ISLANDS

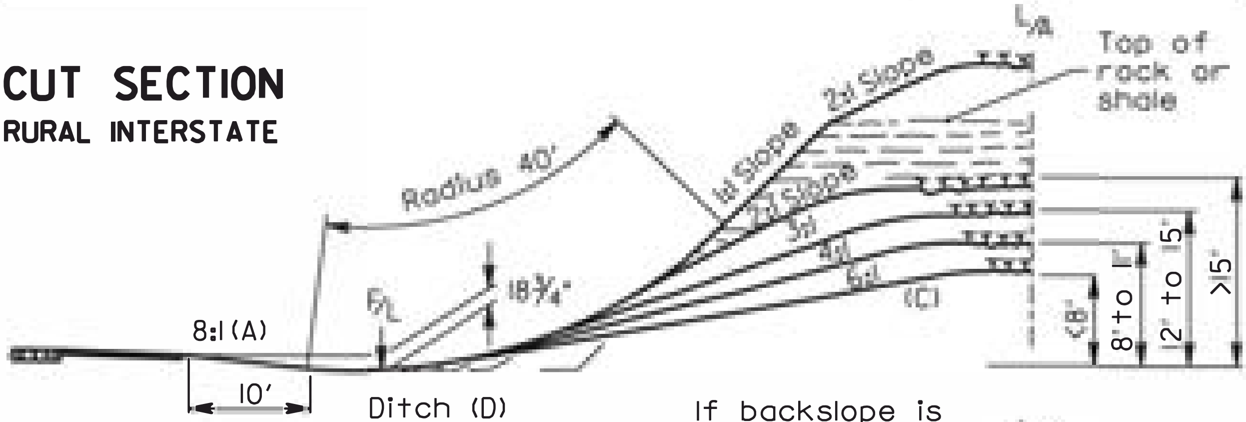
SAFETY GRADING SECTIONS

307-1

REFERENCE SECTIONS 307.2.1, 307.2.3

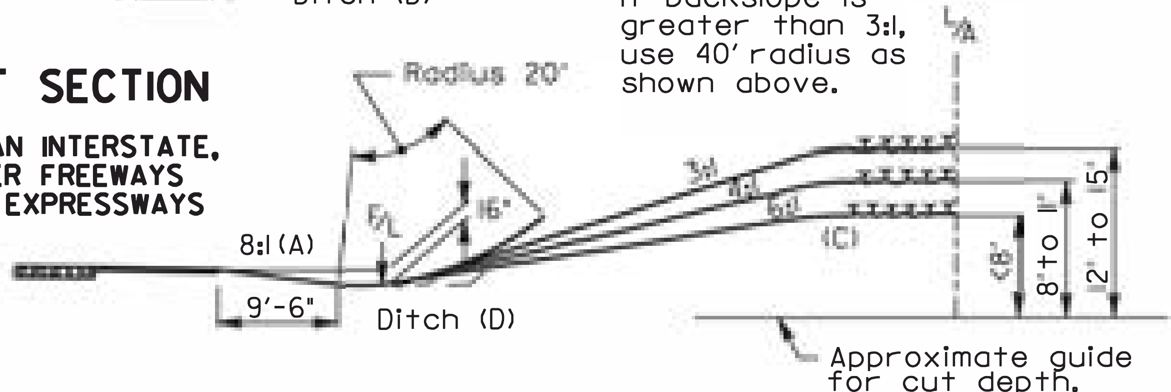
CUT SECTION

RURAL INTERSTATE



CUT SECTION

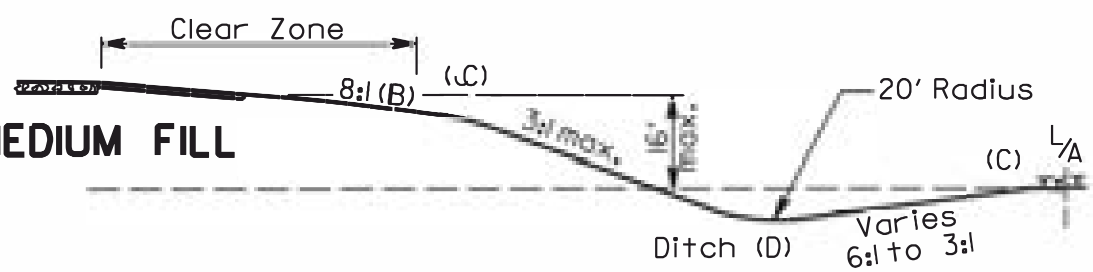
URBAN INTERSTATE, OTHER FREEWAYS AND EXPRESSWAYS



SHALLOW CUT OR LOW FILL

Slope transition between low fill design and medium fill design should be such that the flowline of the roadside ditch does not turn towards the roadway.

MEDIUM FILL



Application of these sections may vary to avoid frequent slope changes and to maintain reasonably straight ditches.

NOTES:

(A) 6:1 slope may be used with horizontal distance remaining the same to increase the depth.

(B) 6:1 slope may be used

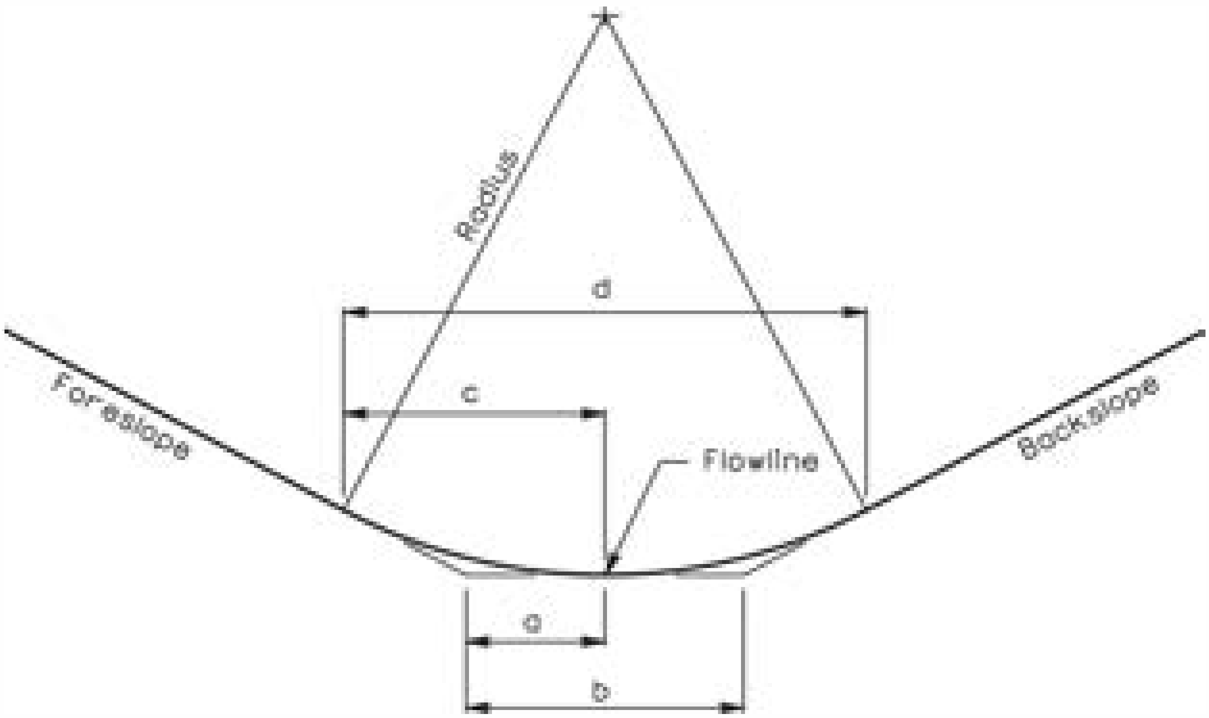
(C) 4' Rounding

(D) See **Fig. 307-2** for ditch sections to be used with safety grading

DITCH SECTIONS FOR SAFETY GRADING

307-2

REFERENCE SECTIONS
307.2.1



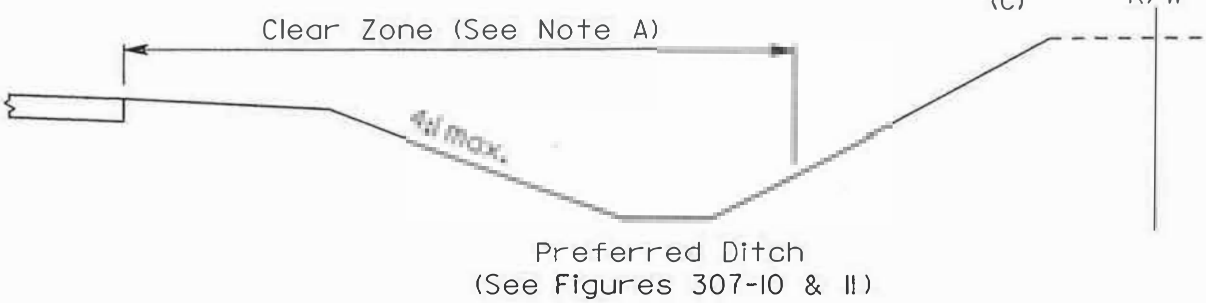
Fore-slope	40' RADIUS		Backslope									
			6:1		4:1		3:1		2:1		1:1	
	a	c	b	d	b	d	b	d	b	d	b	d
8:1	2'-6"	5'-0"	5'-10"	11'-6"	7'-5"	14'-8"	9'-0"	17'-7"	11'-11"	22'-10"	19'-0"	33'-3"
6:1	3'-4"	6'-7"	6'-7"	13'-2"	8'-3"	16'-3"	9'-10"	19'-3"	12'-9"	24'-6"	19'-11"	34'-10"

Fore-slope	20' RADIUS		Backslope					
			6:1		4:1		3:1	
	a	c	b	d	b	d	b	d
8:1	1'-3"	2'-6"	2'-11"	5'-9"	3'-8"	7'-4"	4'-6"	8'-10"
6:1	1'-8"	3'-3"	3'-4"	6'-7"	4'-1"	8'-2"	4'-11"	9'-7"
4:1	2'-6"	4'-10"	4'-1"	8'-2"	4'-11"	9'-8"		
3:1	3'-3"	6'-4"	4'-11"	9'-7"				

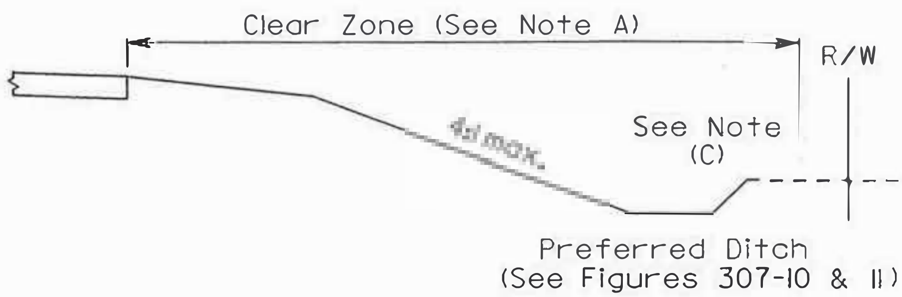
CLEAR ZONE GRADING SECTIONS

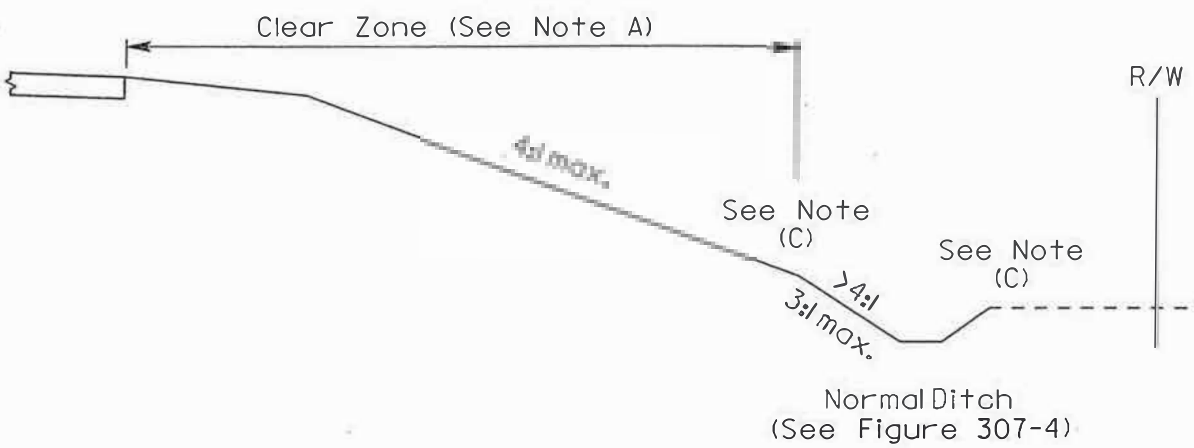
307-3
REFERENCE SECTIONS 307.2.1, 307.2.3

CUT SECTION



FILL SECTIONS (B)





NOTES:

(A) See Figure 600-1 for Clear Zone widths.

(B) For fill heights over 16 ft., use barrier grading (See Figure 307-4)

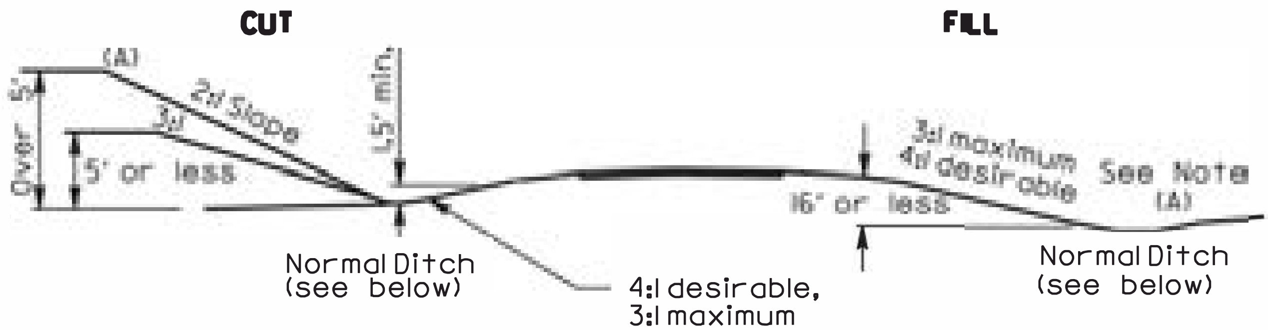
(C) 4 ft. Rounding

COMMON AND BARRIER GRADING SECTIONS

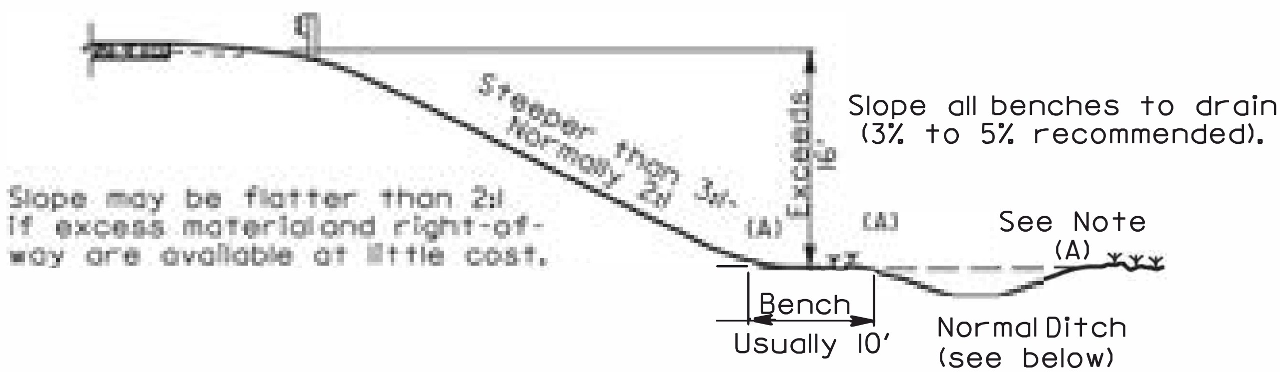
307-4

REFERENCE SECTIONS 307.2.1, 307.2.3

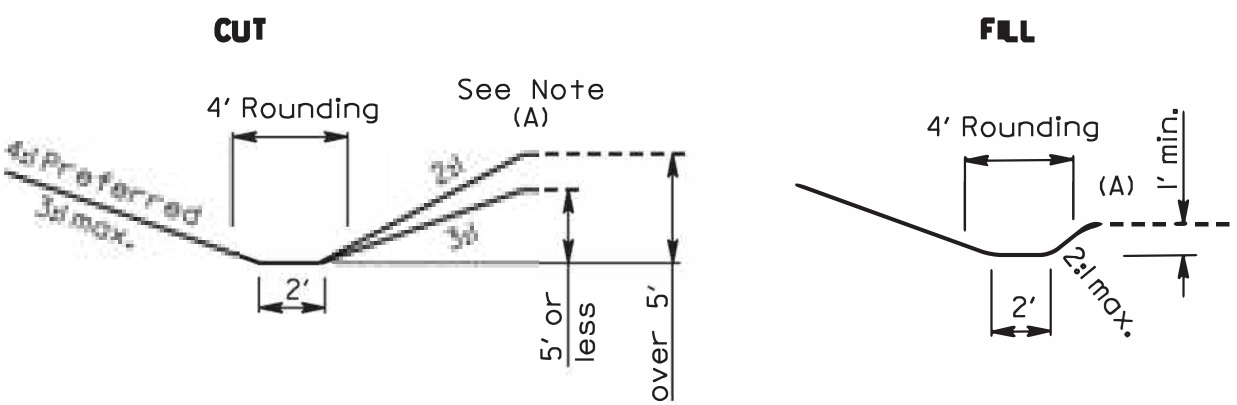
COMMON GRADING



BARRIER GRADING



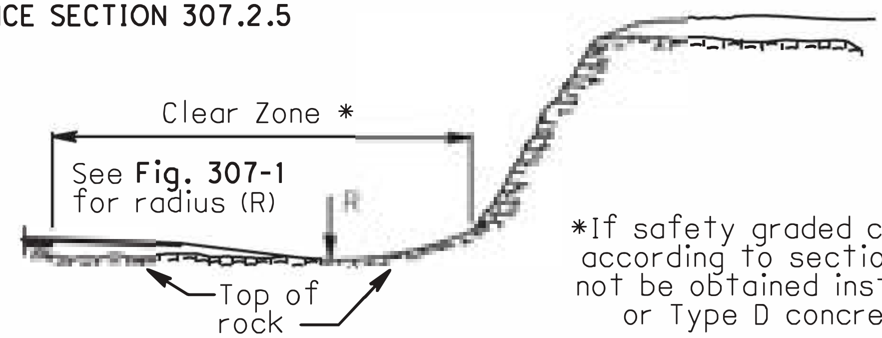
NORMAL DITCH SECTIONS



NOTES: (A) 4' Rounding

DESIGNS FOR ROCK CUTS WITH SAFETY GRADING

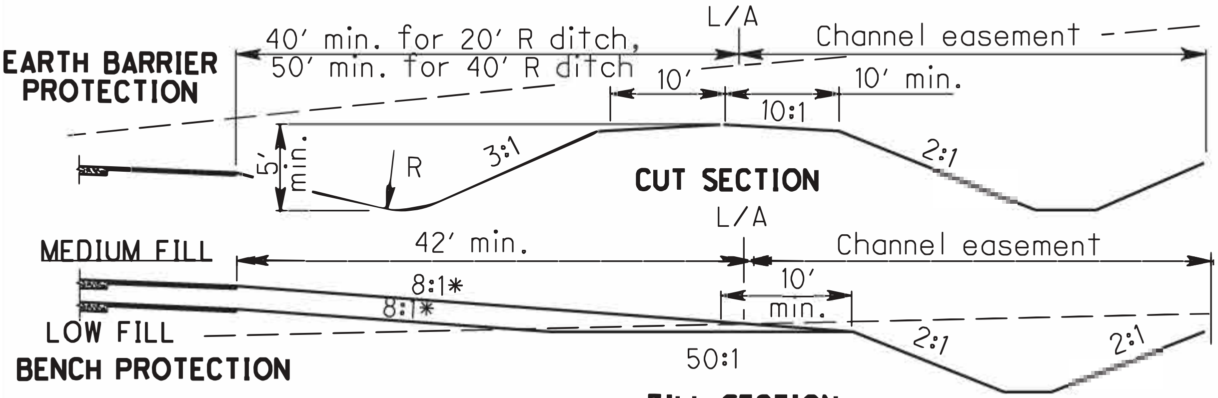
REFERENCE SECTION 307.2.5



*If safety graded cross sections according to section 307.2.1 can not be obtained install guardrail or Type D concrete barrier

DESIGNS FOR DEEP PARALLEL SIDE DITCHES

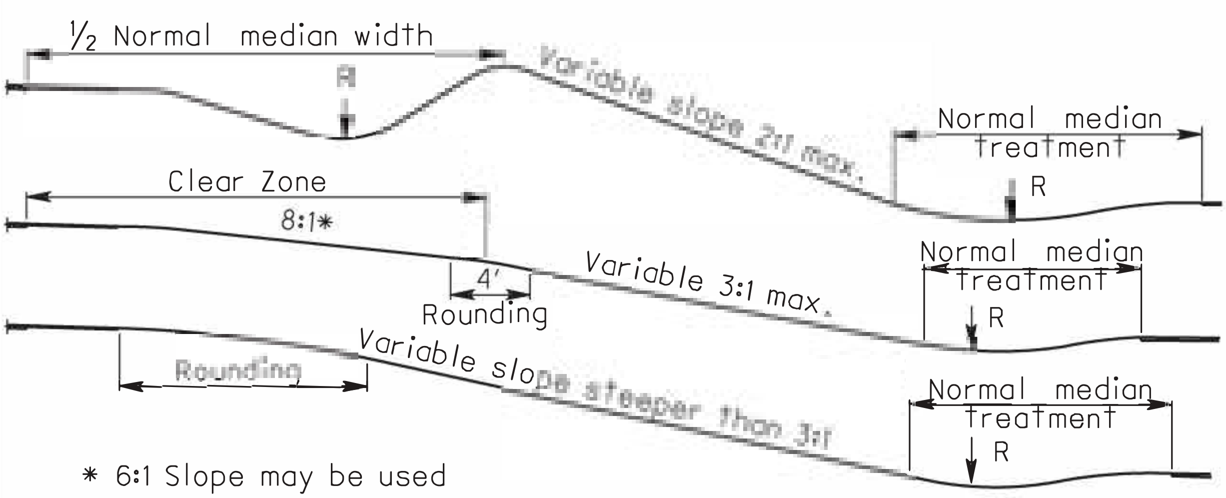
REFERENCE SECTIONS 307.3 & 307.4



*6:1 Slope may be used

ALTERNATE MEDIAN DESIGNS-SEPARATE PROFILES

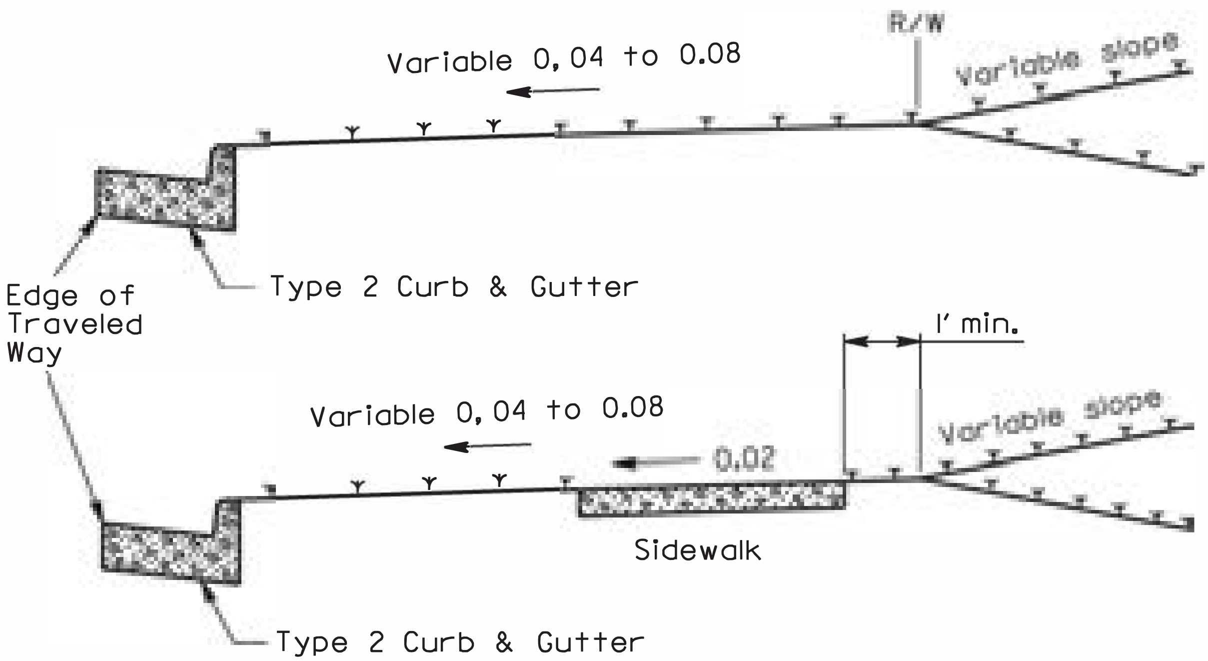
REFERENCE SECTION 307.2.4



* 6:1 Slope may be used

SLOPE TREATMENT ADJACENT TO CURBED STREETS

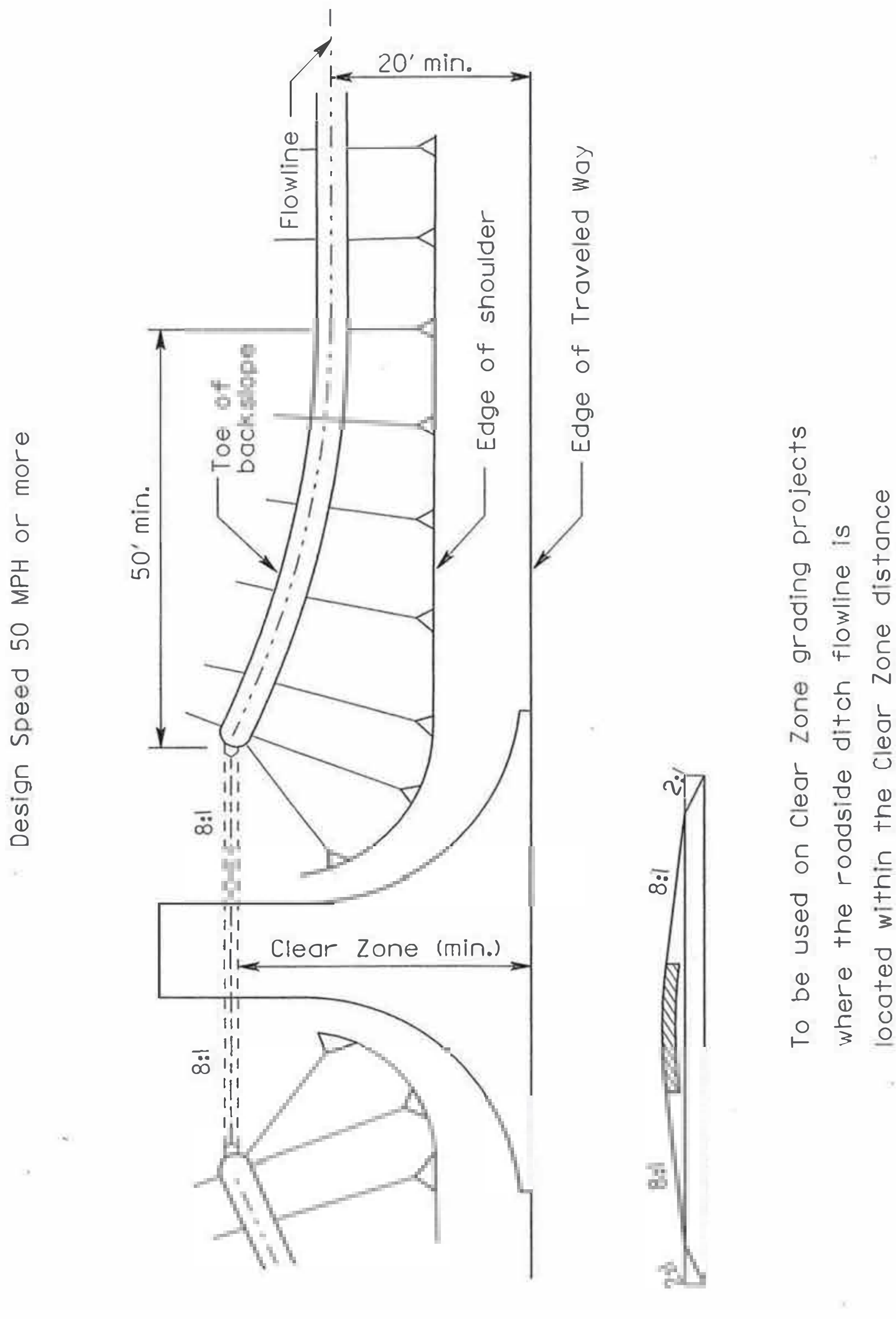
307-6
REFERENCE SECTIONS 307.2.6



The 4% to 8% slopes in the top detail were extended to the R/W line to prevent runoff from the right-of-way entering private property. This slope may be broken if the highway runoff can be maintained within the highway right-of-way.

SLOPES AND DITCHES AT DRIVEWAYS AND CROSSROADS IN CUT OR LOW FILL

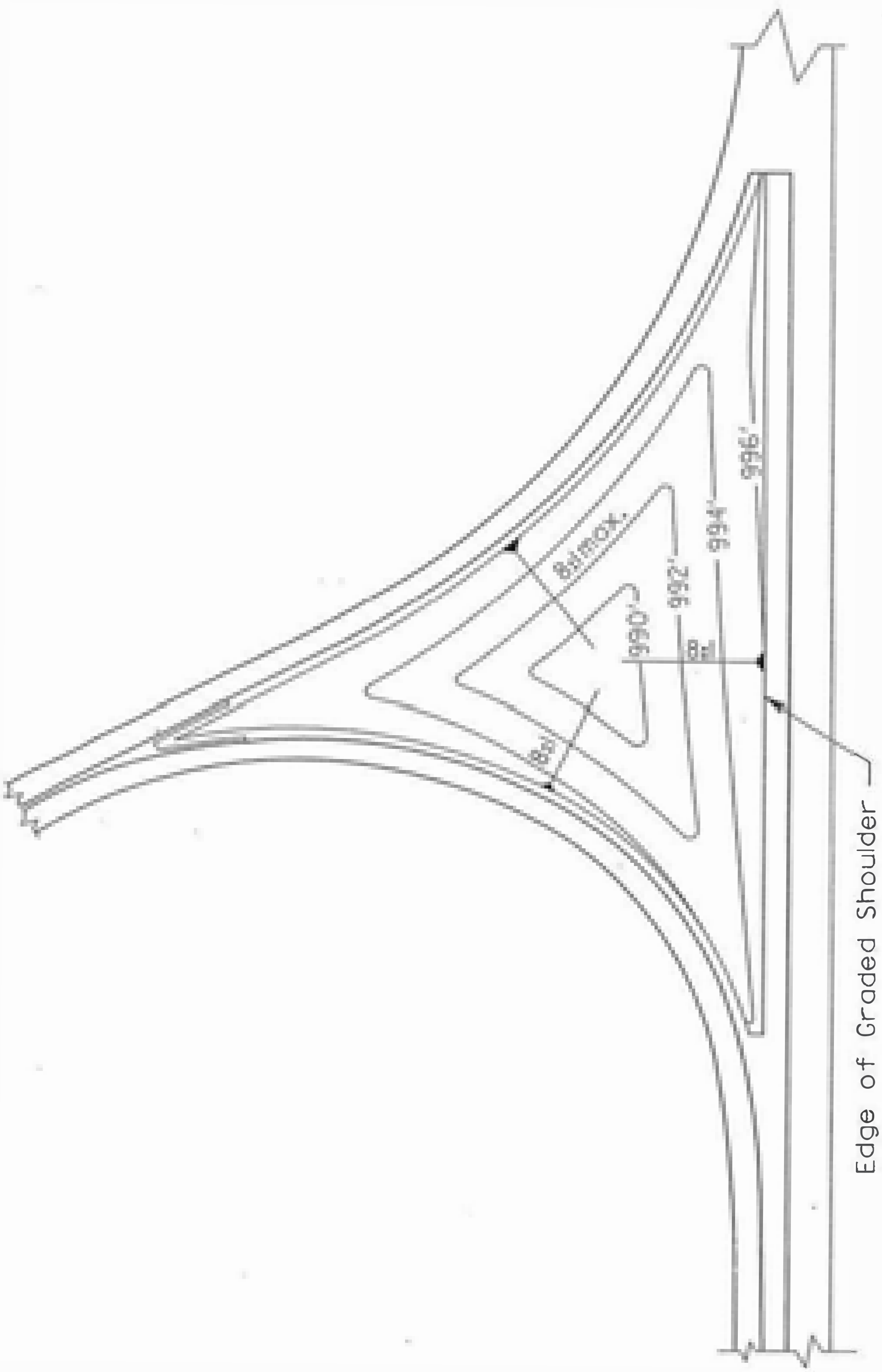
307-7
REFERENCE SECTIONS 307.2.7



CONTOUR GRADING OF TRUMPET INTERIORS

307-8

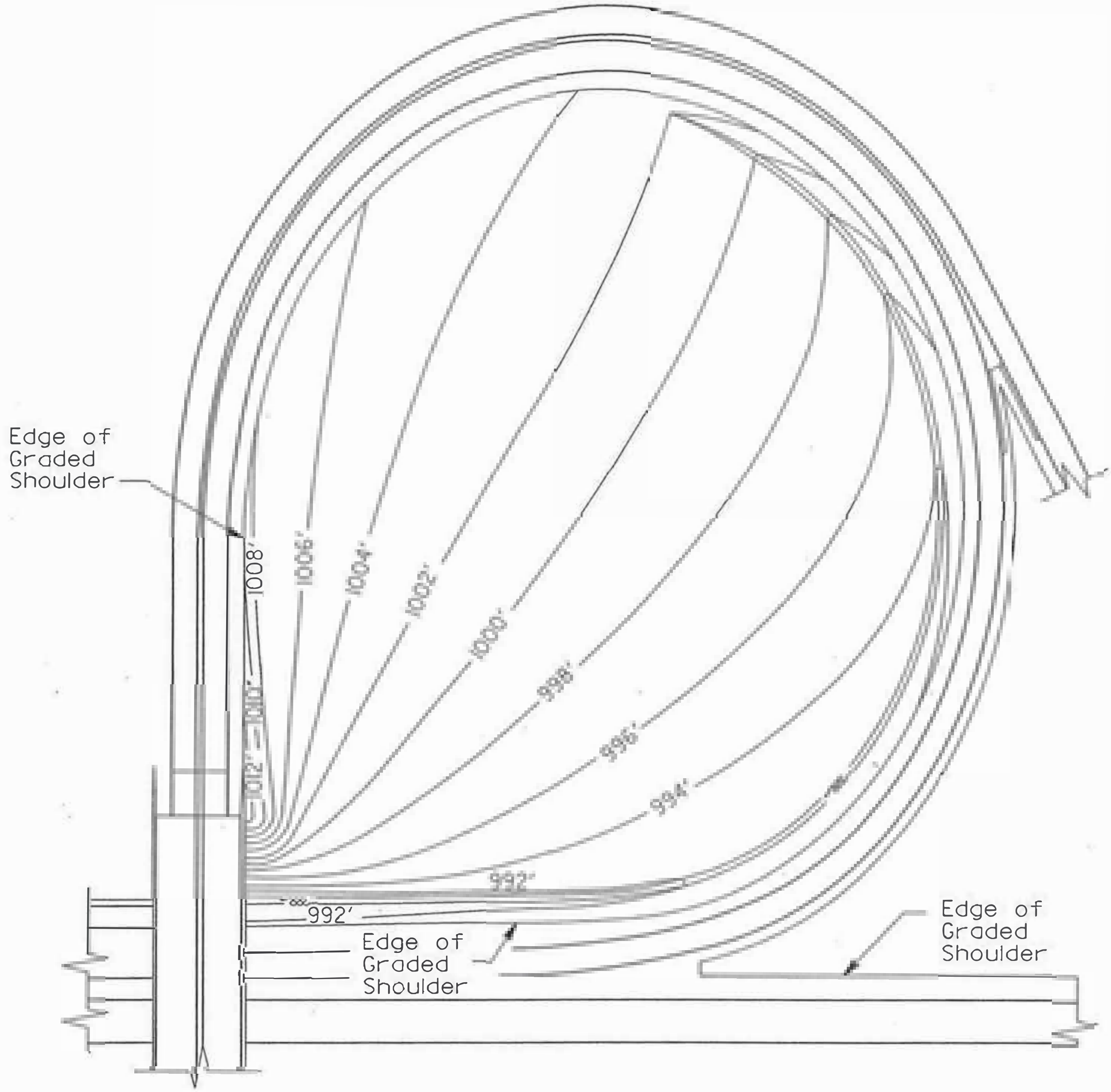
REFERENCE SECTIONS 307.5.3 & 307.5.4



CONTOUR GRADING OF LOOP INTERIORS IN CUT

307-9

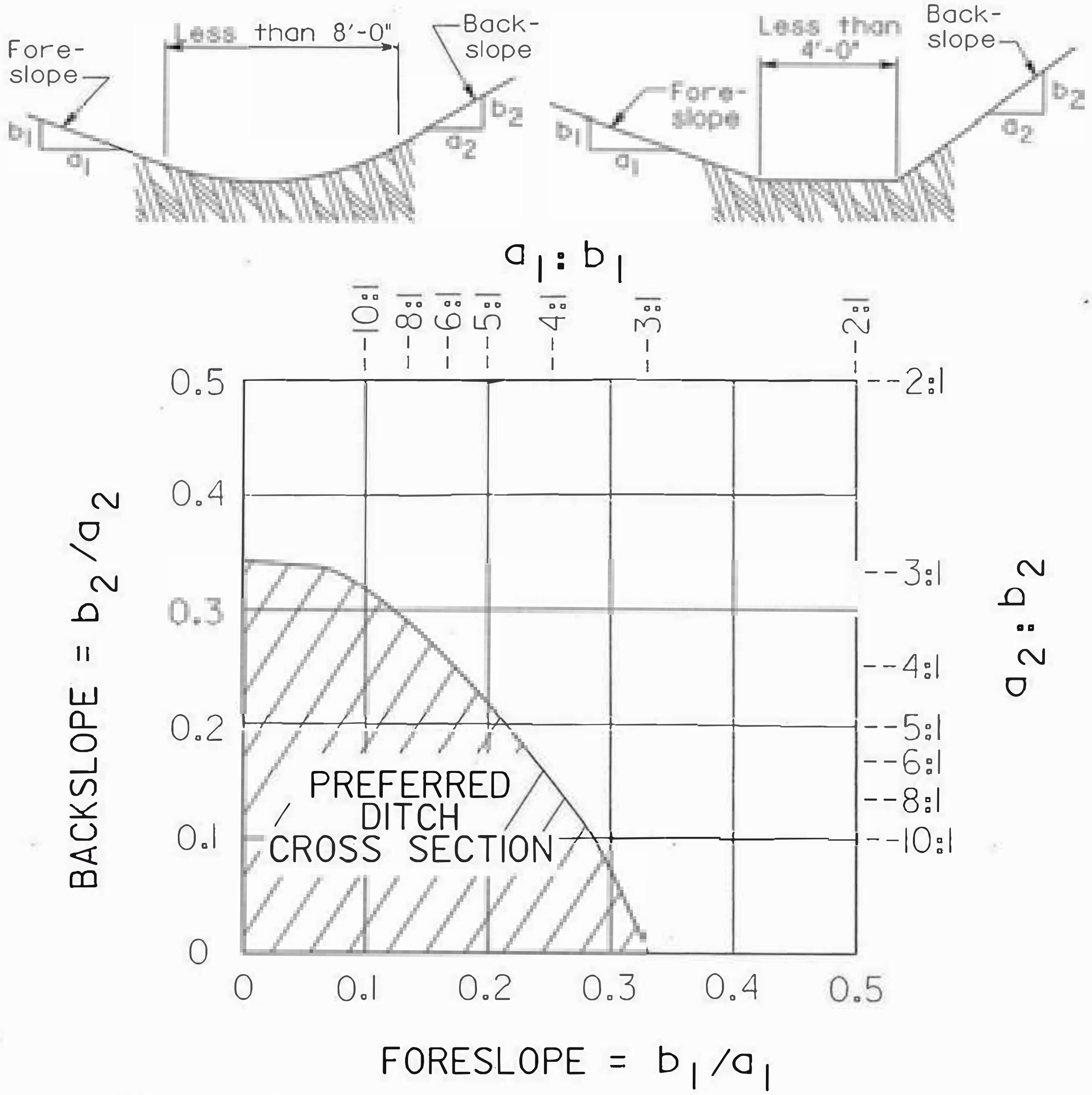
REFERENCE SECTIONS 307.5.5



PREFERRED CROSS SECTIONS FOR DITCHES WITH ABRUPT SLOPE CHANGES

307-10

REFERENCE SECTIONS 600.2.1

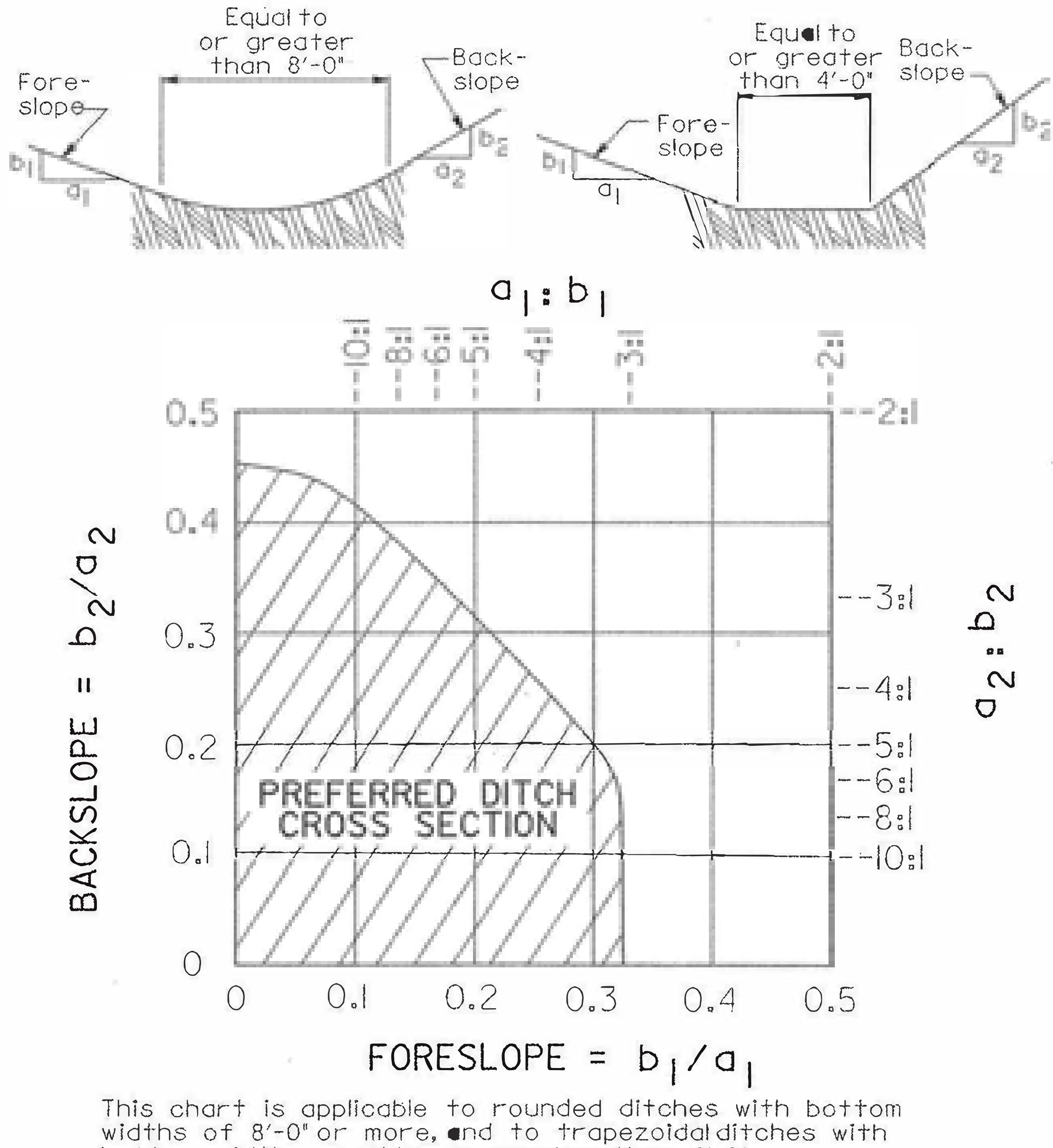


This chart is applicable to vee ditches, rounded ditches with bottom widths less than 8'-0", and trapezoidal ditches with bottom widths less than 4'-0".

Ditch sections that fall within the shaded areas of the figure above are considered traversable and are preferred for use within the Clear Zone. Ditch sections that fall outside the shaded areas are considered non-traversable and should generally be located outside the Clear Zone.

November 2002

PREFERRED CROSS SECTIONS FOR DITCHES WITH GRADUAL SLOPE CHANGES



This chart is applicable to rounded ditches with bottom widths of 8'-0" or more, and to trapezoidal ditches with bottom widths equal to or greater than 4'-0".

Ditch sections that fall within the shaded areas of the figure above are considered traversable and are preferred for use within the Clear Zone. Ditch sections that fall outside the shaded areas are considered non-traversable and should generally be located outside the Clear Zone.

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400 Intersection Design

Table of Contents

400 Intersection Design

401 INTERSECTIONS AT-GRADE

401.1 Intersection Locations

Care should be taken in locating new at-grade intersections. The alignment and grade on the mainline roadway should, as a minimum, provide stopping sight distance as discussed in ***Section 201.2***. The criteria for intersection sight distance (see ***Section 201.3***) should also be met wherever possible.

It is best to avoid locating an intersection on a curve. Since this is often impossible, it is recommended that intersection sites be selected where the curve superelevation is 0.04 or less. It is also recommended that intersections be located where the grade on the mainline roadway is 6 percent or less, with 3 percent being the desirable maximum.

401.2 Intersection Traffic Control and Operational Analysis

The type of traffic control at intersections directly affects the geometric design. An early determination of the type of intersection control must be made (i.e. stop signs, signal, or roundabout). Whenever there is doubt as to the adequacy of stop control during the design life of the project, traffic signal warrants, as outlined in the **Ohio Manual of Uniform Traffic Control Devices (OMUTCD)** should be investigated.

Intersection capacity analysis procedures of the current edition of the Highway Capacity Manual shall be used to determine the number and type of lanes at intersections. Intersections shall be analyzed and designed to accommodate traffic volumes as per Section 102.2. Analyses shall be performed using the current version of Highway Capacity Software (HCS). Other software may be used to supplement the HCS analysis, depending upon intersection type. Refer to ***Figures 401-14a*** thru ***401-14c*** for software guidance.

401.2.1 Signals

In general, when performing capacity analysis of signalized intersections, the critical (worst) delay of the north/south approach should approximately equal the worst delay of the east/west approach. Approach delays are considered balanced when they are within 3 seconds. Additionally, the highest control delays should be balanced, preferably within 5 seconds. When intersections are severely over capacity, balancing the approach and control delays may not be feasible; however, the critical delays should be as close as practical. This methodology provides a common basis for comparison between the no-build and build alternatives and is not meant to provide a signal timing plan for daily operations. This methodology defines the intersection size in consideration of the 20-year design.

Determination of the necessary number and type of lanes for the build condition is based upon a signal design where all individual lane v/c ratios are less than 1.0, and preferably less than 0.93.

401.2.2 Stop Control

For stop controlled intersections, where signal warrants are not anticipated to be met by the design year or would not be installed due to access management controls, ***Figures 401-5a*** thru ***401-6d*** are provided to determine the need for turn lanes. The stopped approaches may be evaluated using HCS to determine the necessary number and type of lanes to improve the Levels of Service.

401.2.3 Roundabouts

Use HCS for the analysis of a roundabout. Use SIDRA when HCS is incapable of analyzing a roundabout configuration. Configurations that HCS cannot analyze include: when more than two lanes enter a roundabout on an approach (not including a right-turn bypass lane), or when there are more than two lanes within the circulatory roadway. Turn lane lengths for the approach lanes of the roundabout shall be determined by accommodating the 95th percentile queue lengths as identified by HCS or SIDRA. Refer to ***Figures 401-14a*** thru ***401-14c*** for software guidance.

While it is important to plan for future traffic volumes and capacity needs, the immediate effects on users should also be considered including costs. A roundabout constructed with a wide cross section (multilane) can negatively impact user (pedestrian, bicycle, unfamiliar drivers) movements. Therefore, a phased implementation on multilane roundabouts is required if the single lane construction of the roundabout can meet acceptable levels of service based on an interim design year. The phased implementation should be based on the available and future funding resources and location (rural or urban, drivers familiar or unfamiliar). The current users' needs will be accommodated while still providing an opportunity for the roundabout to be expected for future traffic volume growth.

When using a phased approach, it is important to design the full build layout footprint to ensure right-of-way is secured for future planned improvements. It is also beneficial to plan the construction of the roundabout to potentially allow for easier expansion in the future.

401.3 Crossroad Alignment

Intersection angles of 70 degrees to 90 degrees are to be provided on all new or relocated highways. An angle of 60 degrees may be satisfactory if: (1) the intersection is signalized; or (2) the intersection is skewed such that a driver stopped on the side road has the acute angle (at center of intersection) on his left side (vision not blocked by his own vehicle).

Relocation of the crossroad is often required to meet the desired intersection location, to avoid steep crossroad profile grades and to adjust intersection angles. Horizontal curves on crossroads should be designed to meet the design speed of the crossroad. The crossroad alignment should be as straight as possible. ***Figure 401-1*** shows an example of a crossroad relocation. Both curve 1 and curve 3 may be reduced per the figure.

401.4 Crossroad Profile

401.4.1 Intersection Area

The portion of the intersection located within 60 ft. of the mainline edge of traveled way, measured along the crossroad centerline, is considered to be the "intersection area". The pavement surface within this "intersection area" should be visible to drivers within the limits of the minimum stopping sight distance shown on ***Figure 201-1***. By being able to see the pavement surface (height of object of 0), drivers (height of eye of 3.5 ft.) will be able to observe the radius returns, pavement markings, and recognize that they are approaching an intersection. ***Figure 401-2*** shows the "intersection area".
Combinations of pavement cross slopes and profile grades may produce unacceptable edge of traveled way profiles in the "intersection area". For this reason, edge of traveled way profiles should be plotted and graphically graded to provide a smooth profile.

401.4.2 Drainage

Within the intersection area, the profile of the crossroad should be sloped wherever possible so the drainage from the crossroad will not flow across the through road pavement. For a stop condition, the 10 ft. of crossroad profile adjacent to the through pavement is normally sloped away from the through pavement, using at least a 1.6 percent grade, as shown on ***Figure 401-2***.

401.4.3 Profile at Stop Intersections

Profile grades within the "intersection area" for stop conditions are shown in ***Figures 401-2*** and ***401-3***. The grade outside the "intersection area" is controlled by the design speed of the crossroad. Normal design practices can be used outside the "intersection area" with the only restriction on the profile being the sight distance required in ***Section 401.4.1***.

Grade breaks are permitted at the mainline edge of traveled way for a stop condition as discussed in Note 3 of ***Figure 401-2***. If these grade breaks are exceeded, they should be treated according to Note 3 on ***Figure 401-3***. Several examples are shown on ***Figure 401-3*** of the use of grade breaks or short vertical curves adjacent to the mainline edge of traveled way.

401.4.4 Profile at Signalized Intersections

Signalized intersections require a more sophisticated crossroad profile. Whenever possible, profiles through the intersection area of a signalized intersection should be designed to meet the design speed of the crossroad. ***Figure 401-4*** shows three examples of crossroad profiles at intersections. On Examples A and B (***Figure 401-4***), the mainline cross slopes will need to be adjusted to match the crossroad profile within the intersection area. Grade breaks shown on Examples A and C should be in accordance with ***Section 203.3.2***. Since the grade break across a normal crowned pavement is 3.2 percent, it should be noted that the crown must be flattened (See Example C). This will allow vehicles on the crossroad to pass through the intersection on a green signal safely without significantly adjusting their speed. The sight distance requirements of ***Section 401.4.1*** within the "intersection area" are also applicable for signalized intersections.

401.5 Approach Radii

401.5.1 Rural

Approach radii in rural areas shall normally be 50 ft., except that radii less than 50 ft. (minimum 35 ft.) may be used at minor intersecting roads if judged appropriate for the volume and character of turning vehicles.

Radii larger than 50 ft., a radius with a taper, or a three center curve, should be used at any intersection where the design must routinely accommodate semi-trailer truck turning movements. Truck turning templates should be used to determine proper radii and stop bar location. When truck turning templates are used, a 2 foot clearance should be provided between the edge of traveled way and the closest tire path.

Normally the approach width at the ends of the radius returns should be 24 ft. The pavement width shall be tapered back to the normal pavement width at a rate of 10:1 if the taper is adjacent to the radius returns.

401.5.2 Urban

Corner radii at street intersections should consider the right of way available, the intersection angle, pedestrian traffic, approach width and number of lanes. The following should be used as a guide:

1. 15 to 25 ft. radii are adequate for passenger vehicles and may be provided at minor cross streets where there are few trucks or at major intersections where there are parking lanes.

2. 25 ft. or more radii should be provided at minor intersections on new or reconstruction projects where space permits.

3. 30 ft. radii or more should be used where feasible at major cross street intersections.

4. Radii of 40 ft. or more, three-centered compound curves or simple curves with tapers to fit truck paths should be provided at intersections used frequently by buses or large trucks.

401.5.3 Curbed to Uncurbed Transitions

Figures 401-4a and ***401-4b*** show acceptable methods to transition from curbed to uncurbed roadways at intersections. ***Figure 401-4a*** shows two options to transition from an uncurbed mainline roadway to a curbed approach roadway. ***Figure 401-4b*** shows the transition from a curbed mainline roadway to an uncurbed approach roadway. See ***Section 305.4*** for additional information.

401.6 Approach Lanes

401.6.1 Left Turn Lanes

Probably the single item having the most influence on intersection operation is the treatment of left turn vehicles. Left turn lanes are generally desirable at most intersections. However, cost and space requirements do not permit their inclusion in all situations. Intersection capacity analysis procedures of the current edition of the Highway Capacity Manual should be used to determine the number and use of left turn lanes. For unsignalized intersections, left turn lanes may also be needed if they meet warrants as provided in ***Figures 401-5a, 401-5b***, and ***401-5c***. The warrants apply only to the free-flow approach of the unsignalized intersection. Refer to ***Section 401.2*** and **Figures *401-14a*** thru ***401-14c*** for analysis criteria and software guidance.

Left turn lanes should be placed opposite each other on opposing approaches to enhance sight distance. They are developed in several ways depending on the available width. The first example on ***Figure 401-7*** shows the development required when additional width must be generated. The additional width is normally accomplished by widening on both sides. However, it could be done all on one side or the other. In the second example on ***Figure 401-7***, the median width is sufficient to permit the development of the left turn lane. ***Figure 401-8*** shows the condition where an offset left turn lane is required to obtain adequate sight distance in wide medians.

In developing turn lanes, several types of tapers may be involved as shown in ***Figure 401-7***.

1. Approach Taper - An approach taper directs through traffic to the right. Approach taper lengths are calculated using the following:

   Design Speed of 50 mph or more: L=WS

   Design speed less than 50 mph: L=WS2/60

   Where: L = Approach taper length in feet
   W = Offset width in feet

S = Design Speed

2. Departure Taper - The departure taper directs through traffic to the left. Its length should not be less than that calculated using the approach taper equations. Normally, however, the departure taper begins opposite the beginning of the full width turn lane and continues to a point opposite the beginning of the approach taper.

3. Diverging Taper - The diverging taper is the taper used at the beginning of the turn lane. The recommended length of a diverging taper is 50 ft..

Figures 401-9 and ***401-10*** have been included to aid in determining the required lengths of left turn lanes at intersections. An example problem that illustrates the use of these figures is included along with the figures.

After determining the length of a left turn lane, the designer should also check the length of storage available in the adjacent through lane(s) to assure that access to the turn lane is not blocked by a backup in the through lane(s). To do this, ***Figure 401-10*** may be entered using the average number of through vehicles per cycle, and the required length read directly from the table. If two or more lanes are provided for the through movement, the length obtained should be divided by the number of through lanes to determine the required storage length.

It is recommended that left turn lanes be at least 100 ft. long, and the maximum storage length be no more than 600 ft.

The width of a left turn lane should desirably be the same as the normal lane widths for the facility. A minimum width of 11 ft. may be used in moderate and high speed areas, while 10 ft. may be provided in low speed areas. Additional width should be provided whenever the lane is adjacent to a curbed median as discussed in ***Section 305.3***.

401.6.2 Double Left Turn Lanes

Double left turn lanes should be considered at any signalized intersection with left turn demands of 300 vehicles per hour or more. The actual need shall be determined by performing a signalized intersection capacity analysis. Refer to ***Section 401.2*** and ***Figures 401-14a*** thru ***401-14c*** for analysis criteria and software guidance. Fully protected signal phasing is required for double left turns.

When the signal phasing permits simultaneous left turns from opposing approaches, it may be necessary to laterally offset the double left turn lanes on one approach from the left turn lane(s) on the opposing approach to avoid conflicts in turning paths. All turning paths of double left turn lanes should be checked with truck turning templates allowing 2 ft. between the tire path and edge of each lane. Expanded throat widths are necessary for double left turn lanes. For details on double left turn lanes, see ***Figures 401-11 and 401-12.***

401.6.3 Right Turn Lanes

Exclusive right turn lanes are less critical in terms of safety than left turn lanes. Right turn lanes can significantly improve the level of service of signalized intersections. They also provide a means of safe deceleration for right turning traffic on high-speed facilities and separate right-turning traffic at stop-controlled intersections.

To determine the need for right turn lanes, intersection capacity analysis procedures of the current edition of the Highway Capacity Manual should be used. For unsignalized intersections, right turn lanes may also be needed if they meet warrants as provided in ***Figures 401-6a, 401-6b, 401-6c*** and ***401-6d***. The warrants apply only to the free-flow approach of the unsignalized intersection. Refer to Section ***401.2*** and ***Figures 401-14a*** thru ***401-14c*** for analysis criteria and software guidance.

Figure 401-7 shows the design of right turn lanes. ***Figure 401-10*** may be used in preliminary design to estimate the storage required at signalized intersections. The recommended maximum length of right turn lanes at signalized intersections is 800 ft., with 100 ft. being the minimum length.

The blockage of the right turn lane by the through vehicles should also be checked using ***Figure 401-10***. With right-turn-on-red operation, it is imperative that access to the right turn lane be provided to achieve full utilization of the benefits of this type of operation.

The width of right turn lanes should desirably be equal to the normal through lane width for the facility. In low speed areas, a minimum width of 10 ft. may be provided. Additional lane width should be provided when the right turn lane is adjacent to a curb.

401.6.4 Double Right Turn Lanes

When they are justified, it is generally at an intersection involving either an off-ramp or a one-way street. Double right turn lanes require a larger intersection radius (usually 75 ft. or more) and a throat width comparable to a double left turn (See ***Section 401.6.2*** and ***Figure 401-11***).

401.6.5 Additional Through Lanes

Normally the number of through lanes at an intersection is consistent with the number of lanes on the basic facility. Occasionally, through lanes are added on the approach to enhance signal design. As a general suggestion, enough main roadway lanes should be provided so that the total through plus turn volumes does not exceed 450 vehicles per hour per lane. See ***Figure 402-1***.

401.6.6 Recovery Area at Curbed Intersections

When a through lane becomes a right turn lane at a curbed intersection, an opposite-side tapered recovery area should be considered. The taper should be long enough to allow a trapped vehicle to escape, but not so long as to appear like a merging lane. See ***Figure 402-1***.

401.7 Islands

401.7.1 Characteristics

An island is a defined area between traffic lanes used for control of vehicle movement. Islands also provide an area for pedestrian refuge and traffic control devices. Islands serve three primary functions: (1) to control and direct traffic movement, usually turning; (2) to divide opposing or same direction traffic streams usually through movements; and (3) to provide refuge for pedestrians. Most islands combine functions.

Although certain situations require the use of islands, they should be used sparingly and avoided wherever possible.

401.7.2 Channelizing Islands

Channelizing islands control and direct traffic into the proper paths for the intended use and are an important part of intersection design. They may be of many shapes and sizes, depending on the conditions and dimensions of the intersection. A common form is the corner triangular shape that separates right turning traffic from through traffic. ***Figures 401-13a***, ***401-13b***, ***401-13c*** and ***401-13d*** detail Channelizing Island designs for various vehicle combinations.

1. Channelizing Islands are used at intersections for the following reasons:
2. Separation of conflicts.
3. Control of angle of conflict.
4. Reduction in excessive pavement areas.
5. Indication of proper use of intersection
6. Favor a predominant turning movement.
7. Pedestrian protection.
8. Protection and storage of vehicles.
9. Location of traffic control devices.

These islands should be placed so that the proper course of travel is immediately obvious and easy for the driver to follow. Care should be given to the design when the island is on or beyond a crest of a vertical curve, or where there is a substantial horizontal curvature on the approach to or through the channelized area.

Properly placed islands are advantageous where through and turning movements are heavy.

401.7.3 Island Treatments

401.7.3.1 Curbed Islands

Curbed islands are most often used in urban areas where traffic is moving at relatively low speeds (less than 50 mph. The smallest curbed island that should normally be considered is 50 sq. ft. in an urban area and 75 sq. ft. if used in a rural area. A 100 sq. ft. island is preferred in either case. Curb Islands are sometimes difficult to see at night, so the intersection should have fixed source lighting.

401.7.3.2 Painted Islands

Islands delineated by pavement markings are often preferred in rural or lightly developed areas, when approach speeds are relatively high, where there is little pedestrian traffic, where fixed-source lighting is not provided, or where traffic control devices are not located within the island.

401.7.3.3 Nonpaved Islands

Nonpaved islands are normally used in rural areas. They are generally turf and are depressed for drainage purposes.

401.8 Designing Roadways to Accommodate Pedestrians

Designing a roadway that successfully meets the needs of both vehicular traffic and pedestrians can be a challenging task. Basic roadway design parameters such as roadway widths, corner turning radii and sight distances affect the ability of that roadway to accommodate pedestrians.

For example, the wider the roadway, the more difficult it is for pedestrians to cross, and the greater the barrier effect of this roadway on the communities through which it passes. Undivided six-lane arterials, with or without parking, are not usually pedestrian friendly, while eight and ten-lane arterials create an even more formidable barrier.

The size of a corner radius can also have a significant effect on the overall operation and safety of an intersection. Large corner turning radii promote higher turning speeds, as well as increasing the pedestrian crossing distance and exposure time. Large curb radii also reduce the space for pedestrians waiting to cross, move pedestrians out of the turning motorists line of sight, and make it harder for the pedestrian to see turning cars. However, in some cases, corners with small turning radii can impact the overall operating efficiency of an arterial intersection, as well as cause the curb to be hit by a turning vehicle.

The designer must keep in mind that, as important as it is for the motorist to see everything adjacent to the roadway, it is of equal importance for the pedestrian, particularly children and wheelchair users, to be able to view and react to potential conflicts. At no area is this issue more critical than at crosswalk locations. Vehicles parked near crosswalks can create a sight distance problem.

401.8.1 Curb Radii

The radius used at urban and suburban locations at both signalized and unsignalized intersections, where there may be pedestrian conflicts, must consider the safety and convenience aspects of both the motorist and pedestrian. The radius should be the smallest possible for the circumstances rather than design for the largest possible design vehicle, which often accounts for less than 2 percent of the total users. A large radius can increase the speed of turning motorists and the crossing distance for pedestrians, creating increased exposure risks.

Two distinct radii need to be considered when designing street corners. The first is the radius of the street corner itself, and the second is the effective turning radius of the selected design vehicle. The effective turning radius is the radius needed for a turning vehicle to clear any adjacent parking lanes and/or to align itself with its new travel lane. Using an effective turning radius allows a smaller curb radius than would be required for the motorist to turn from curb lane to curb lane. Parking lanes should end at least 20 ft. in advance of the intersection.

401.8.2 Crossing Distance Considerations

Short crosswalks help pedestrians cross streets. Excessive crossing distances increase the pedestrian exposure time, increase the potential of vehicle-pedestrian conflict, and add to vehicle delay.

Curb extensions reduce the crossing distance and improve the sight distances for both the vehicle and the pedestrian. In general, curb extensions should extend the width of the parking lane, approximately 6 ft. from the curb for a minimum length of 20 ft.

401.8.3 Crossing Islands and Medians

Where a wide intersection cannot be designed or timed to accommodate all the pedestrian crossing needs across one leg of the intersection at one time, a median or crossing island (often referred to as a refuge island) should be considered. Medians are raised or painted longitudinal spaces. Triangular channelization islands adjacent to right turning lanes can also act as crossing islands.

Desirably, crossing islands should be at least 5 ft. wide. A width of 8 ft. is needed to accommodate bicycles, wheelchairs, scooters and groups of pedestrians. Crossing island width should be a minimum of 8 ft. on roadways with speeds of 50 mph or greater.

401.8.4 Turning Movements

At both signalized and unsignalized intersections, steps should be taken to ensure that turning speeds are kept low and that sight distance is not compromised for either the motorist or pedestrian.

402 TWO WAY LEFT TURN LANES (TWLTL)

402.1 General

Midblock left turns are often a serious problem in urban and suburban areas. They can be a safety problem due to angle accidents with opposing traffic as well as rear end accidents with traffic in the same direction. Midblock left turns also restrict capacity. Two way left turn lanes (TWLTL) have proven to be a safe and cost-effective solution to this problem.

402.2 TWLTL Justification

TWLTL should be considered whenever actual or potential midblock conflicts occur. This is particularly true when accident data indicates a history of midblock left turn related accidents. Closely spaced driveways, strip commercial development or multiple-unit residential land use along the corridor are other indicators of the possible need for a TWLTL.

Some guidelines which may be used to justify the use of TWLTL are listed below:

1. 10,000 to 20,000 vehicles per day for four lane highways.

2. 5,000 to 12,000 vehicles per day for two lane highways.

3. 70 midblock turns per 1000 ft. during peak hour.

4. Left turn peak hour volume 20 percent or more of total volume.

5. Minimum reasonable length of 1000 ft. or two blocks.

402.3 TWLTL Design

Widths for TWLTL are preferably the same as through lane widths (See ***Section 301.1.2***). A 10 ft. lane may be used in restricted areas. Care should be taken not to make a TWLTL wider than 14 ft. since this may encourage shared side-by-side use of the lane. TWLTL markings shall be in accordance with the OMUTCD. See ***Section 301.1.5*** for location of the crown point.

402.4 Reversible Lanes

A reversible lane is a lane on which the direction of traffic flow can be changed to utilize maximum roadway capacity during peak demand periods. Reverse-flow operation on undivided streets generally is justified where 65 percent or more of the traffic moves in one direction during peak periods, where the remaining lanes are adequate for the lighter flow period when there is continuity in the route and width of the street, where there is no median and where left turn and parking can be restricted. Reverse flow operations require special signing and additional control devices. Refer to the federal MUTCD for further guidance. Reverse flow on a divided facility is termed “contra-flow operation.” While the principle of reverse-flow operation is applicable to divided arterials, the arrangement is more difficult than on an undivided roadway.

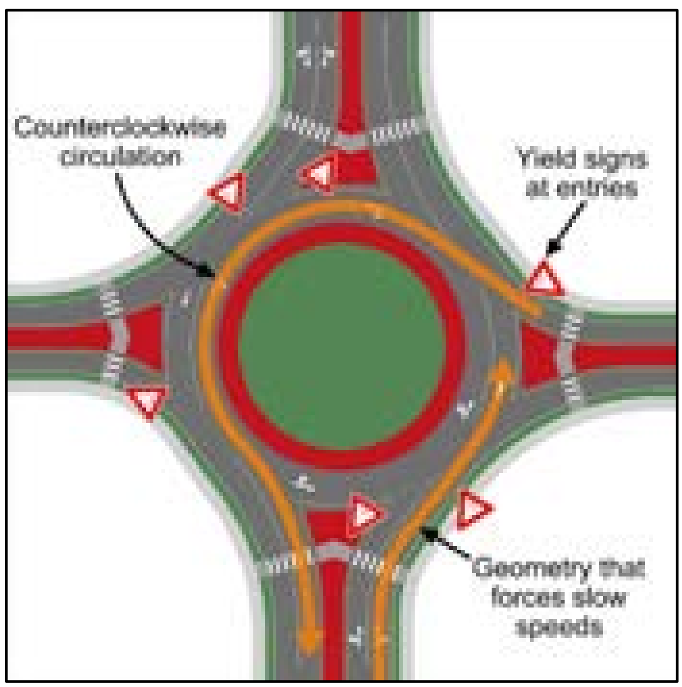
403 ROUNDABOUTS

403.1 General

A roundabout is a form of circular intersection in which circulating traffic travels counterclockwise around a central island in which entering traffic must yield to the circulating traffic. The term “modern roundabout” is used in the United States to differentiate modern roundabouts from traffic calming circles and rotaries that have been in use for many years. The term modern roundabout and roundabout are used interchangeably throughout this document.

Modern roundabouts are defined by three basic characteristics shown and described below:

Exhibit 403.1-1 Characteristics of a Roundabout



1. Circular Shape: Roundabouts are generally circular in shape with traffic circulating in a counterclockwise fashion.
2. Yield-at-Entry to Circulating Traffic: Yield-at-entry requires all entering vehicles on the approaches to yield to the circulatory roadway traffic in the roundabout. The circulating traffic have the right-of-way and all entering vehicles on the approaches must wait for a gap in the circulating flow before

entering the circulatory roadway. Yield signs are used as the traffic control on each approach. Modern roundabouts are not designed for high speed or weaving maneuvers, and therefore have smaller diameters (less than 300 ft.) than large traffic circles/rotaries.

3. Slow Speeds for all Vehicles: Modern roundabouts have geometric features that slow all vehicles down, regardless of the posted speed limits on the approaching roadways. Entering traffic is slowed down and deflected to the right by the approach splitter island into an appropriate curved path along the circulating roadway and around the central island of the roundabout.

Generally, roundabouts can process high left turn volumes more efficiently than all-way stop control or traffic signals; as well as, accommodate a wide range of side road volumes.

Roundabouts can improve safety over that of a conventional intersection through the reduction of vehicular speeds through the roundabout (average 15-25 mph), the reduction of crossing conflicts where the paths of opposing vehicles intersect and a lower angle of impact in the event of a crash. Roundabouts are recognized by FHWA as a Proven Safety Countermeasure.

For additional information not detailed in this section, see **NCHRP and FHWA's Roundabouts: An Informational Guide (NCHRP Report 672)**.

Based on **NCHRP/FHWA's Roundabouts: An Informational Guide (NCHRP Report 672)** roundabouts are separated into three basic categories:

1. Mini-Roundabouts
2. Single-Lane Roundabouts
3. Multi-Lane Roundabouts

403.2 Designing Roundabouts for Future Conditions

Once a roundabout has been selected the design and analysis should consider the potential to phase improvements to reduce excessive capacity in the early years and improve safety and driver/public acceptance. Designers should evaluate whether it is best to construct a roundabout based on an interim year traffic that can easily be converted when future traffic volumes dictate the need for expansion and additional capacity. Reducing the number of entry and circulating lanes reduces the number of potential conflicts with multi-lane roundabouts. When considering an interim roundabout condition that may be converted in the future, the designer should evaluate the right-of-way and geometric needs for both the interim and multilane configurations. When projected traffic volumes indicate a multilane roundabout is required in the design year, designers should evaluate how long an interim configuration (such as a single-lane roundabout) will operate acceptably using the appropriate capacity methodologies before requiring additional lanes. Designers should consider constructing and operating a roundabout in a single-lane configuration if the single-lane roundabout will provide sufficient capacity for a portion of the project design life until traffic volumes dictate an expansion to a multilane roundabout is warranted.

Expansion from the single-lane to the multilane roundabout can be accomplished in two ways:

Expansion to the Outside – The additional entering, circulating and exiting lanes are constructed to the outside of the single-lane roundabout. This option allows for easier future expansion to the multilane configuration since construction can occur while traffic is maintained on the existing single-lane pavement. Care needs to be exercised to ensure the proper geometric design of the entry and splitter islands allow for good speed reduction and the proper path alignment for the ultimate build-out of the roundabout. It is advisable to initially design the ultimate multilane configuration and remove the outside lanes from the design to form the initial single-lane roundabout. Acquire enough right-of-way to accommodate the future condition. It is preferred that pedestrian and off-street bicycle facilities; as well as, utilities be placed in their permanent location from the start rather than relocated at the time of the full build.

Expansion to the Inside – The additional entering, circulating and exiting lanes are constructed to the inside of the single-lane roundabout. The initial single-lane configuration occupies the same inscribed circle as the ultimate multilane roundabout allowing the designer to set the outer limits of the roundabout during the initial construction. Impacts to surrounding properties is minimized under this scenario. The single-lane design will provide wider splitter islands and an enlarged central island that occupies the space of the future inside lanes. Future expansion to the multilane roundabout is accomplished by reducing the width of the splitter islands and contracting the diameter of the central island.

403.3 Operational Analysis

Figure 403-1 is intended to provide rules of thumb for sizing a roundabout. All roundabout designs should be supported with analyses using capacity analysis software. See ***Section 401.2.3*** and ***Figures 401-14a - 401-14c***.

403.4 Design Principles and Objectives

The geometric design of a roundabout is generally an iterative process looking to balance competing considerations such as safety, operations and costs. Several general design principles should be applied to provide a safe and efficient roundabout design:

1. The entry curvature should provide enough deflection to provide slow entry speeds; as well as, consistent speeds through the roundabout.

2. The appropriate number of lanes and the proper lane assignments should be provided to ensure adequate capacity, volume balance and lane continuity to achieve acceptable operations.

3. Provide smooth channelization that is intuitive and will naturally guide drivers into the intended lanes.

4. Roundabout elements should be adequately designed to allow a WB-62 or larger vehicle (on state maintained roadways) that will be using intersection to enter, circulate and depart the roundabout efficiently.

5. Design for non-motorized users such as pedestrians and bicyclists.

6. Provide appropriate sight distance and visibility for driver recognition of the roundabout and conflicting users

403.4.1 Speed Management

The most critical objective is to maintain low and consistent speeds through the roundabout. Roundabouts are safest and most effective when the geometry allows the traffic to enter, circulate and exit at slow consistent speeds. Speed management is a combination of managing speeds at the roundabout itself and managing speeds on the approaching roadways.

Since the design of the roundabout requires entering vehicles to negotiate the roundabout at slow speeds, the design speed may vary within the roundabout intersection. See ***Section 403.7.1*** for information on Fastest Path Speeds. The roundabout approach design speed range is determined by engineering judgment based on several conditions:

1. The roadway classification for each leg of the roundabout and their respective design speeds (high speed or low speed).

2. The intersection users (pedestrians, bicycles, pedestrian generators, large vehicle generators, etc.

3. The location of the roundabout:
    - Rural or urban.
    - Users familiar or unfamiliar with roundabouts.

4. The type of roundabout: Mini-Roundabout, Single-Lane Roundabout or Multilane Roundabout.

5. Desirable maximum entry design speed:
    - Mini-Roundabout (15 to 20 mph)
    - Single-lane roundabout (20 to 25 mph)
    - Multilane roundabout (25 to 30 mph)

6. Design speed consistency:
    - The relative speeds between consecutive geometric elements should be minimized.
    - The relative speeds between conflicting traffic streams should be minimized.
    - Maximum speed differential between movements should be no more than approximately 10 to 15 mph.

7. It is recommended that approach speeds approximately 100 ft. prior to the entrance line (yield line) of the roundabout be limited to approximately 35 mph or less to minimize high-speed rear-end and entering-circulating vehicle crashes. On high-speed rural approaches, it is recommended to use any or some combination of the following techniques to slow down drivers approaching the roundabout:

    - Maximize the Visibility of the Central Island
    - Curbing
    - Splitter Islands
    - Reverse Curves as shown in Exhibit 6-70 of the **Roundabouts: An Informational Guide (NCHRP Report 672)**
        - Superelevation of curves on approaches to roundabouts should be based on the low speed urban street criteria L&D ***Figure 202-9***. The design speed for these approach curves is based on its distance from the yield line and the deceleration length determined from AASTHO GDHS (Greenbook), 7th edition, Figure 2-34. If superelevation is not required, tangents should still be provided between reverse curves. The length of the tangent should be a minimum of 65’ between the broad radius and moderate radius, and 50’ between the moderate radius and entry radius (See NCHRP 672 - Exhibit 6-70).

403.4.2 Lane Arrangements

The operational analysis (See ***Section 401.2.3***) will determine the required number of entry lanes to serve each approach into the roundabout. In the case of multilane roundabouts care must be taken to ensure the design provides the appropriate number of circulating and exit lanes to ensure lane continuity is preserved.

403.4.3 Appropriate Path Alignment

The geometry of the roundabout at the entry will affect the path that vehicles take circulating through and exiting the roundabout. Guided by the lane markings vehicles will continue along their natural trajectory into the circulating roadway based on their speed and orientation. At multilane roundabouts if the entry radius is small there may be a tendency for the vehicle in the outside lane to overlap paths and come into conflict with the vehicle in the inside lane of the circulatory roadway. Overlapping paths presents the potential for inefficient operations due to driver reluctance to use one or more of the entry lanes or sideswipe crashes. Likewise, overly small radii at the exit may result in vehicle path overlap.

At multilane roundabouts the entries and exits should be aligned to position vehicles into their appropriate lanes within the circulatory roadway.

403.4.4 Design Vehicle

Another important factor determining a roundabout's layout is the need to accommodate the largest vehicle reasonably anticipated to use the intersection with some frequency. The turning path of this design vehicle controls many of the roundabout's dimensions.

Because roundabouts are intentionally designed to slow traffic, narrow curb-to-curb widths and tight turning radii are typically used. However, if the widths and turning radii are designed too tight, difficulties for large vehicles may be created. Large trucks and buses often dictate many of the roundabout's dimensions, particularly for single-lane roundabouts. Therefore, it is very important to determine the design vehicle at the start of the design and investigative process.

The choice of a design vehicle will vary depending upon the approaching roadway types and the surrounding land use characteristics. A WB-62 design vehicle should be used on roundabouts at interchanges with interstates, freeways, expressways and at intersections on arterial streets and highways. Smaller design vehicles may be chosen at local street intersections. At a minimum, fire engines, transit vehicles, and single-unit delivery vehicles should be considered in urban areas. In rural environments, school buses or farming equipment may govern design vehicle needs.

Design vehicles shall traverse the roundabout without off-tracking over the outside curbing or onto the splitter island curbing. All vertical and sloped curbing shall be placed to avoid trailer off-tracking (rear axles passing over curbing). The central island curbing, the curbing along the outside of the roundabout and the splitter island curbing should be vertical unless mountable slope curbing is needed for specific reason.

403.4.4.1 Design Vehicle Swept Path

Since roundabouts are designed to slow traffic, narrow curb-to-curb and tight turning radii are typically used. If these curb-to-curb widths and turning radii are too tight difficulties may occur for larger vehicles. It is very important to select the proper design vehicle, usually a WB-62 or larger vehicle (on state maintained roadways) and check the vehicle sweep or tracking using CAD-based vehicle turning software. A minimum clearance of 2 ft. should be provided between the outside edge of the vehicle's swept path and the curb line.

403.4.5 Sight Distance and Visibility

The visibility of the roundabout as vehicles approach the intersection and the sight distance for viewing vehicles already operating within the roundabout are key components for providing safe roundabout operations. Roundabouts require both the stopping sight distance and the intersection sight distance to be met.

The design should check to verify that stopping sight distance can be provided at every point within the roundabout; as well as, each entering and exiting approach so that a driver can react to objects or conflicting users within the roadway.

Intersection sight distance must be verified for any roundabout design to ensure there is sufficient distance available for drivers to perceive and react to conflicting vehicles or pedestrians.

See ***Section 403.7.2*** for more information on Sight Distance.

403.4.5.1 Intersection Sight Distance

Intersection Sight Distance is the distance required for a driver without the right-of-way to perceive and react to the presence of conflicting vehicles. The intersection sight distance is measured by sight triangles defining the space a driver should be able to identify gaps in circulating traffic and presence of pedestrians at crosswalks.

403.4.6 Approach Design

The primary safety concern with a high-speed roundabout is to make drivers aware of the approaching roundabout with ample distance to decelerate comfortably to the appropriate speed. To achieve the appropriate speed designs should follow these principles:

- Provide the desirable stopping sight distance at the entry point based on approach operating speeds.
- Align approach roadways and set vertical profiles to make the central island conspicuous.
- Splitter islands should extend upstream of the yield line to the point at which entering drivers are expected to begin decelerating from the approach speed to the entry design speed.
- Approach curves should be gentle, become successively smaller and should be sized based on the design speed and expected speed change.
- Tangents should be used between reverse curves to prevent truck rollovers.
- Use landscaping if sight lines are not impacted to create a tunnel effect.
- Provide illumination in transition to the roundabout.
- Use signs and pavement markings to effectively advice drivers of the change in speed and path.

403.5 Size, Position and Alignment of Approaches

Three decisions are needed in the design of roundabouts to balance the geometric design principles and objectives described in ***Section 403.4***. The design decisions are:

- Size
- Position

- Alignment of the Approach Legs

403.5.1 Size of the Roundabout (Inscribed Diameter)

The best parameter determining the size of the roundabout is the inscribed circular diameter which is the distance across the circle inscribed by the outer edge of the curb. In all cases, the design vehicle shall have the capability of traversing the roundabout within the circulatory roadway using the truck apron when needed. **Exhibit 403.6-1 Design Elements of a Roundabout** shows the Inscribed Diameter measurement.

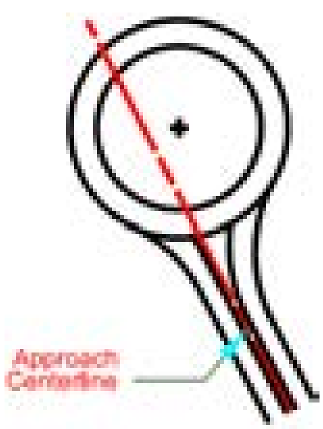
Typical Inscribed Circle Diameters:

- Mini-Roundabout: 45 to 90 ft.
- Single-Lane Roundabout: 90 to 180 ft.
- Multilane (2-lane) Roundabout: 150 to 215 ft.
- Multilane (3-lane) Roundabout: 215 to 300 ft.

403.5.2 Position of Approaches

The position is determined by the alignment of the approach leg in relationship to the center of the inscribed circle. The position of the approach alignment for each of the legs has a primary effect on the deflection and speed control. The three positions are:

- Offset Alignment to the Left of Center
- Alignment through the Center of the Roundabout
- Offset Alignment to the Right of Center



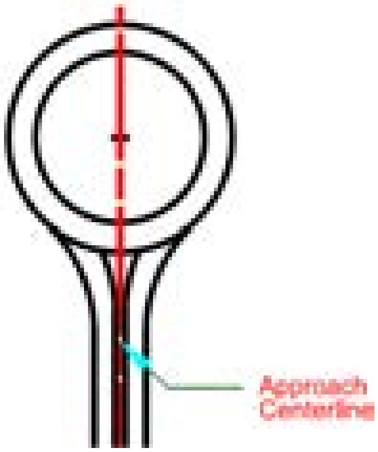
<u>Position 1: Offset Alignment to the Left of Center</u>

Advantages:

- Allows for increased deflection
- Beneficial for accommodating large trucks with small inscribed circle diameter, allows for larger entry radius while maintaining deflection and speed control
- May reduce impacts to the right-side of the roadway

Trade-offs:

- Increased exit radius or tangential exit reduces control of exit speeds and acceleration through crosswalk area
- May create greater impacts to the left side of the roadway



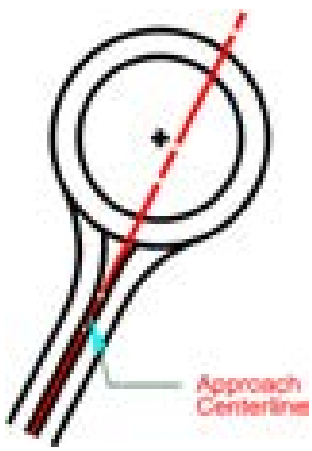
Position 2: Alignment through Center of Roundabout

Advantages:

- Reduces amount of alignment changes along the approach roadway to keep impacts more localized to intersection
- Allows for some exit curvature to encourage drivers to maintain slower speeds through the exit

Trade-offs:

- Increased exit radius reduces control of exit speeds/acceleration through crosswalk area
- May require a slightly larger inscribed circle diameter (compared to offset-left design) to provide the same level of speed control



Position 3: Offset Alignment to the Right of Center

Advantages:

- Could be used for large inscribed circle diameter roundabouts where speed control objectives can still be met
- Although not commonly used, this strategy may be appropriate in some instances (provided that speed objectives are met) to minimize impacts, improve view angles, etc.

Trade-offs:

- Often more difficult to achieve speed control objectives, particularly at small diameter roundabouts
- Increases the amount of exit curvature that must be negotiated

403.5.3 Alignment Approach Legs

Generally, it is preferable to have the approaches intersect at or near perpendicular. Approach legs intersecting at an angle greater than 105 degrees can result a lack of deflection may result in fast vehicle speeds. Approach legs intersecting at an angle less than 75 degrees can result in difficulty for the design vehicle to navigate the entry into the roundabout.

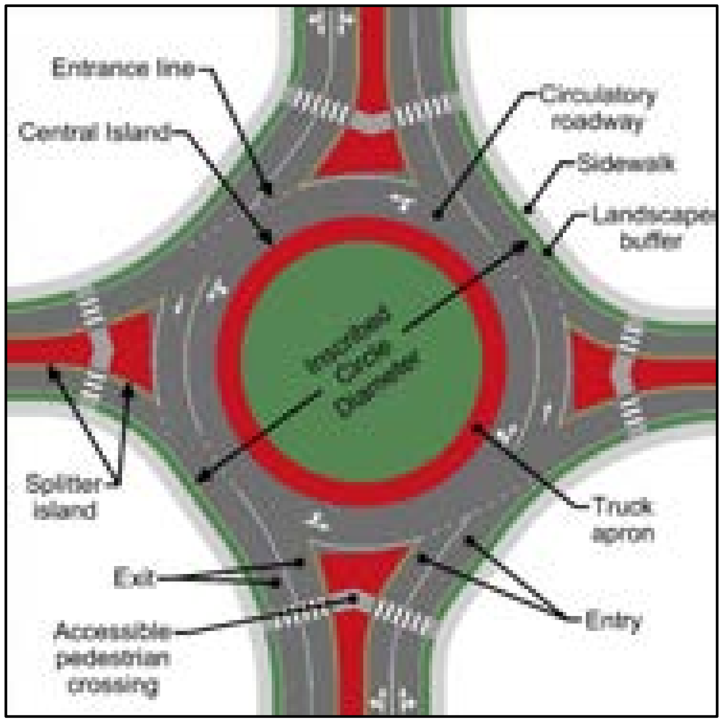
403.6 Design Elements of a Roundabout

This section describes the individual geometric elements comprising a roundabout.

403.6.1 Inscribed Diameter (Outside Limits of Circulatory Roadway)

See ***Section 403.5.1*** and ***Exhibit 403.6-1***.

Exhibit 403.6-1 Design Elements of a Roundabout



403.6.2 Circulatory Roadway Width

The required width of the circulatory roadway width is determined from the number of the entering lanes and turning requirements of the design vehicle.

Circulatory Roadway Widths:

- Single Lane: 16 to 20 feet
- Two Lane: 28 to 32 feet

403.6.3 Central Island

The central island is the area surrounded by the circulatory roadway. The central island should be raised or mounded to ensure approaching drivers are aware of the approaching roundabout. The central island is mainly non-traversable, but it does include the truck apron which is traversable. Teardrop shaped islands may be used where certain movements do not exist such as an interchange or where certain turning movements cannot be accommodated.

Central Island Treatment

- Mini-Roundabout: Fully Traversable
- Single-Lane Roundabout: Raised (with traversable truck apron).
- Multilane Roundabout: Raised (with traversable truck apron).

403.6.4 Truck Apron

A truck apron is typically used to provide additional traversable area around the central island to accommodate large vehicles without compromising the deflection for the smaller vehicles. Truck apron widths should be checked with truck turning templates allowing for 2 ft. between the tire path and inside edge of the truck apron. See **NCHRP/FHWA's Roundabouts: An Informational Guide (NCHRP Report 672)** for additional truck apron details.

Truck Apron Widths: 3 to 15 feet. For ODOT maintained facilities on the State Highway System (SHS) provide 12 ft. min. Truck apron reveal to circulatory roadway: 2-3 inch maximum.

403.6.5 Entry and Exit Widths

The entry and exit widths should accommodate the desired design vehicle and avoid off-tracking over curbing and beyond pavement limits.

Preferred Entry and Exit Widths:

- Single Lane: 14 to 18 feet
- Two Lane: 28 to 30 feet

403.6.5.1 Development of Entry Widths for Multilane Roundabouts

Locations where additional entry capacity is required, ***Figures 402-2*** and ***402-3*** show acceptable methods of widening the roadway prior to the roundabout.

403.6.5.2 Exit Width Reduction of Multilane Roundabouts

Locations where additional capacity is required within the roundabout but immediately reduced on the exit leg of the roundabout, the Roundabout Taper Option on ***Figure 402-2*** shows the acceptable method of reducing lanes along the roadway of the exit leg. Lane reductions at the exit could affect lane utilization at entries.

403.6.6 Gore Striping

An allowable technique to accommodate the design vehicle is to provide gore striping. Gore striping involves placing a striped vane island between the entry lanes to help center the vehicles within the lane and allow a cushion for off-tracking by the design vehicle.

403.6.7 Right-Turn Bypass Lanes

At locations with a high volume of right-turning traffic, a right-turn bypass lane may allow a single-lane roundabout to continue to function acceptably and avoid the need to upgrade to a multilane roundabout. For additional information, see **NCHRP/FHWA's Roundabouts: An Informational Guide (NCHRP Report 672)**.

403.6.8 Cross Slopes

Generally, the circulating roadway and truck apron should be designed to slope away from the central island. This will promote safety by improving the visibility of the central island, lowering the vehicle speeds in the circulating roadway due to the reverse superelevation and minimizing the breaks in the cross slopes of the entry and exit lanes.

Preferred Cross Slopes:

- Circulatory Roadway: 2% away from the central island.
- Truck Apron: 2% towards the outside.

403.6.9 Splitter Islands

Splitter islands should be provided at all roundabout approaches. Their purpose is to provide refuge for pedestrians, assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic and deter wrong-way movements. The total length of the raised island should be at least 50 ft. although 100 ft. is preferable. On higher speed roadways, the island length should be 150 ft. to 200 ft. The minimum island width at a crosswalk should be 6 ft. to provide a refuge for pedestrians.

403.6.10 Curbing

Curbing should be provided on the outside edge of the circulating roadway of the roundabout; as well as, the approach and exit legs. The outside curbs should be 6 inches in height. Refer to ***Section 305.2***. Outside curbing provided positive guidance, slows entry speeds, and prevents parking within the roundabout. The curbing around the truck apron should have a 2-3 inch curb reveal to prevent a truck from overturning.

403.6.11 Lighting

Lighting must be provided per **Traffic Engineering Manual (TEM) Section 1140-4.6.10 Roundabouts**.

403.6.12 Spirals

Spirals are introduced within the circulatory roadway of the roundabout as a series of lane gains and lane drops to lead drivers into the appropriate lane for their desired exit. Spirals guide drivers entering the roundabout on the inside lane to shift into the outside lane within the circulatory roadway enabling the driver to exit from the outside lane. The spiral is designed to prevent vehicles from becoming trapped on the inside lane or requiring drivers to make a quick lane change within the circulatory roadway to exit. Spirals can be installed to accommodate heavy left turning movements. Spirals should only be considered where the circulatory roadway has enough width to provide two or more lanes of traffic and where the geometry and traffic volumes are determined to warrant the use of spirals. Circulatory roadway spirals require considerable engineering judgment to design and locate properly. Although they are intended to guide drivers, they may be confusing to properly understand and not always intuitive to the driver.

A spiral should be developed from the central island by curb and gutter until a full lane width is available. A spiral can be developed with the use of pavement markings, but experience shows drivers may ignore those markings which may lead to crashes within the circulatory roadway.

An example of a roundabout spiral is shown in ***Exhibit 403.6-2***. This spiral is used to shift the westbound left turn to the outside lane. The spiral is used because the southbound exit is only a single lane exit and the southbound entrance allows dual left turns. To exit without conflict, the westbound left turn needs to be spiraled to the outside lane. Without the spiral, the left turn would be trapped on the inside lane and would do a U-turn or have to crossover lanes.

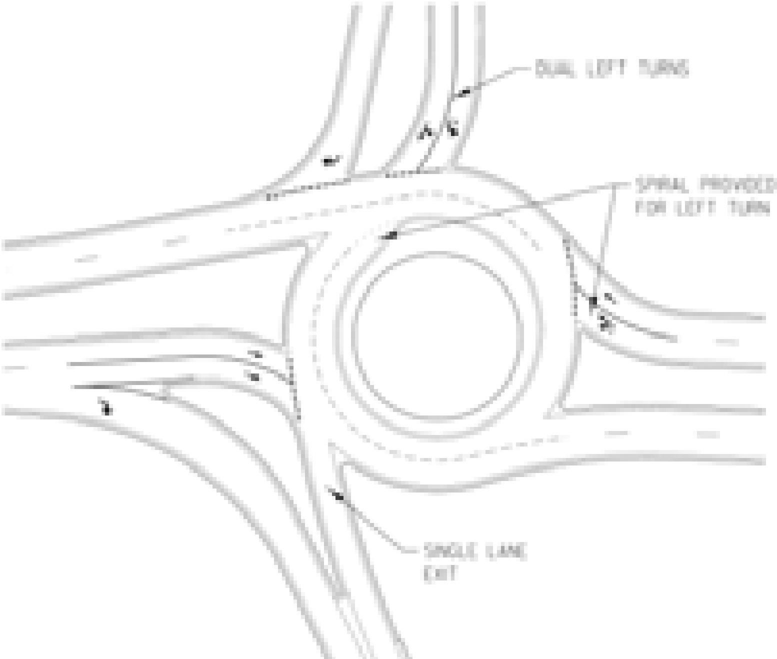


Exhibit 403.6-2 Spiral within a Roundabout

403.7 Performance Checks

This section describes the individual performance checks vital to good roundabout design. The following performance checks will help the designer and reviewer determine whether the design meets the design principles and objectives described in ***Section 403.4***.

A roundabout design checklist shall be submitted with the Stage 1 plan submission to aid the reviewer of the roundabout. See ***Figure 403-2*** for the Roundabout Critical Design Parameters Checklist.

403.7.1 Fastest Path Speeds

The fastest path determines the potential driver speed based on the geometry for a vehicle traversing into, through and exiting the roundabout. It is the smoothest, flattest path possible for a single vehicle in the absence of traffic and any regard for the lane markings. The fastest path must be drawn for all approaches and all movements. The fastest path doesn't necessarily reflect the vehicle speeds expected through the roundabout, but rather the speeds a vehicle could theoretically attain.

There are five critical path radii to be checked for each approach. The five critical path radii are defined as follows:

- Entry Path Radius (R1) is the minimum radius prior to the entrance line.
- Circulating Path Radius (R2) is minimum radius around the central island.
- Exit Path Radius (R3) is the minimum radius into the exit.
- Left-Turn Path Radius (R4) is the minimum radius on the path of the left-turn movement
- Right-Turn Path Radius (R5) is the minimum radius on the path of the right-turn movement.

See **Exhibit 403.7-1** for a visual representation of the fastest path.

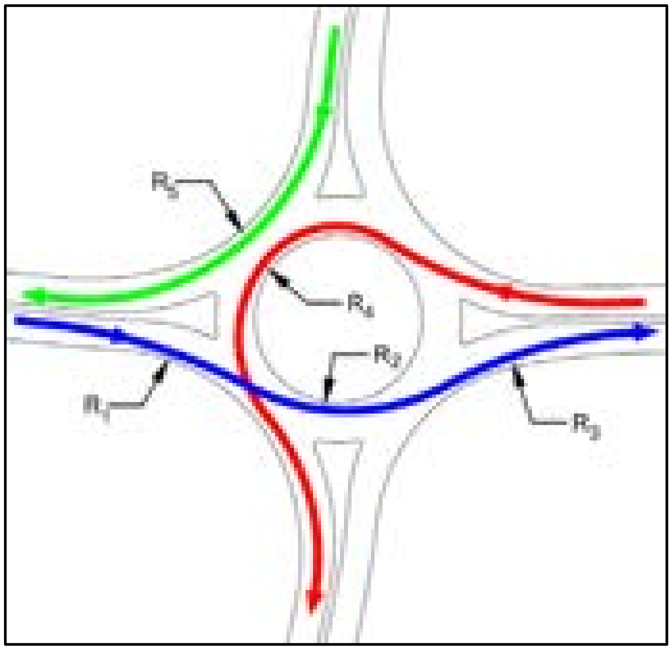


Exhibit 403.7-1 Fastest Path Radii

The performance of the roundabout is chiefly influenced by the Entry Path Radius (R1) due to its ability to control the speed of vehicles entering the roundabout. The recommended maximum theoretical design speeds for the Entry Path Radius (R1) are as follows:

- Mini-Roundabout: 20 mph
- Single Lane: 25 mph
- Multilane: 25 to 30 mph

The design speeds for the movements within the roundabout (R2, R4, R5) should be designed using the following parameters:

- Thru Movements (R2): Provide path deflection of the approach vehicle such that the vehicle is required to slow to 15-20 mph within the circulatory roadway. R2, circulatory radius is critical for controlling thru traffic speed.
- Left Turn Movement (R4): Travel speed is controlled by truck apron diameter (typically 10-15 mph).
- Right Turn Movement (R5): Travel speed is a function of the curb radius and splitter islands between adjacent approaches.

Avoid designing strictly for R1/R2/R3 relationship as described in **Roundabouts: An Informational Guide (NCHRP Report 672)** since this can result in a very tight design for trucks to negotiate.

See ***Section 403.7.1.1*** for guidance on constructing the fastest path.

403.7.1.1 Construction of Vehicle Fastest Path

The fastest path radii should not be mistaken for the curb radii. The vehicle width is assumed to be 6 ft. and maintain a minimum clearance of 2 ft. offset from a roadway centerline or concrete curb and flush with a painted edge line. The path of the fastest path is drawn with the follow distances:

- 5 ft. from a concrete curb
- 5 ft. from a roadway centerline
- 3 ft. from a painted edge line

The construction of the fastest path should begin at least 165 ft. in advance of the entrance line using the appropriate offsets identified above and the Entry Path Radius (R1) should be measured as the best-fit curve over a distance of at least 65 to 80 ft. near the entrance line.

See **Exhibits 403.7-2** and **403.7-3** for Fastest Path movements.

Exhibit 403.7-2 Fastest Path for Through Movements (R1/R2/R3)

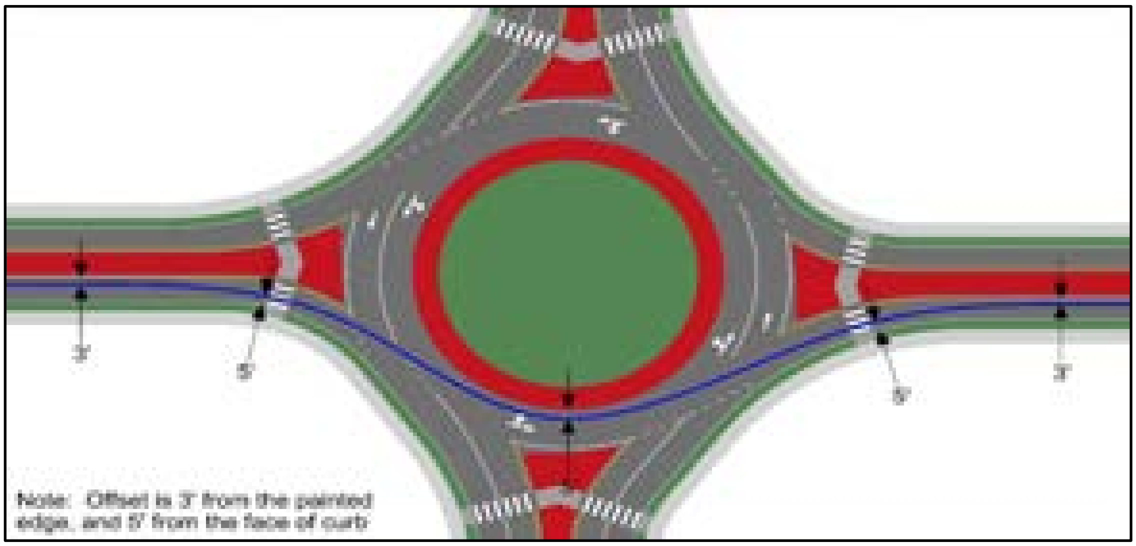
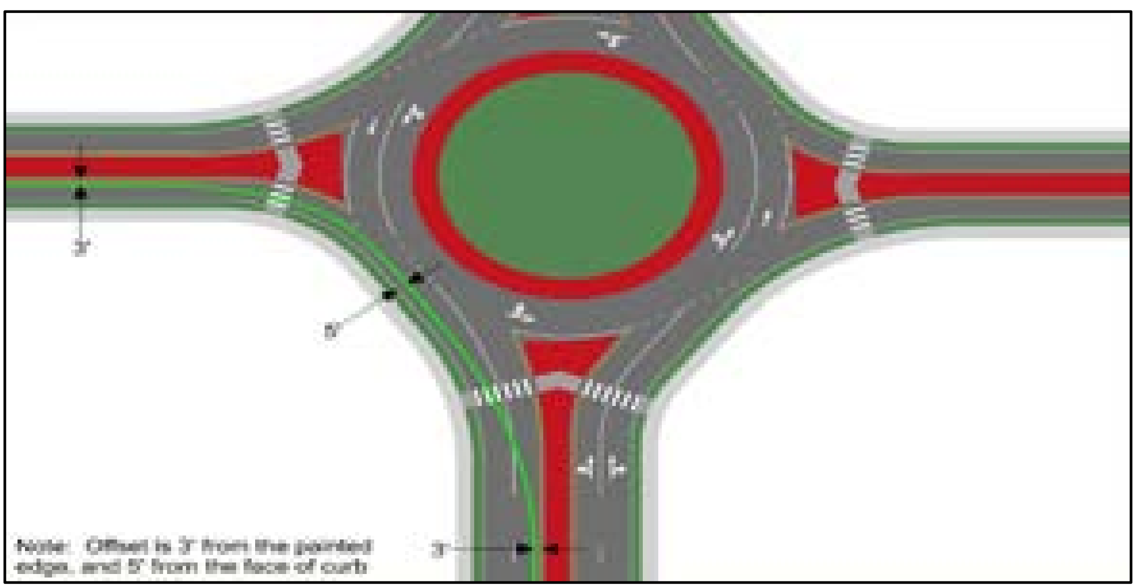


Exhibit 403.7-3 Fastest Path for Right Turn Movements (R5)



403.7.2 Sight Distance

The two aspects of sight distance are the stopping sight distance and the intersection sight distance.

403.7.2.1 Stopping Sight Distance

The stopping sight distance is the distance along a roadway required for a driver to perceive and react to an object in the roadway and brake to a complete stop before reaching the object. The stopping sight distance should be provided at every point within the roundabout and on each entering and exiting approach. For more information regarding stopping sight distance see ***Section 201.2*** and ***Figure 201-1***.

At roundabouts, three stopping sight distance checks should be performed:

- Sight Distance on the Approach
- Sight Distance in the Circulating Roadway
- Sight Distance to the Crosswalk on Exit

See ***Exhibits 403.7-4 - 403.7-6*** below:

Exhibit 403.7-4 Sight Distance on the Approach

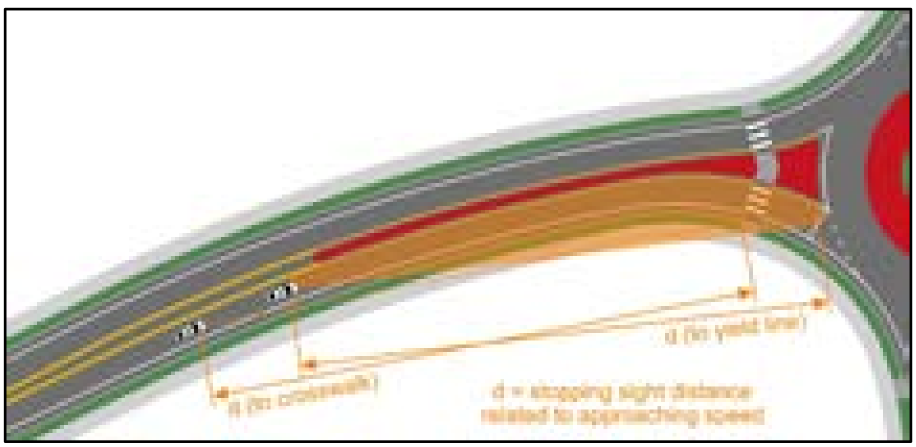


Exhibit 403.7-5 Sight Distance in the Circulating Roadway

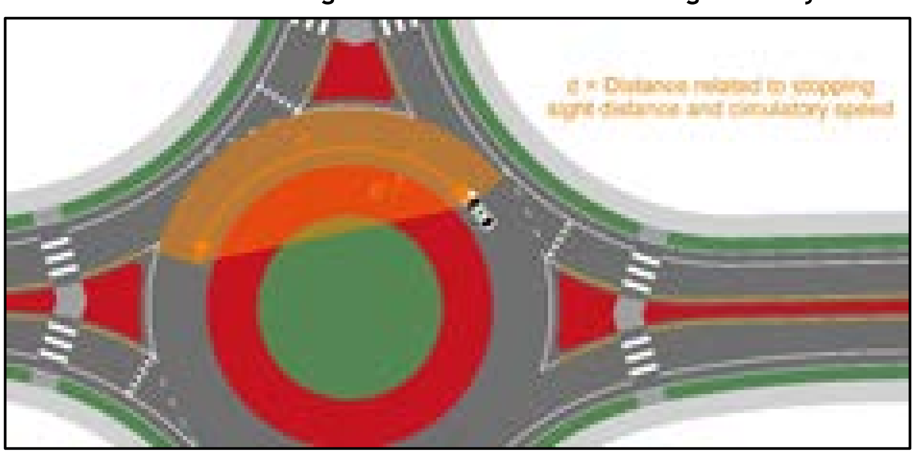
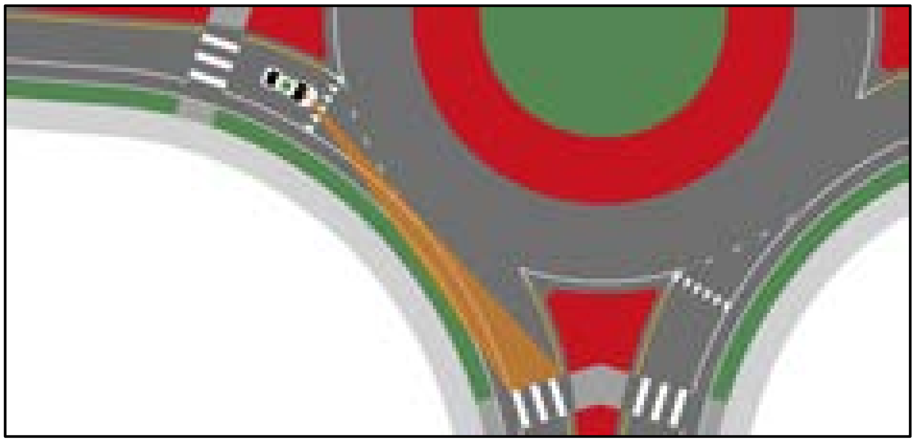


Exhibit 403.7-6 Sight Distance on the Crosswalk on Exit

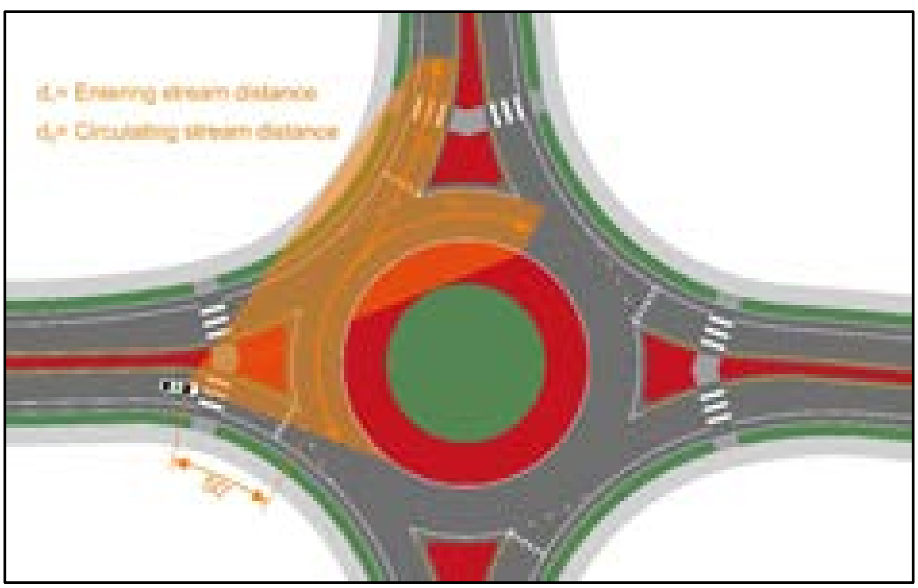


403.7.2.2 Intersection Sight Distance

The intersection sight distance is the distance required for a driver without the right-of-way to perceive and react to the presence of conflicting vehicles. The intersection sight distance is determined with the use of sight triangles that allow a driver to see and safely react to conflicting vehicles. For more information regarding intersection sight distance see ***Section 201.3*** and ***Figures 201-4*** **and** ***201-5***. For roundabouts the only location requiring intersection sight distance checks are the entries.

The intersection sight distance is traditionally measured by the determination of a sight triangle. The triangle is bounded by a length of roadway defining a limit away from the intersection on each of the two conflicting approaches and by a line connecting those two limits. These legs are assumed to follow the curvature of the roadway. These distances should not be measured as straight lines, but rather as the distance along the vehicular path. The length of the approach leg should be limited to 50 ft. from the entrance line. See ***Exhibit 403.7-7***.

Exhibit 403.7-7 Intersection Sight Distance Checks



403.7.3 Entry Angle, Phi

The phi angle is a gauge of the sight to the left and ease of entry to the right. This affects both the capacity and safety at the roundabout. The typical range for the phi angle is between 20 and 30 degrees with 25 degrees being the optimal although designs may operate safely and efficiently with a phi angle as low as 16 degrees. Even if phi angles are not in the desirable range provided the fastest path speeds are relatively low, the phi angle is not a controlling criterion.

There are three situations or design conditions in which Phi can be measured. They are:

- Condition 1: phi = 2 x (phi/2) where the distance between the left sides of an entry and the next exit are not more than approximately 100 feet. In Condition 1, the acute angle is denoted as 2 phi in which the actual value must be divided by two to obtain phi (see Method 1 below).

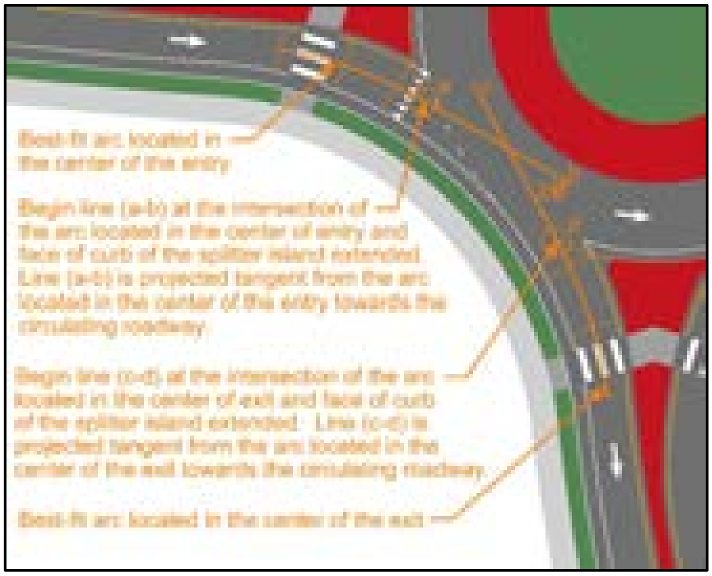
- Condition 2: phi = phi if the distance between the left sides of an entry and the next exit are more than approximately 100 feet (see Method 2 below).

- Condition 3: Applicable when an adjacent exit does not exist, or an exit located at such a distance or obtuse angle to render the circulatory roadway a dominating factor of an entry (such as in a "3-leg" intersection). Used at "T" intersections or where the adjacent entrance and exit lane(s) are far apart (see Method 2 below).

The two methods of measuring phi are described below:

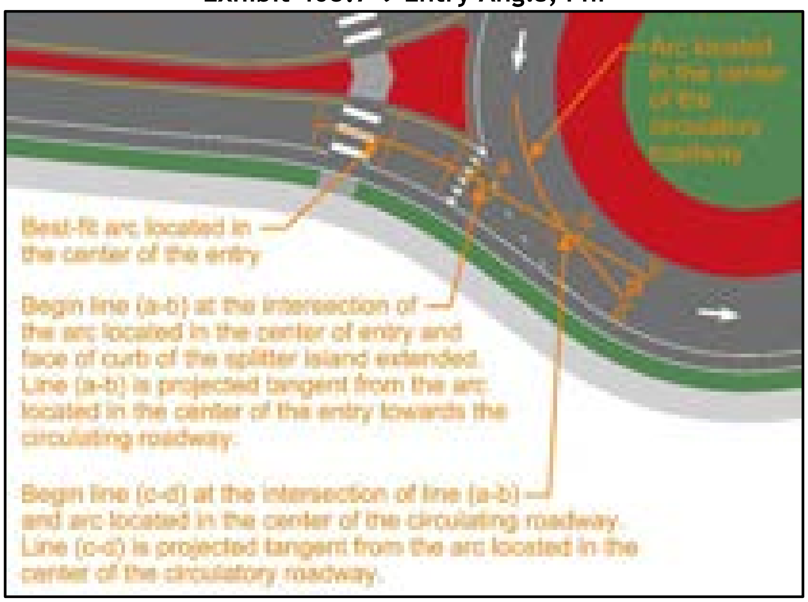
- Method 1 phi is measured by dividing the entry and exit radii into three segments. The midpoint of the lane for each segment is best fit with a curve that extends to the face of curb of the splitter island extended. Begin line (a-b) and (c-d) at the intersection of the best fit arc and face of curb of the splitter island extended. Line (a-b) and (c-d) are then projected tangent from the best fit arc towards the circulating roadway, the angle formed by the intersection of the two lines is twice the value of phi. See **Exhibits 403.7-8** and **403.7-9**.

Exhibit 403.7-8 Entry Angle, Phi



- Method 2 phi is measured by dividing the entry radii into three segments. The midpoint of the lane for each segment is best fit with a curve that extends to the face of curb of the splitter island extended. Begin line (a-b) at the intersection of the best fit arc and face of curb of the splitter island extended. Line (a-b) is then projected tangent from the best fit arc towards the circulating roadway. Begin line (c-d) at the intersection of line (a-b) and the arc located at the center of the circulating roadway. Line (c-d) is then projected tangent from the arc located in the center of the circulating roadway. The angle formed by the intersection of (a-b) and (c-d) is phi.

Exhibit 403.7-9 Entry Angle, Phi



404 RESTRICTED CROSSING U-TURNS (RCUTS)

404.1 General

A Restricted Crossing U-turn (RCUT) also referred to as a J-turn intersection, a superstreet intersection or a synchronized street intersection is a reduced conflict intersection that displaces left turn and through movements from the minor intersecting roadway. The minor roadway left turns and through movements are accommodated by providing one-way median openings located at least 600 feet on each side of the main intersection and requiring a driver from the minor road to turn right onto the main road (multi-lane divided facility) and make a U-turn maneuver downstream at a one-way median opening. When the left-turn volumes are low the left turn movements from the main road can also be prohibited at the main intersection and accommodated at the one-way median opening. There are three main types of RCUT intersections:

- **Merge or Yield-Controlled**: Merge or Yield-Controlled RCUT intersections allow for a high-speed divided four-lane facility in a rural area to function like a freeway corridor. These are a good alternative when funding for interchanges and overpasses may not be readily available. Longer distances between the intersection and the U-turn crossovers will be needed for the weaving movement.

- **Stop-Controlled**: Stop-Controlled RCUT intersections are a good safety treatment for an isolated intersection on a four-lane divided arterial in a rural area. The stop-controlled RCUT intersection can be upgraded to a signal-controlled intersection as traffic volumes increase.

- **Signal-Controlled**: Signal-Controlled RCUT intersections provide good progression along an urban or suburban corridor. Efficiency is provided due the signals requiring only two phases. Speed and signal spacing do not adversely impact the performance of the RCUT intersection. A signalized RCUT intersection can accommodate pedestrians and adjacent driveways. Capacity limits for the minor street may limit the effectiveness of the RCUT intersection so this option may not be appropriate at the intersection of two high volume arterials.

Hybrids of these three types are possible.

The RCUT is a very adaptable intersection which can be used in both a rural and urban setting. The RCUT intersection can be a corridor treatment for arterials and some freeway look-alikes, but they are typically not suitable at the intersection of two arterials.

404.2 RCUT Advantages and Disadvantages

RCUT intersections have unique features and characteristics regarding multimodal considerations, safety performance, operations geometric design, spatial requirements, constructability and maintenance. This section will provide the advantages and disadvantages of RCUT intersections.

Pedestrians/Bicycles

Advantages:

- Reduces conflicts between vehicles and pedestrians for most crossing movements.
- Creates shorter pedestrian crossing distance for some movements.
- Creates opportunities to install mid-block signalized crossings along an arterial.

Disadvantages:

- Increases conflicts between vehicles and pedestrians for some crossing movements.
- Creates longer pedestrian crossing distances for some movements which could add delay and reduce convenience.
- Requires pedestrians to cross in two stages in some cases which could add delay and reduce convenience.
- Pedestrian wayfinding may require additional signage and/or other features to create appropriate crossings for pedestrians of all abilities.
- Provisions for bicycle facilities may be different from conventional intersections and may result in reduced convenience.

Safety

Advantages:

- Reduces the number of vehicle-vehicle conflict points at the intersection which should reduce the potential for crashes.
- At rural four-lane sites, reduces crashes, injuries and fatalities.
- Potentially reduces high severity turning and angle crashes.
- Less severe crash types compared to a conventional intersection.
- Reduces vehicle-pedestrian conflict points.

Disadvantages:

- Increases sideswipe crashes.
- Introduces weaving movements on the high speed mainline roadway.
- Increases travel distances which could lead to more crashes related to distance traveled.

Operations

Advantages:

- Creates the possibility for the largest possible progression bands in both directions of the arterial at any speed with any signal spacing.
- Provides potential to reduce overall travel time at signalized sites.
- Provides potential to reduce delay and travel time for arterial through traffic at signalized sites.
- Provides potential for shorter signal cycle lengths.
- Allows larger portion of signal cycle to be allocated to the arterial through movement.
- Reduces delay for the major street movements by using a two phase rather than a four phase traffic signal control.
- Reduces the need for signalization of intersections along rural, high-speed, divided highways.
- Improves minor street travel times if traffic gap times are insufficient at conventional intersections.

Disadvantages:

- Increases travel distance (and potentially travel time) for minor street left turn and through movements.
- Experiences a firm capacity.
- Creates potential for spillback out of the crossover storage lane.
- Minor-street left turn and through traffic must make unusual maneuvers and may require additional guidance.

Access Management

Advantages:

- Allows multiple driveway or side street locations along the RCUT corridor.
- Signals for driveways or side streets may be installed without introducing significant delay for arterial through movement.
- Allows flexibility for crossover locations to accommodate adjacent driveways and side streets.
- Does not require frontage roads.

Disadvantages:

- Does not allow for a driveway or a side street near entrance to U-turn crossover.
- Landowners will not have driveways with direct left turns out of their properties.

Traffic Calming

Advantages:

- Two-way progression provides the opportunity to set any progression speed (even low speed).
- Provides an additional barrier to fast moving minor-street through traffic.

Disadvantage:

- The additional barrier to direct minor-street through traffic may cause separation to a community.

Footprint

Advantages:

- The greater arterial throughput creates the possibility to reduce the basic number of through lanes on the arterial and still achieve similar service levels.

Disadvantages:

- May require additional right-of-way for loons or wider medians.

Maintenance
Advantages:

- Less queuing on the arterial may reduce pavement rutting and wear.

Disadvantages:

- When signalized, there are more signal controllers and cabinets than a comparable conventional intersection.
- There are more signs than a comparable conventional intersection.
- If designed with a larger median, there is more pavement to maintain than a comparable conventional intersection.
- More pavement to maintain in U-turn crossovers and loons.

Aesthetics
Advantages:

- Median and islands provide opportunity for landscaping.

Costs
Advantages:

- A more cost-effective design as compared to a grade separated facility.

Construction
Advantages:

- Fewer construction impacts.
- Project can be implemented quickly reducing inconvenience to the motorist.

Education
Disadvantages:

- Lack of driver familiarity may require investments in public education and outreach.

404.3 Applications

RCUT intersections can be a great alternative to a conventional intersection under certain conditions. RCUT intersections are most effective at intersections with at least one of the following attributes:

- Intersections where side-street left turn and through volumes are relatively low and the left turn volumes from the major road are high; the ratio of minor road total volume to total intersection volume should be less than or equal to 0.20.

- Areas where median widths are greater than 40 feet; for narrower medians, loons or bulb-outs on the shoulders need to be constructed to allow all vehicles to perform the U-turn movement.

- Intersections with a high number of far-side right-angle crashes.

A table has been provided below for guidance on the feasible demand conditions for an RCUT intersection.

Minor Street Demand (D_{Minor})	Type of RCUT Intersections
D_{Minor} < 5,000 vpd	Consider unsignalized RCUT intersections
5,000 vpd < D_{Minor} < 25,000 vpd	Consider signalized RCUT intersections
D_{Minor} > 25,000 vpd	There are most likely other alternative intersections such as a MUT or DLT intersection that would generally serve the minor street more efficiently.

404.3.1 Signalized RCUT

On a major road where the progression of the through route needs to be maintained with more green time than a conventional signalized intersection can provide the table below summarizes rules-of-thumb for various percentage of green time allocated to the major/minor road. Note an RCUT intersection may be an appropriate solution based on the analyses of traffic operations and geometric requirements.

	Major Street	Minor Street	Notes
Green %	< 60%	≥ 40%	Other intersection designs will likely serve the relatively heavy minor street demand more efficiently
	< 67%	≥ 33%	Minor street demands may be relatively too heavy for an RCUT
	67%	33%	Signalized RCUT is probably appropriate
	75%	25%	
	>75%	≤ 25%	Minor street demands may be too light for signals to be warranted

404.4 Geometric Design

RCUT intersection geometrics create some design principles not present at conventional intersections. These principles include:

- A wide median can make accommodating U-turn movements easier or more straightforward. Bump-outs or loons can be used at narrower medians.
- Positive guidance through design elements and traffic control reduce driver error and promote efficient movements that might otherwise be unexpected.
- U-turn crossovers are large enough to accommodate the design vehicle and allow for more efficient movements through the U-turn by passenger vehicles.
- Corridor-wide access management strategies promote safe and efficient access to properties.
- Stopping sight distance at all points through the intersection and intersection sight distance for all movements is provide at each RCUT intersection junction.

Designing the geometric layout of an RCUT intersection requires considering the relationship between safety, operations and design.

For signal-controlled RCUTs, the main intersection operates as a two-phase signal which may require a shorter cycle length than a conventional signal-controlled intersection. The shorter cycle length may result in shorter traffic queues, less storage lengths and shorter travel distances from the main intersection to the U-turn crossovers.

RCUT intersections are very versatile and offer a wide range of possibilities. A list of design possibilities is provided below:

- Major streets can range from four lanes to eight lanes.
- Minor streets can be up to four through lanes wide.
- U-turn crossovers can be one or two lanes wide.
- Left-turn crossovers can range from one to three lanes wide.
- The distance from the main intersection to the U-turn crossover can vary from 600 feet for a stop or signal controlled RCUT intersection to one-half mile for a merge-controlled RCUT intersection.

The orientation of an RCUT intersection angle may influence the ability to convert a conventional intersection into an RCUT intersection. An RCUT intersection at an acute angle (< 90 degrees) between the major street and the minor street can serve the left turns on the major street more efficiently than one at an obtuse angle (> 90 degrees). See the illustration below.

Better Candidate for RCUT Conversion

Less Ideal for RCUT Conversion

404.4.1 Main Intersection

404.4.1.1 Left Turns

At a four-legged RCUT, the left-turns are served only on the major street. The number of left-turn lanes shall be based on operational analyses using HCS. The length of the left-turn lanes shall be calculated per ***Figures 401-9*** **and** ***401-10***. Typically, any channelization for the left turn radii should be designed using a 10 to 15 mph design speed.

Main road left-turn movements can be prohibited when volumes are low and they can be accommodated at the downstream one-way median opening. This treatment is referred to as a Thru-Turn and does not preclude constructing a main road left turn in the future.

404.4.1.2 Minor Street Right Turn Lanes

Since the minor street left turn and through movements are being converted into right turning movements, the design of the right turn lanes must be sufficient to accommodate all traffic from the minor street. Based on the right turning movement volumes an operational analysis shall be performed to determine the number and length of the right turn lanes required.

The minor street approaches need to be designed to prevent wrong-way maneuvers. If the minor street is not a boulevard or a multi-lane divided facility, right-turn channels should be used to minimize wrong way maneuvers. The distance to the U-turn may need to be increased if the right turn is channelized. If multiple right turn lanes are provided, channelization of the lanes may help direct vehicles into the appropriate receiving lanes reducing initial driver confusion and downstream lane changes. A down-side to channelizing the right turn lanes may be the creation of imbalanced lane utilization.

Three options exist for handling the channelizing of the minor street traffic:

- No channelizing island
- A channelizing island (median) separating the traffic turning right from the minor street onto the major street and the traffic turning right from the major street onto the minor street.
- A channelizing island separating minor street right turns that will remain on the major street from the minor street right turns that subsequently makes a U-turn movement on the major street.

The advantages and disadvantages of right turn channelization on the minor street of an RCUT intersection are as follows:

Advantages:
- Drivers are more firmly guided, likely reducing sideswipe conflicts during the turn.
- Shortens pedestrian crossing distances to a pedestrian refuge.
- Reduces paved surface areas.
- Provides the opportunity for a lane addition and a free right turn (merge), reducing delay.

Disadvantages:
- Requires pedestrians to cross more vehicle pathways, with the right turns moving faster and/or more freely.
- Creates the potential for uneven lane utilization on the minor street.
- Drivers on the minor street must select a lane (preposition) earlier.
- Increases right-of-way to accommodate the channelization.

404.4.1.3 Right Turn (Minor Street to Major Street) without Acceleration Lanes

RCUT intersections with stop signs or signals controlling the minor streets can eliminate weaving movements and thus the need for acceleration lanes by extending the U-turn lane back to the main intersection. By providing pavement markings guiding drivers directly into the U-turn lane drivers can wait for an acceptable gap or a green signal. Extending the U-turn lanes to the main intersection is the preferred design and can shorten the distance between the minor street right turn and the median U-turn crossover.

404.4.1.4 Right Turn (Minor Street to Major Street) with Acceleration Lanes

RCUT intersections with acceleration lanes join the major road as an add-lane, then merge upstream of the U-turn crossover creating weaving movements. Field observations of RCUTs with acceleration lanes determined vehicles destined for the downstream U-turn tend to use the acceleration lane as little as possible. Vehicles tend to make the lane shifts as soon as an appropriate gap is available. Based on these tendencies the use of acceleration lanes upstream of the U-turn crossover should be discouraged especially at signalized RCUT intersections. If acceleration lanes are constructed the downstream U-turn is typically placed further away from the main intersection as compared to when acceleration lanes are not constructed.

404.4.2 Median U-turn Crossover

404.4.2.1 Crossover Spacing

The distance between the main intersection and the U-turn crossover must be designed for both directions of travel on the major street. The distance from the main intersection to the U-turn intersections should be kept as short as operational (queuing) and geometrical (location of nearby intersections, drives, bridges, right-of-way constraints and uphill grades etc.) conditions will allow. The U-turn lane can be extended back to the main intersection so the minor road left/thru vehicles destined for the U-turn can enter the U-turn lane as early as possible.

At a signalized RCUT intersection where the main intersection and the two U-turn crossovers are controlled by traffic signals the recommended spacing between the main intersection and the U-turn crossings ranges from a minimum of 600 ft. to a maximum of 1,000 ft. Several factors should be considered when selecting the appropriate spacing from the main intersection to the U-turn crossover. A longer spacing provides more time and space for driver maneuvers into the proper lane; as well as, read and respond to traffic control markings and highway signs. A shorter spacing translates into shorter driving distances and travel times for the minor street left turn and through movements.

In cases where the major road has higher speeds and volumes, the distance to the U-turn may need to be moved further away from the main intersection. The following steps are required to determine the distance from the main intersection to U-turn intersection:

1. Determine distance from minor road right turn to beginning of U-turn taper by determining the distance required to go from 0 mph to 10 mph below the main road design speed using the chart below.

Speed (mph)	25	30	35	40	45	50	55	60	65	70
Feet to reach speed	150	150	150	225	325	450	600	800	1000	1300

2. Determine the U-turn lane length based on ***Figure 401-9*** and ***Figure 401-10***, or HCS/Sim Traffic 95th percentile queue, whichever controls.

404.4.2.2 Crossover Design

Proper design of U-turns is essential to ensure safe traffic operations. A properly designed U-turn crossover should provide adequate pavement to accommodate a WB-62 without creating conflicts with other vehicles. For crossovers with inadequate median width for the WB-62 to maneuver loons or bulb-outs should be installed to provide adequate turning radii for the tracking of both the front and rear ends of a WB-62.

Four possible U-turn lane destinations exist to account for the truck tracking through the U-turn crossover.

- Inside Lane to Inside Lane
- Inside Lane to Outside Lane
- Inside Lane to Outside Shoulder
- Inside Lane to Loon

Typically, the U-turn crossovers should be designed using a design speed of 15 mph.

404.4.2.3 U-turn Lane Design

U-turn deceleration/storage lengths should be designed to accommodate the 95th percentile queue of U-turning vehicles. See ***Figures 401-9*** and ***Figure 401-10*** for determining the length of the U-turn deceleration/storage length at the approach to the crossover.

Larger vehicles have a larger swept path and may require an additional paved area. For dual U-turn, the U-turn crossover should accommodate one WB-62 and one single unit truck vehicle. Signage should be provided for the WB-62 to use the rightmost. or outermost U-turn lane. The crossover must be designed so the path of the WB-62 in the rightmost lane and the path of the single unit truck in the leftmost lane provides at least a minimum of 2 ft. between the tire paths of the respective vehicles.

404.5 Multimodal Considerations

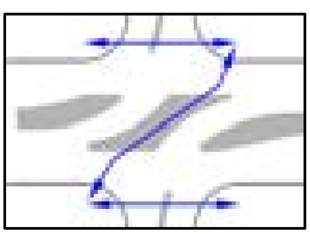
RCUT intersection design should consider all the different transportation modes that will be using the intersection.

Due to their unique geometrics and traffic control features, RCUT intersections can provide both benefits and challenges to non-automobile modes of transportation and these factors need to be considered early in the project development process rather than incorporating them in the later stages of design.

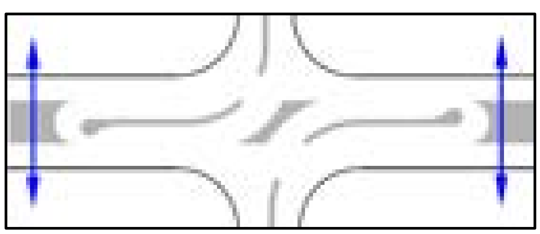
404.5.1 Pedestrians

Factors such as non-signalized movements, a greater percentage of vehicles making right turning movements and the RCUT intersections wide geometric footprint create challenges to accommodating pedestrians. Pedestrian crossing lengths may be longer than those at conventional intersections, but the shorter traffic signal cycle lengths can help make crossing times comparable to those at conventional intersections.
RCUT intersections reduce the number of vehicle-pedestrian conflict points compared to conventional intersections, but they also require pedestrian crossings that differ from conventional intersections. The most common means of serving pedestrians is a “Z” crossing treatment. Other options include a Signalized Mid-Block Crossing and a Median Crossing. Examples of each pedestrian crossing treatment are shown below.

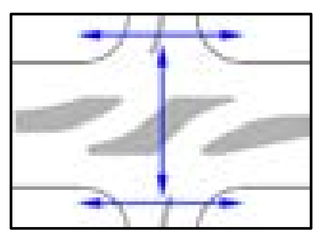
“Z”



Signalized Mid-Block



Median Cross



404.5.2 Bicycles

Bicycles on the major street navigate an RCUT intersection the same way they navigate a conventional intersection. Minor street left-turning or through bicycles do not have a direct route at an RCUT intersection if they are utilizing the vehicular lanes. There are three primary ways to facilitate minor street left-turn and through bicyclists in an RCUT intersection:

- Use a path matching the pedestrians
- Use a path matching the motor vehicle traffic

- Direct bicycle crossings

The choice of which option will depend on the quality of interaction with motorists, the speed of the major street vehicle traffic, shoulder width or the presence of a bicycle lane, the volume of the major street traffic, the distance to the U-turn crossover and the level of bicyclist experience.

LIST OF FIGURES

* Note: For the design criteria pertaining to Collectors and Local Roads with ADT's of 400 or less, refer to the AASHTO Publication - Guidelines for Geometric Design of Very Low-Volume Local Roads ADT≤400).

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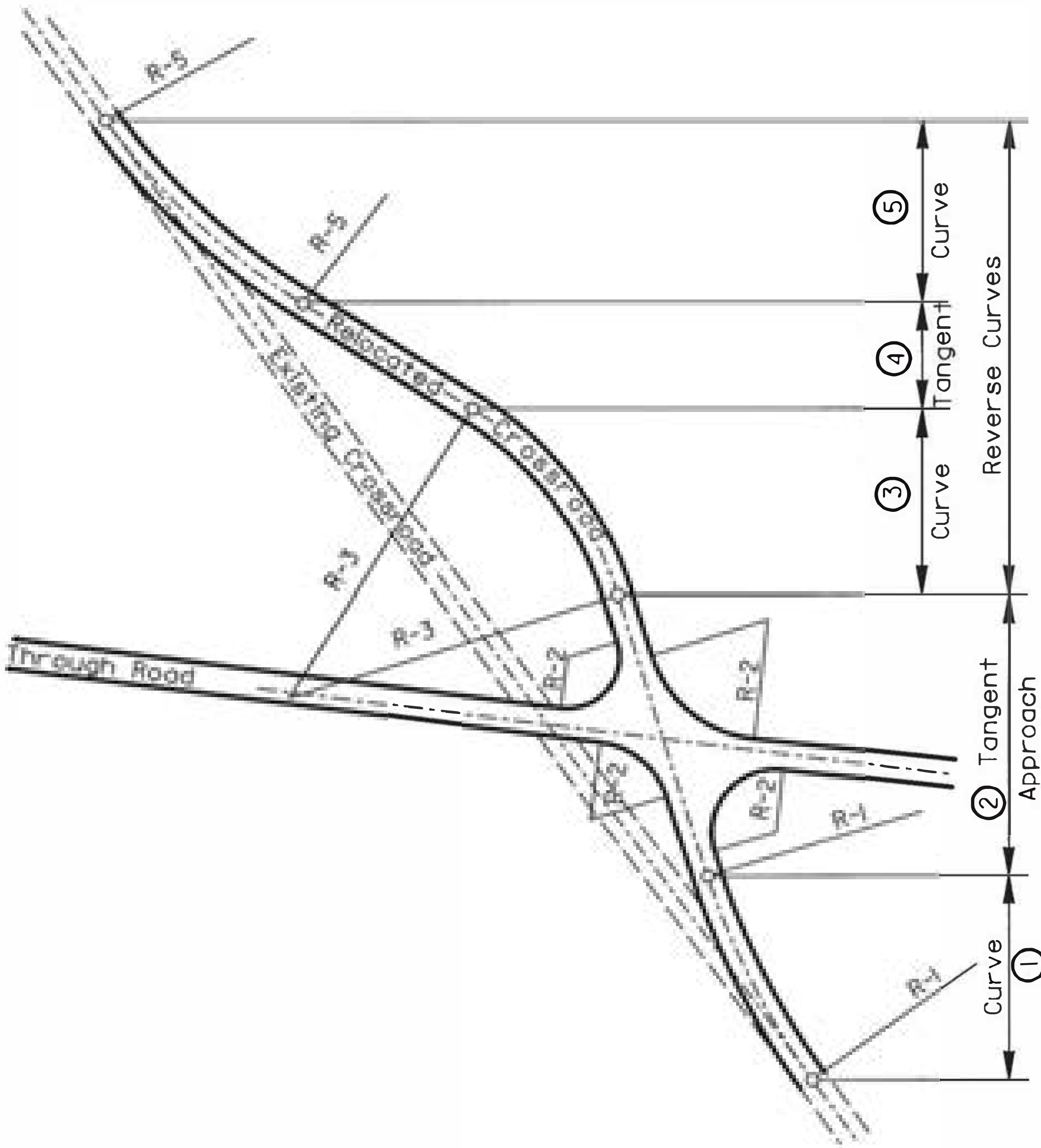
TYPICAL CROSSROAD RELOCATION	401-1
	REFERENCE SECTION 401.3



See explanation on adjacent sheet.

Explanation of Figure 401-1 Typical Crossroad Relocation

1. Curve - This portion of the crossroad can occur by itself at “T” type or three-legged intersections. If possible, the radius of this curve should be commensurate with the design speed of the crossroad. Often, the length of the required profile controls the work length. The horizontal curvature is then chosen so it can be accomplished within this work length. Regardless of the length of the profile adjustment, it is desirable to provide at least a 230 foot radius for this curve. When a 230 foot radius incurs high costs, it is permissible to reduce this radius to a minimum of 150 ft.

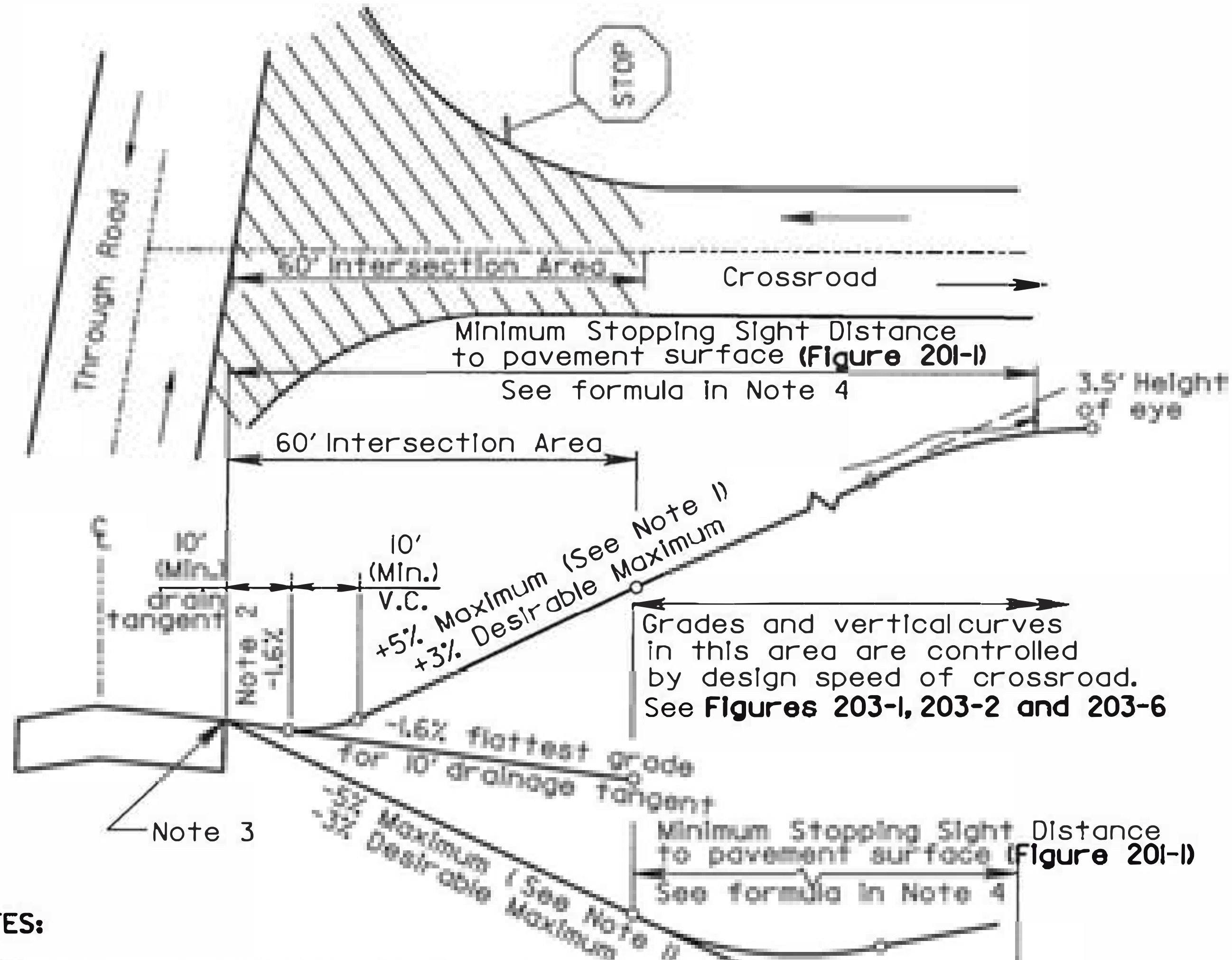
2. Tangent and Approach Radii - The crossroad in this area should have a tangent alignment. For the condition shown, the alignment between the radius returns is tangent from one side of the road to the other. However, at some intersections with a minor through movement (for example, crossroad intersections of standard diamond ramps) it may be desirable to provide different intersection angles on each side of the through road. For approach radii, see discussion in ***Section 401.5.***

3. Curve - The statements in (1) above also apply to this curve. With the reverse curve condition shown, the radius will often not exceed 250 ft. because flatter curves make the relocation extraordinarily long.

4. Tangent - This tangent should be approximately 150 ft. in length for 30 or 40 mph design speeds on the existing road, and approximately 250 ft. for 50 or 60 mph design speeds. These lengths are generous enough to allow reasonable superelevation transitions between the reverse curves. In general, it is usually not desirable to make this tangent any longer than required. If a longer tangent can be used, the curvature or intersection angle can be improved and these two design items are more important.

5. Curve - This curve should be much flatter than the other two curves. It should be capable of being driven at the normal design speed of the existing crossroad.

CROSSROAD PROFILE -STOP CONDITION- THROUGH ROAD NORMAL CROWN	401-2
	REFERENCE SECTIONS 401.4.1 - 401.4.4



NOTES:

1. 5% maximum grades are permitted and shown for illustration but grades of 3% or less are preferred.
2. Grade to be 1.6% or steeper to provide drainage away from through road pavement.
3. Recommended crest break 5% sag break 3%. If break is greater see Figure 401-3
4. Crest Vertical Curve Length (Height of eye 3.5'- height of object 0')

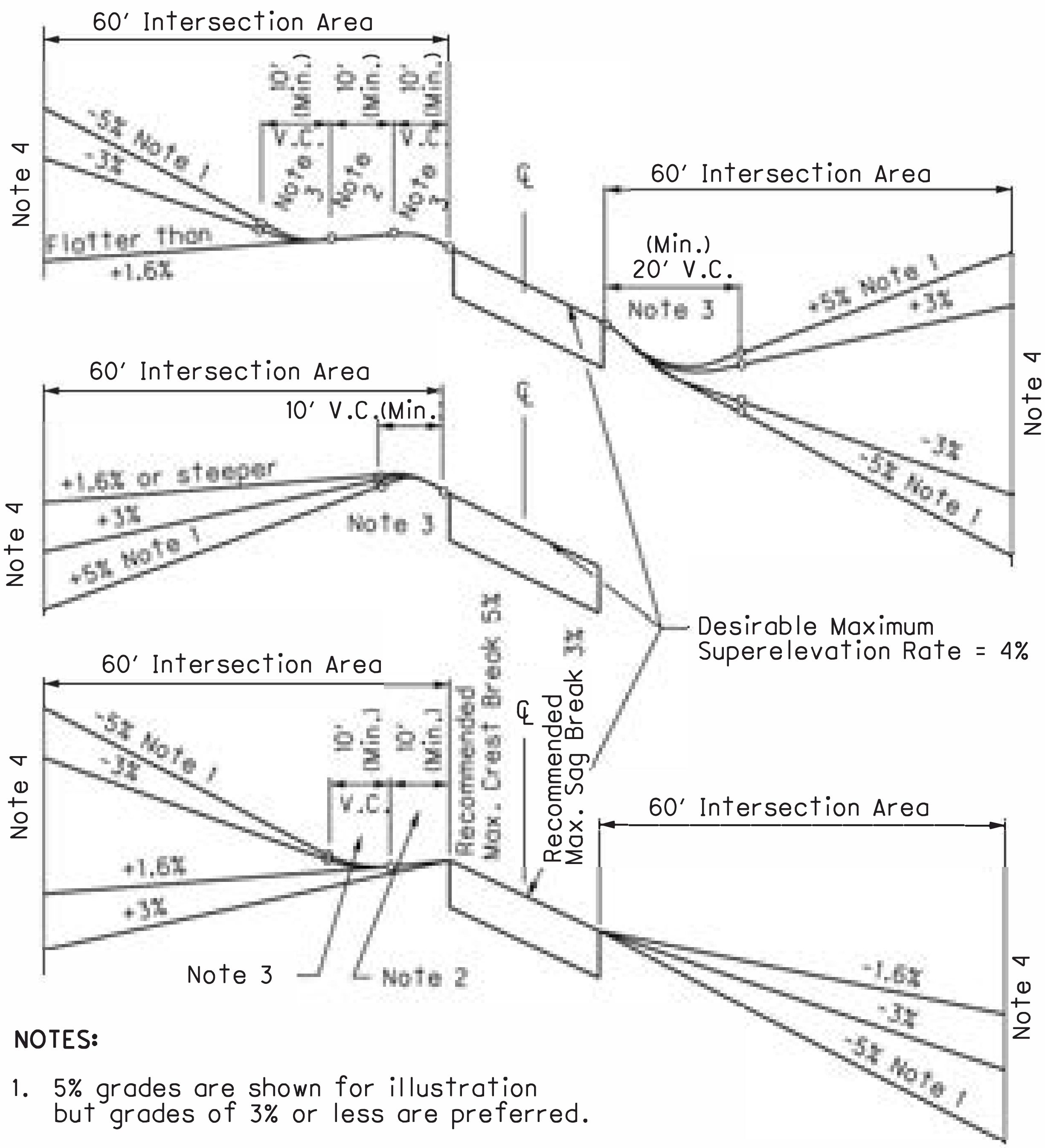
   $S<L \quad L=\frac{AS^2}{700}$

   $S>L \quad L=2S-\frac{700}{A}$

CROSSROAD PROFILE -STOP CONDITION- THROUGH ROAD SUPERELEVATED

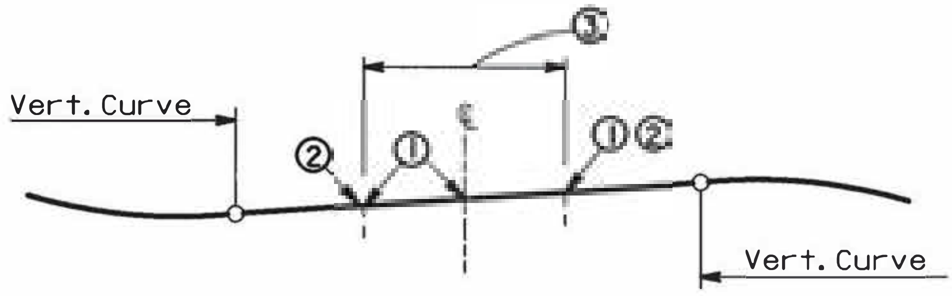
401-3

REFERENCE SECTION 401.1 & 401.4.3



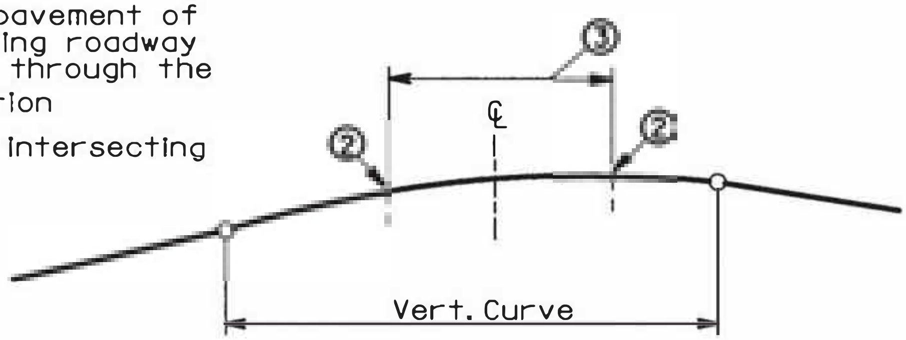
NOTES:

1. 5% grades are shown for illustration but grades of 3% or less are preferred.
2. Grade to be 1.6% or steeper to provide drainage.
3. Crest breaks exceeding 5% shall be rounded using vertical curves having a K of 1 or greater. Sag breaks exceeding 3% shall be rounded using vertical curves having a K of 1.5 or greater
4. For grade treatment of this area, see **Figure 401-2.**

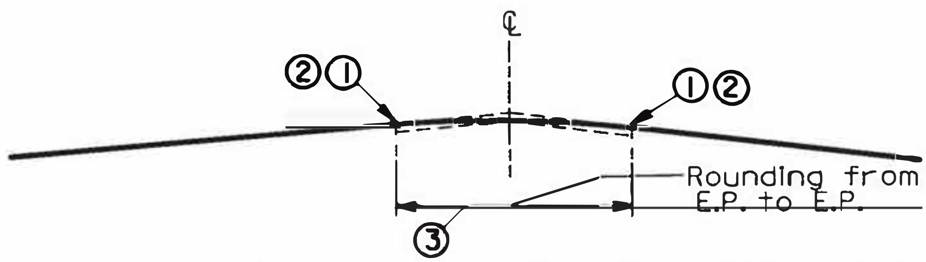


Example A - Crossroad Profile Tangent through Intersection

1. Location of permissable grade break per **Figure 203-2**
2. Edge of pavement of intersecting roadway extended through the intersection
3. Width of intersecting roadway



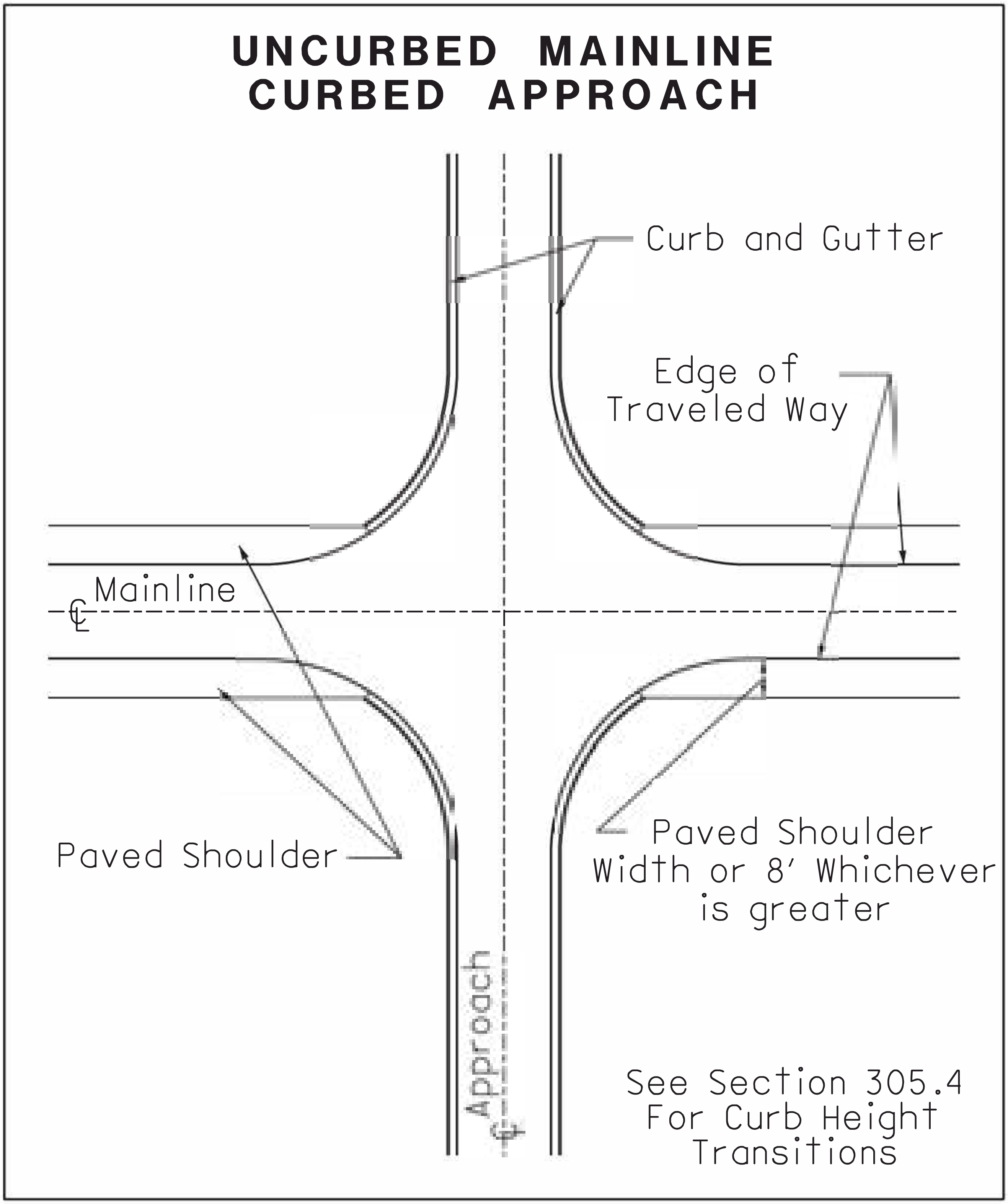
Example B - Crossroad Profile on Vertical Curve through Intersection



Example C - Crossroad Profile Fitted to a Normal Crown on the Mainline Road

RADIUS RETURNS FOR UNCURBED MAINLINE CURBED APPROACH

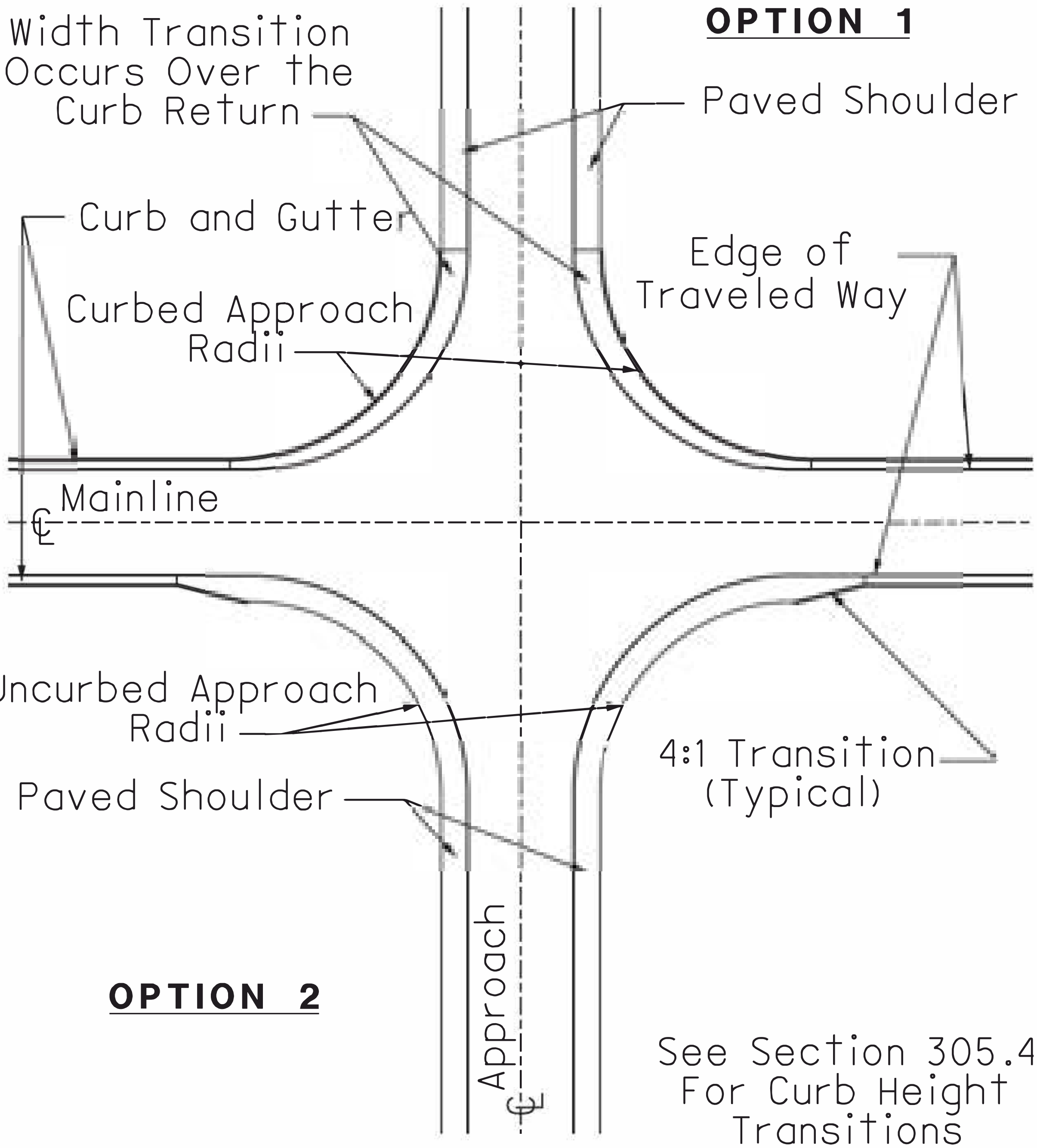
401-4a
REFERENCE SECTION 401.5.3 & 305.4



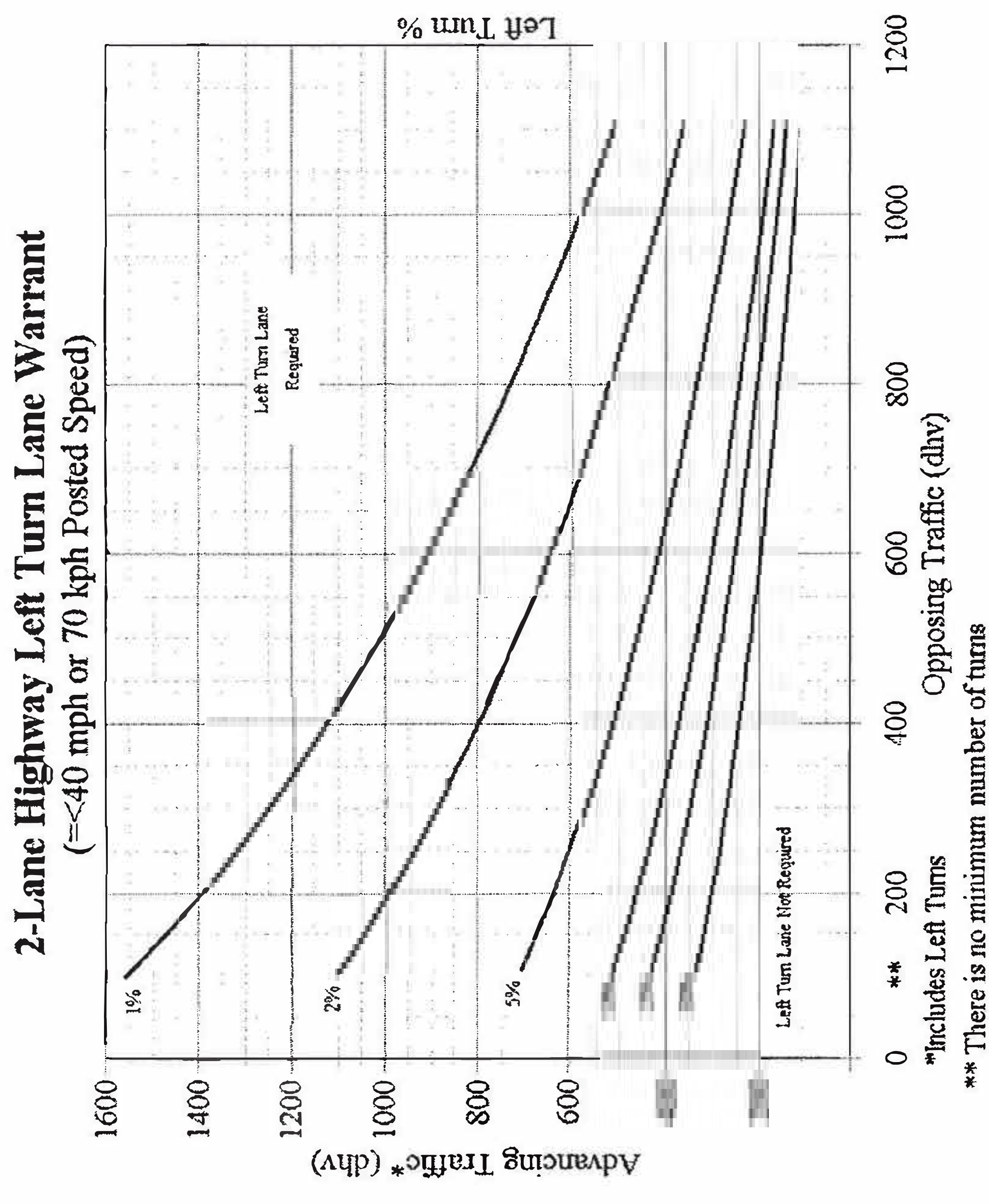
RADIUS RETURNS FOR CURBED MAINLINE UNCURBED APPROACH

401-4b
REFERENCE SECTION 401.5.3 & 305.4

CURBED MAINLINE UNCURBED APPROACH



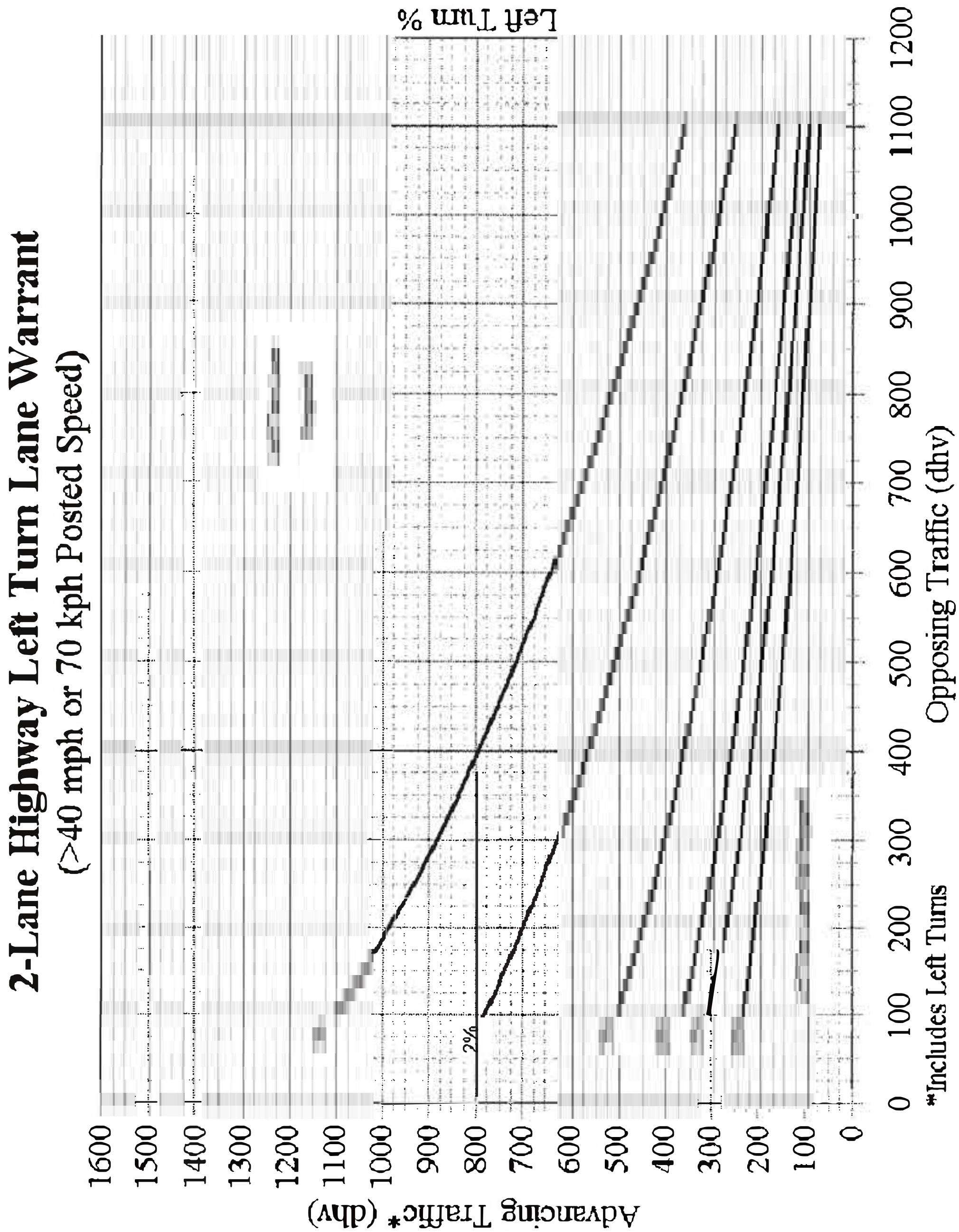
2-LANE LEFT TURN LANE WARRANT (LOW SPEED)	401-5a
	REFERENCE SECTION 401.6.1



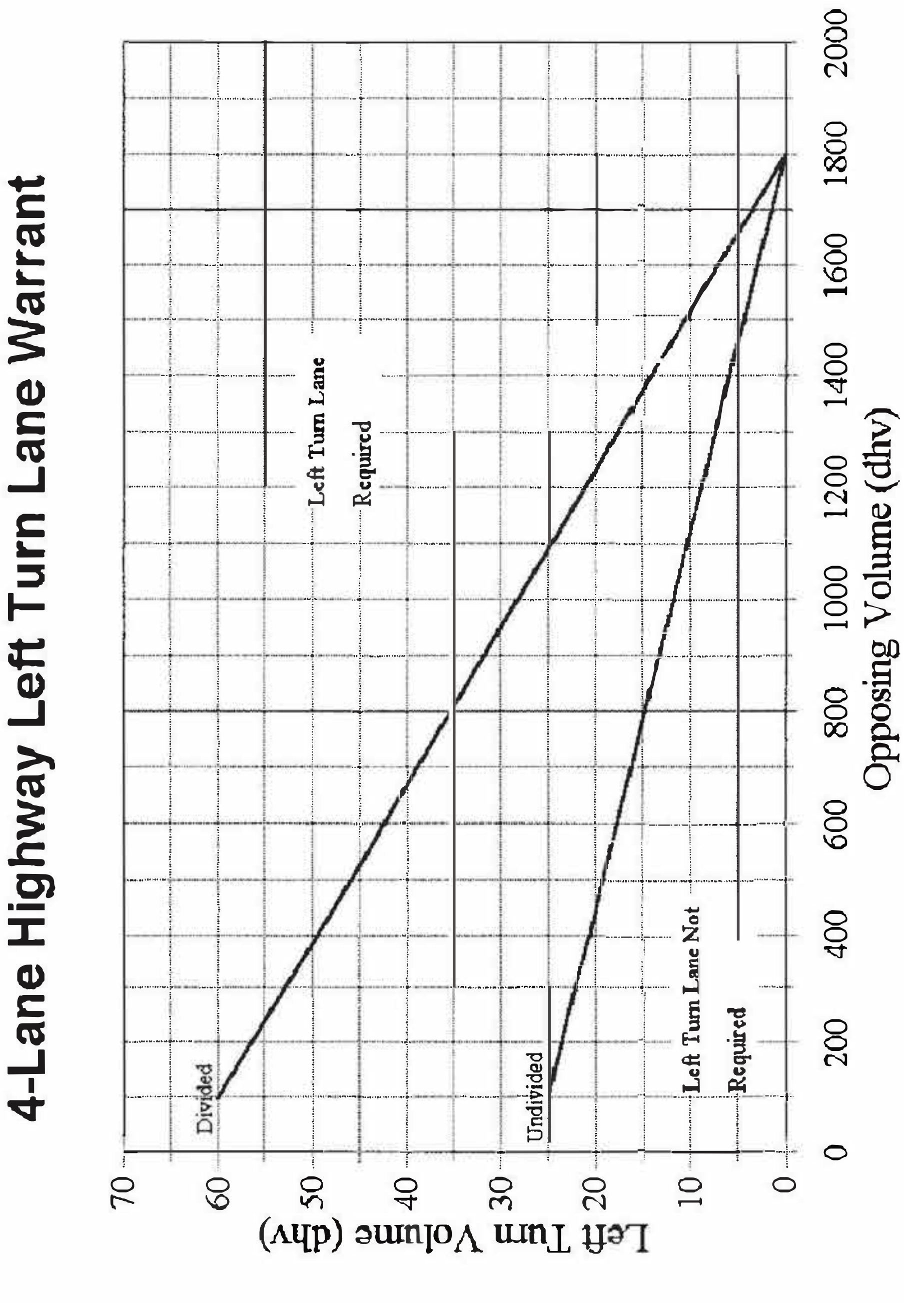
2-LANE LEFT TURN LANE WARRANT (HIGH SPEED)

401-5b

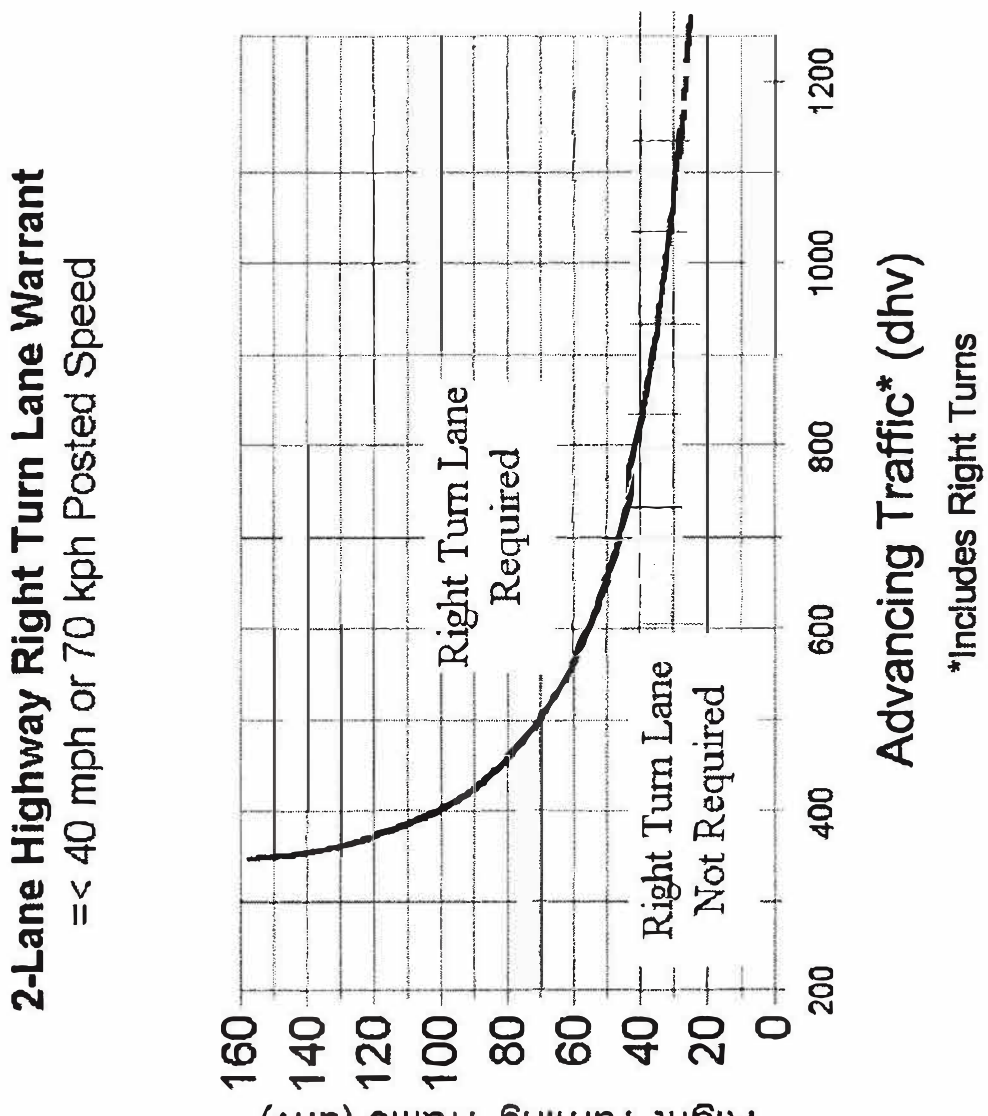
REFERENCE SECTION 401.6.1



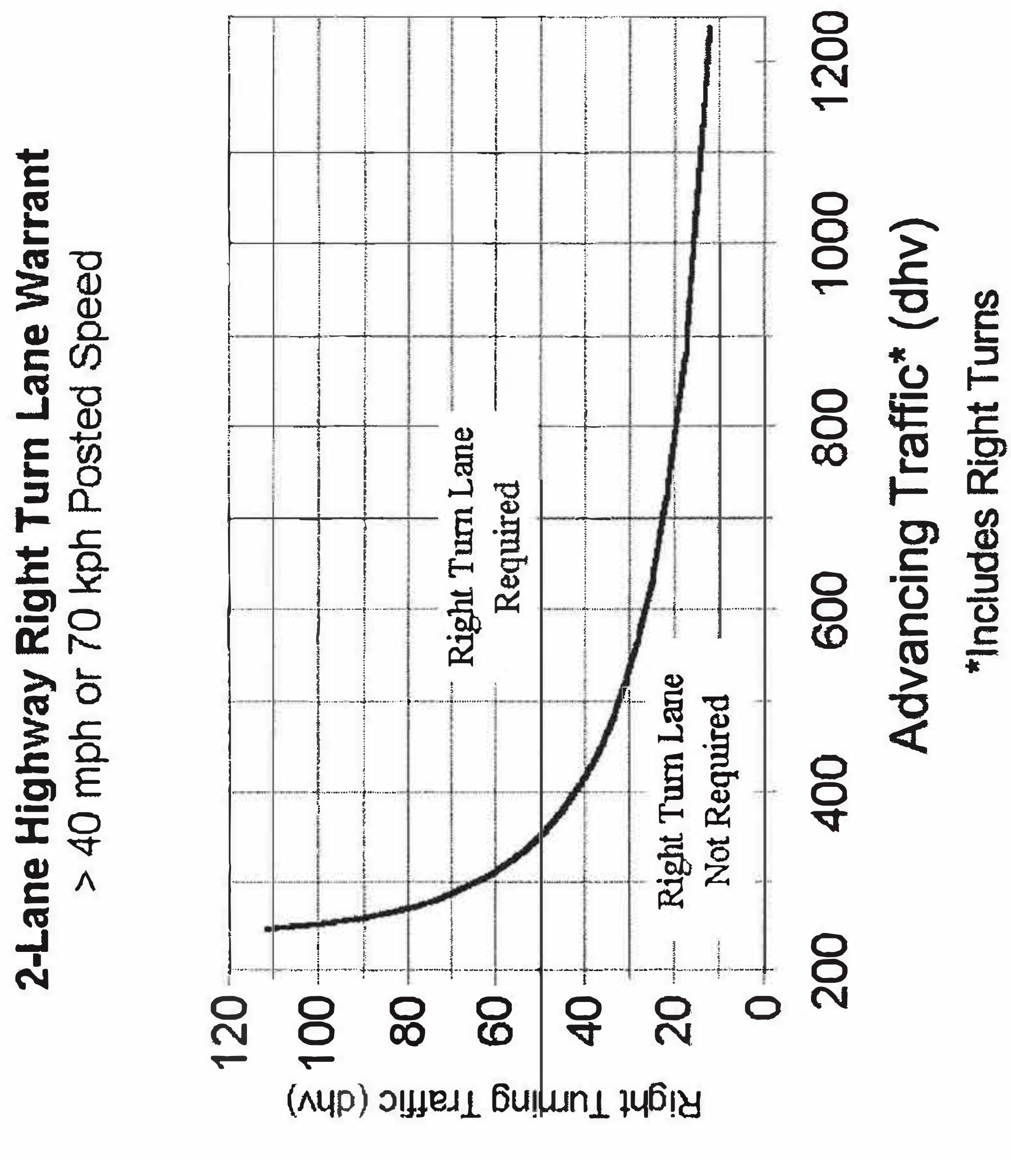
4-LANE LEFT TURN LANE WARRANT	401-5c
	REFERENCE SECTION 401.6.1



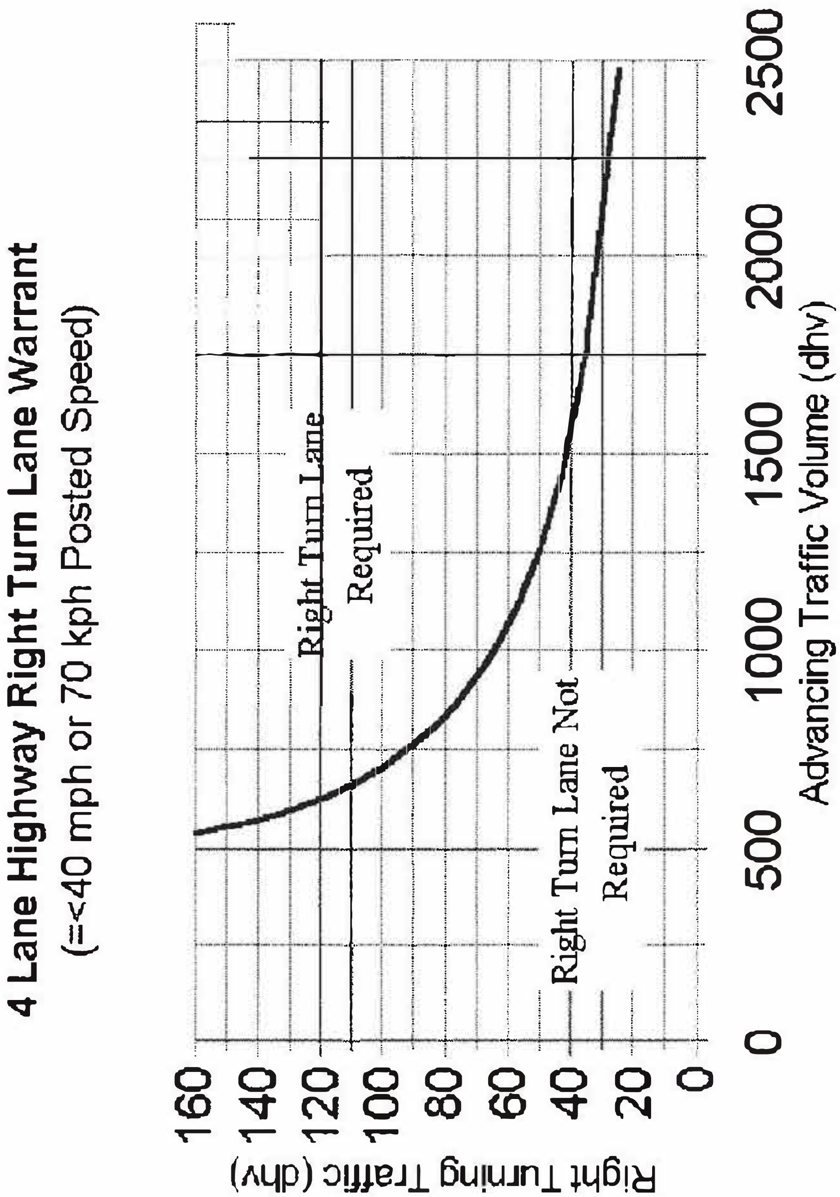
2-LANE RIGHT TURN LANE WARRANT (LOW SPEED)	401-6a
	REFERENCE SECTION 401.6.3



2-LANE RIGHT TURN LANE WARRANT (HIGH SPEED)	401-6b
	REFERENCE SECTION 401.6.3



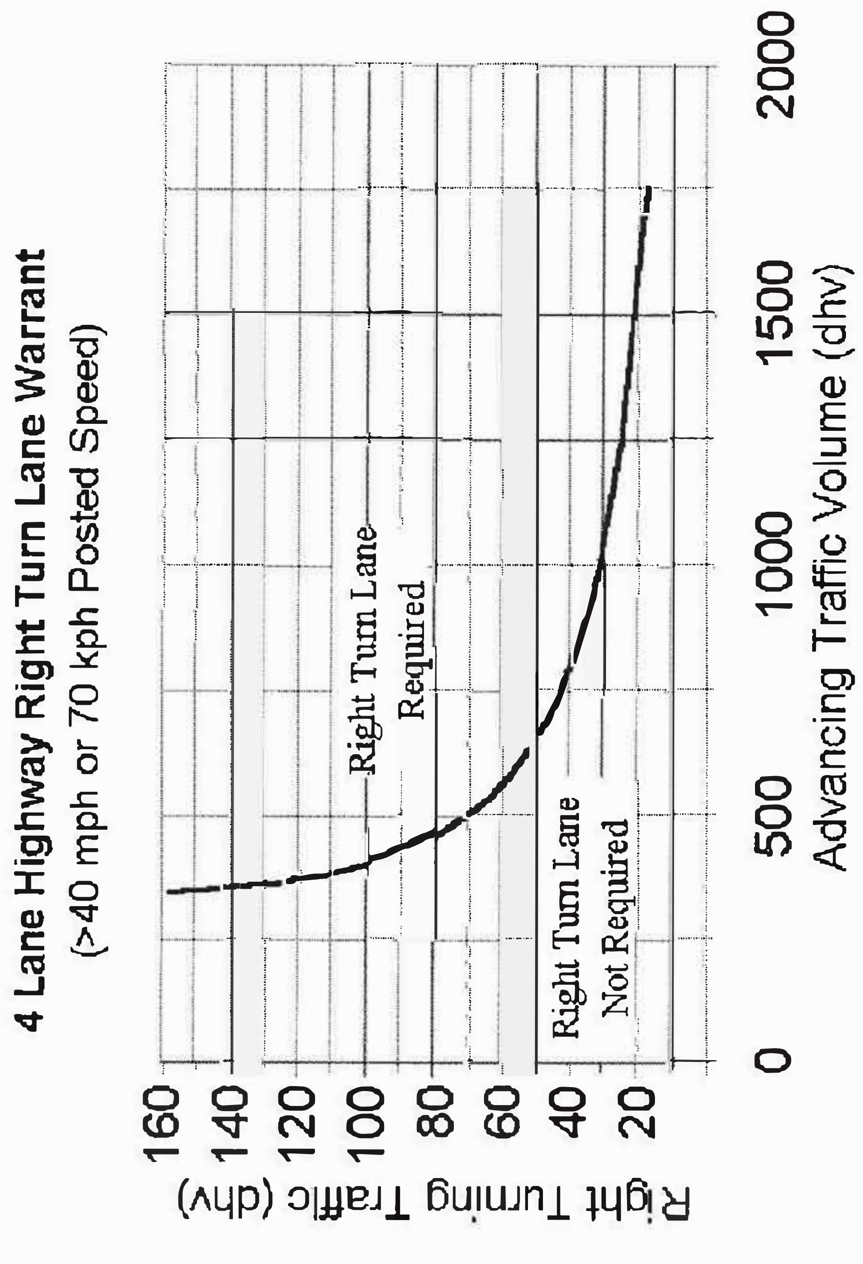
4-LANE RIGHT TURN LANE WARRANT (LOW SPEED)	401-6c
	REFERENCE SECTION 401.6.3



4-LANE RIGHT TURN LANE WARRANT (HIGH SPEED)

401-6d

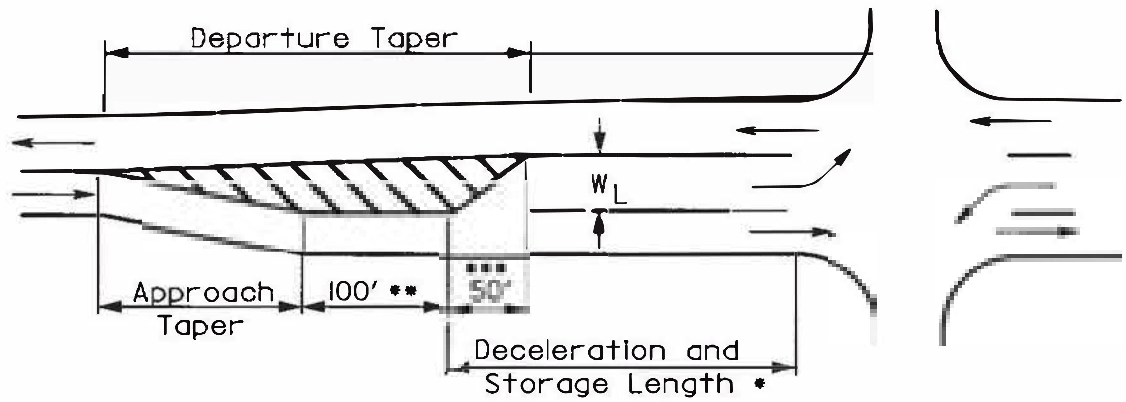
REFERENCE SECTION 401.6.3



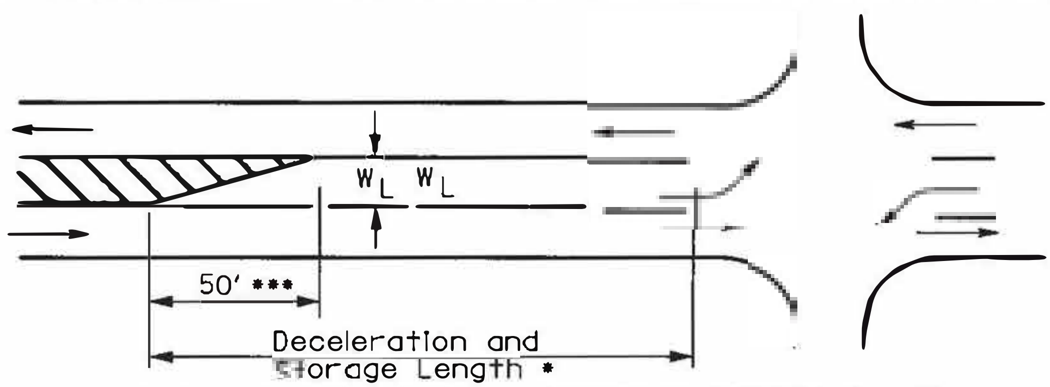
TURNING LANE DESIGN

401-7

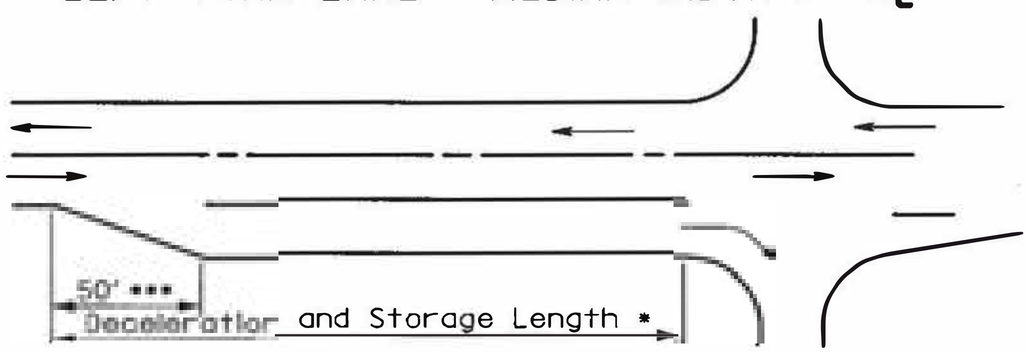
REFERENCE SECTIONS
401.6.1, 401.6.3



LEFT TURN LANE - NO MEDIAN OR MEDIAN WIDTH < W_L



LEFT TURN LANE - MEDIAN WIDTH >= W_L



RIGHT TURN LANE

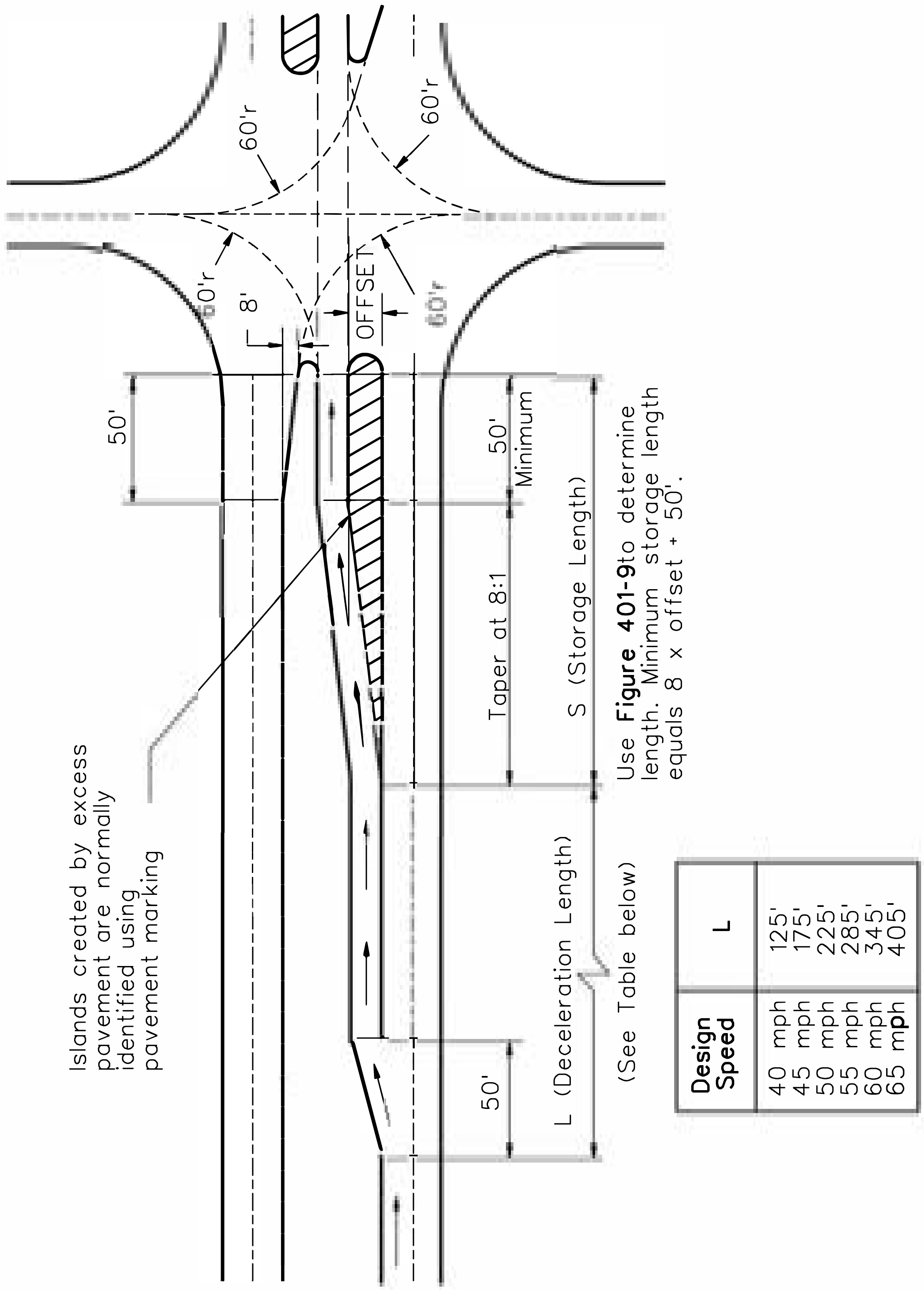
- * See **Figures 401-9 and 401-10** to copmpute length.
- ** May be reduced or eliminated in urban areas if intersection spacing or storage is constraining
- *** Diverging taper

W_L = Turn Lane Width

OFFSET LEFT TURN LANE

401-8

REFERENCE SECTIONS
401.6.1, 401.6.3



Design Speed	L
40 mph	125'
45 mph	175'
50 mph	225'
55 mph	285'
60 mph	345'
65 mph	405'

BASIS FOR COMPUTING LENGTH OF TURN LANES	401-9
	REFERENCE SECTIONS 401.6.1, 401.6.3

Type of Traffic Control	Design Speed		
	30-35	40-65	
	Turn Demand Volume		
	All	Low*	High
Signalized	A	** B or C	** B or C
Unsignalized Stopped Crossroad	A	A	A
Unsignalized Through Road	A	B	** B or C

*Low is considered 10% or less of approach traffic volume
**Whichever is greater

CONDITION A	STORAGE ONLY
Length = 50' (diverging taper) + Storage Length (Figure 401-10)	

CONDITION B	HIGH SPEED DECELERATION ONLY
Design Speed	Length (including 50' Diverging Taper)
40	125
45	175
50	225
55	285
60	345
65	405

CONDITION C	MODERATE SPEED DECELERATION AND STORAGE
Design Speed	Length (including 50' Diverging Taper)
40	115 + Storage Length (Figure 401-10)
45	125 "
50	145 "
55	165 "
60	185 "
65	205 "

For explanation, see Turn Lane Design Example

STORAGE LENGTH AT INTERSECTIONS

401-10

REFERENCE SECTIONS 401.6.1, 401.6.3

* AVERAGE NO. OF VEHICLES/CYCLE	REQUIRED LENGTH (FT.)	* AVERAGE NO. OF VEHICLES/CYCLE	REQUIRED LENGTH (FT.)
1	50	17	600
2	100	18	625
3	150	19	650
4	175	20	675
5	200	21	725
6	250	22	750
7	275	23	775
8	325	24	800
9	350	25	825
10	375	30	975
11	400	35	1125
12	450	40	1250
13	475	45	1400
14	500	50	1550
15	525	55	1700
16	550	60	1850

$$\text{* AVERAGE VEHICLES PER CYCLE} = \frac{\text{DHV (TURNING LANE)}}{\text{CYCLES/HOUR}}$$

<u>IF CYCLES ARE UNKNOWN ASSUME:</u>

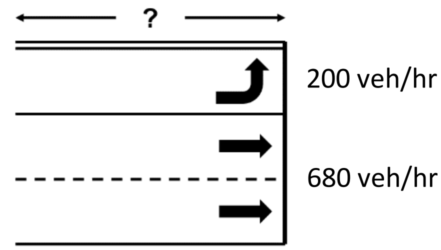
UNSIGNALIZED OR 2 PHASE = 60 CYCLES/HOUR
3 PHASE = 40 CYCLES/HOUR
4 PHASE = 30 CYCLES/HOUR

Example - Turn Lane Design Using Figures 401-9 and 401-10

Problem

Calculate the length of an exclusive left turn lane.

Traffic Control: **Signalized**
Design Speed: **55 mph**
Cycle Length: **90 sec**



Determine Storage and Turn Lane Lengths

Turn Lane Demand (High/Low) $= \dfrac{200 \text{ veh/hr}}{200 \text{ veh/hr} + 680 \text{ veh/hr}} = 23\%$ = **High Demand**

Refer to the matrix in **Figure 401-9**.

For Signalized, 55 mph, High Demand, use Method B or C, whichever is greater.

Method B – For 55 mph, a **285’** turn lane length is required (235’ storage + 50’ taper).

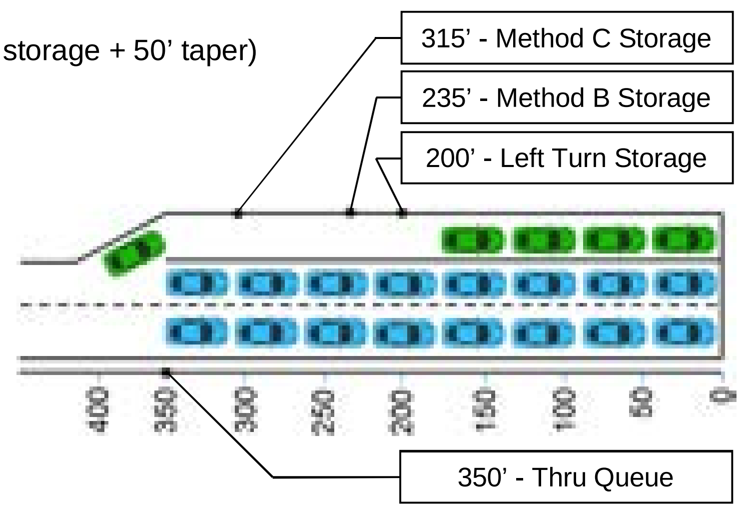
Method C – For 55 mph, 165’ + calculated storage length in **Figure 401-10**.

Average Vehicles per Cycle $= \dfrac{(200 \text{ veh/hr}) * (90 \text{ sec/cyc})}{3600 \text{ sec/hr}} = 5 \text{ veh/cyc} \rightarrow 200'$

Total Length = 165’ + 200’ = **365’** (315’ storage + 50’ taper)

Method C = **365’** > Method B = **285’**

Use Method C



Check Length for Thru-Block

Refer to **Figure 401-10** to calculate thru lane(s) queue distance.

680 veh/hr / 2 lanes = 340 veh/hr/ln

Average Vehicles per Cycle $= \dfrac{(340 \text{ veh/hr/ln}) * (90 \text{ sec/cyc})}{3600 \text{ sec/hr}} = 9 \text{ veh/cyc/ln} \rightarrow$ **350 ft/ln**

Thru Block = **350’** > Method C Storage = **315’** → **Turn Lane Blocked**

Use **350’** storage + **50’** taper = **400’ Turn Lane Length**

DOUBLE LEFT TURN LANES

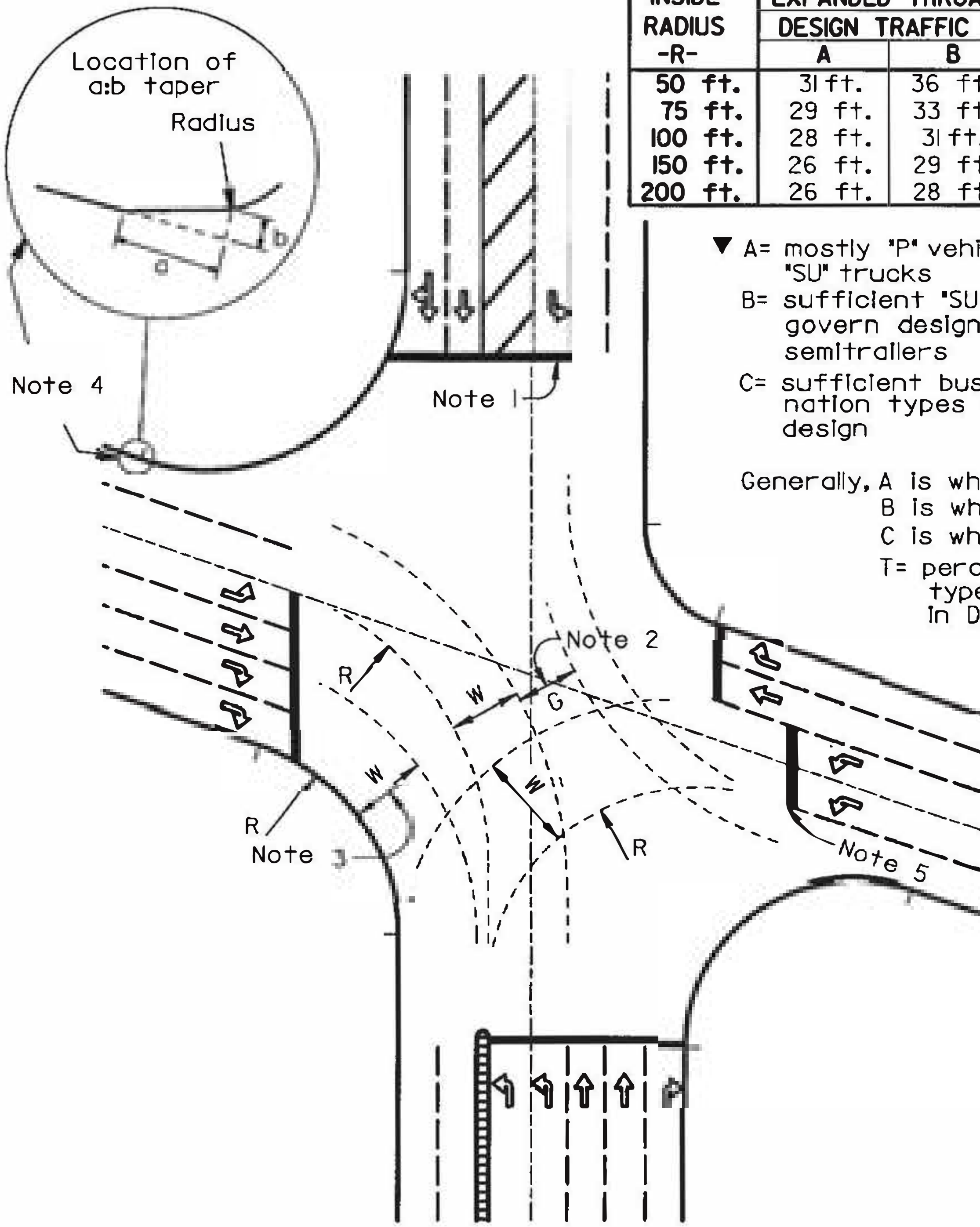
401-11

REFERENCE SECTIONS
401.6.2, 401.6.4

INSIDE RADIUS -R-	EXPANDED THROAT WIDTH -W- DESIGN TRAFFIC CONDITION ▼		
	A	B	C
50 ft.	31 ft.	36 ft.	45 ft.
75 ft.	29 ft.	33 ft.	38 ft.
100 ft.	28 ft.	31 ft.	35 ft.
150 ft.	26 ft.	29 ft.	32 ft.
200 ft.	26 ft.	28 ft.	30 ft.

▼ A= mostly "P" vehicles, some "SU" trucks
B= sufficient "SU" trucks to govern design, some semitrailers
C= sufficient bus and combination types to govern design

Generally, A is when T < 5%
B is when T = 5-10%
C is when T > 10%
T= percentage of type B and C trucks in Design ADT



Notes for Figure 401-11 Double Left Turn Lanes

1. Notice that the single left turn lane at the top of the page has been laterally offset from the through lanes in order to prevent conflicts between opposing turning paths.

2. Opposing turning paths should always be checked to verify that there is no conflict (see dimension "G"). Per AASHTO "Green Book", page 9-138, dimension "G" should be at a minimum, 10'.

3. The double right turn lane design follows the same criteria as the double left turn lane for expanded throat width.

4. The pavement width of the receiving lanes for a double left turn at an intersection needs to be checked to see if design vehicles can complete their turns within the pavement area. This is especially important where the radius returns are curbed. The use of radius templates is one method that can be used to check wheel tracking to see if additional pavement area adjacent to the far return area is needed. If the turning lanes are 12 ft. in width, the following formula is recommended to estimate a need for widening the pavement at the receiving throat:
   $F = (W-24)/2$
   where W is the maximum expanded throat width from the table on ***Figure 401-11.*** If the turn lanes are not 12 ft., use truck turning templates.
   The use the following guidelines:

   If $F < 2.0$, no widening is required.
   If $F = 2.0$ through 3.9, use a 40:4 taper.
   If $F = 4.0$ through 5.9, use a 45:6 taper.
   If $F = 6.0$ through 9.0, use a 50:8 taper.

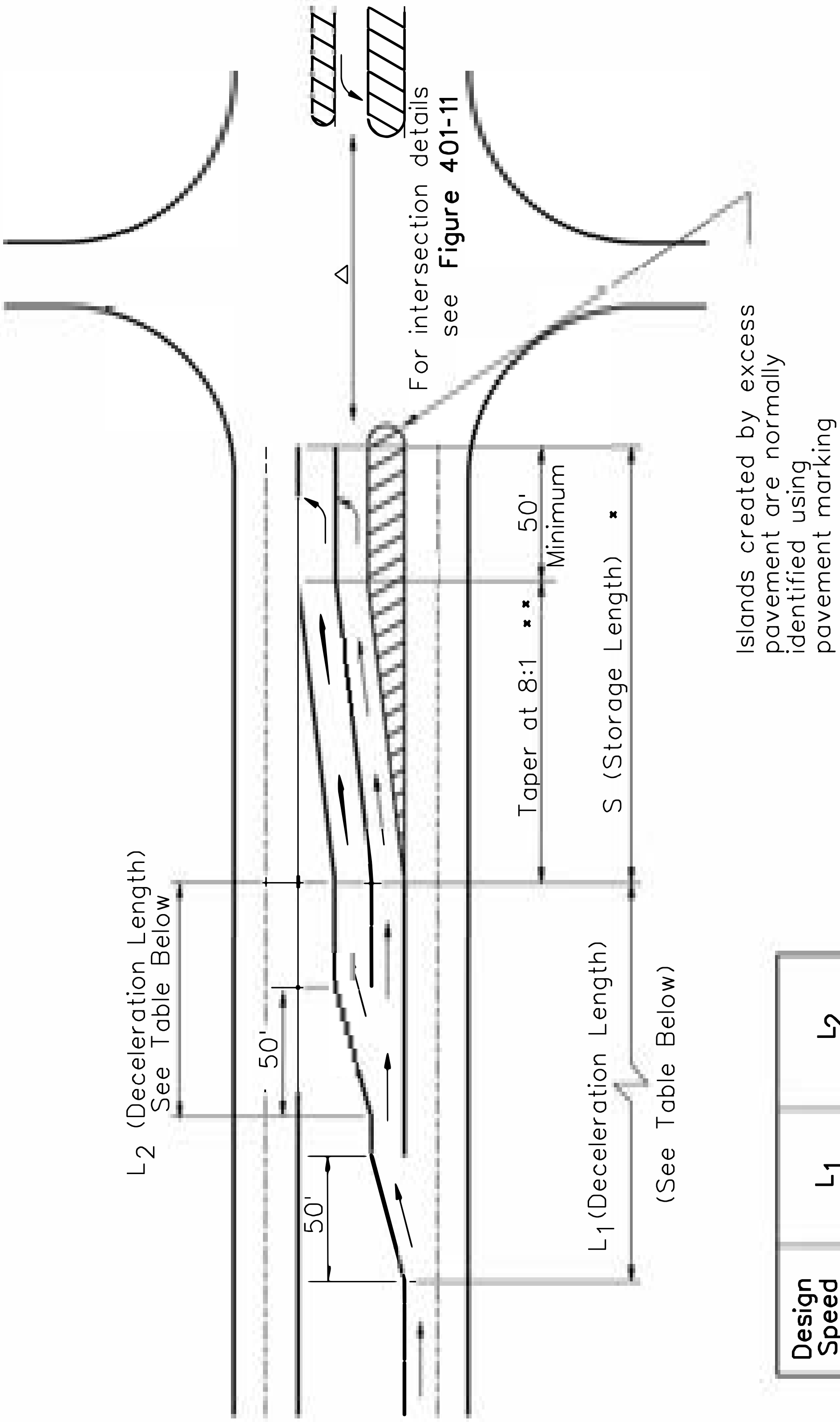
   See ***Figure 503-5*** for examples of how these tapers are used at radius returns.

5. Stop bar locations may need to be adjusted to the inside radius return of the left turn movements.

DEVELOPMENT OF DUAL LEFT TURN LANES

401-12

REFERENCE SECTION 401.6.2



* Use Figure 401-10 to determine storage length. For offset left turn lanes, minimum storage length equals 8 x offset + 50'.

** Taper is used when dual left turn lanes are offset.

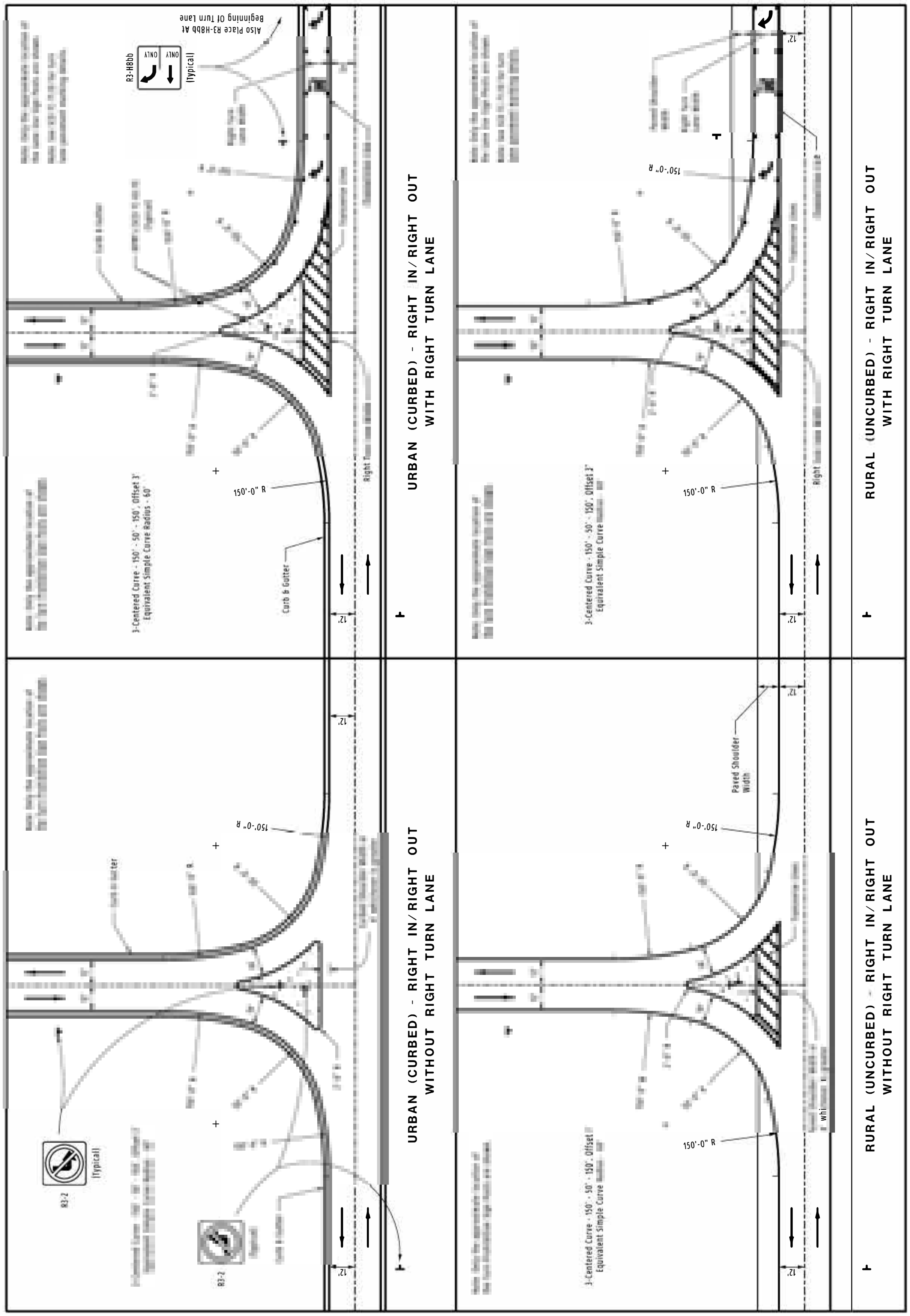
Δ If opposite approach has one left turn lane, these lanes should line up.

Design Speed	L_1	L_2
40 mph	125'	75'
45 mph	175'	125'
50 mph	225'	145'
55 mph	285'	165'
60 mph	345'	185'
65 mph	405'	205'

CHANNELIZING ISLANDS PASSENGER CAR DESIGNS

401-13a

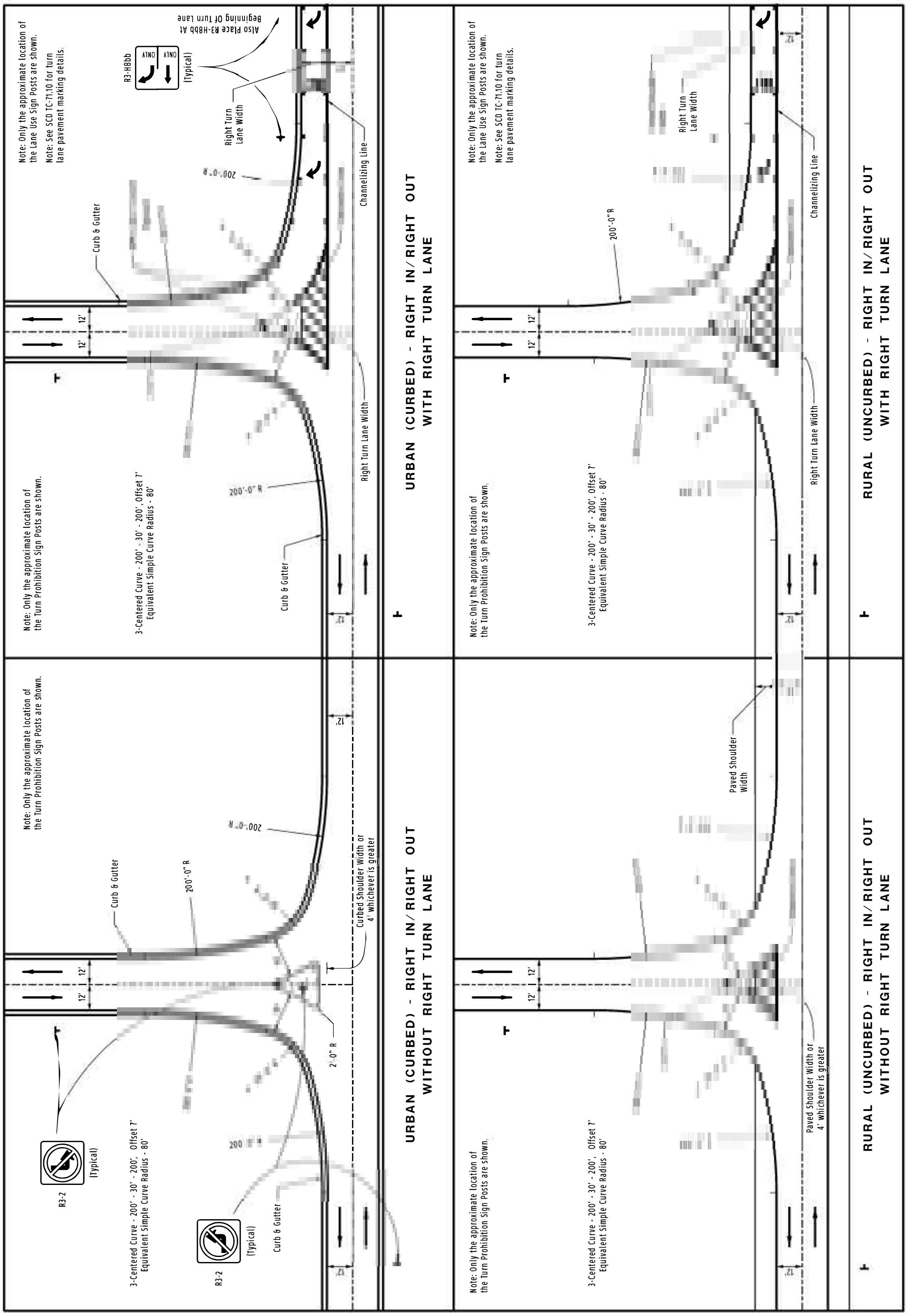
REFERENCE SECTION 401.7.2



CHANNELIZING ISLANDS SU TRUCK DESIGNS

401-13b

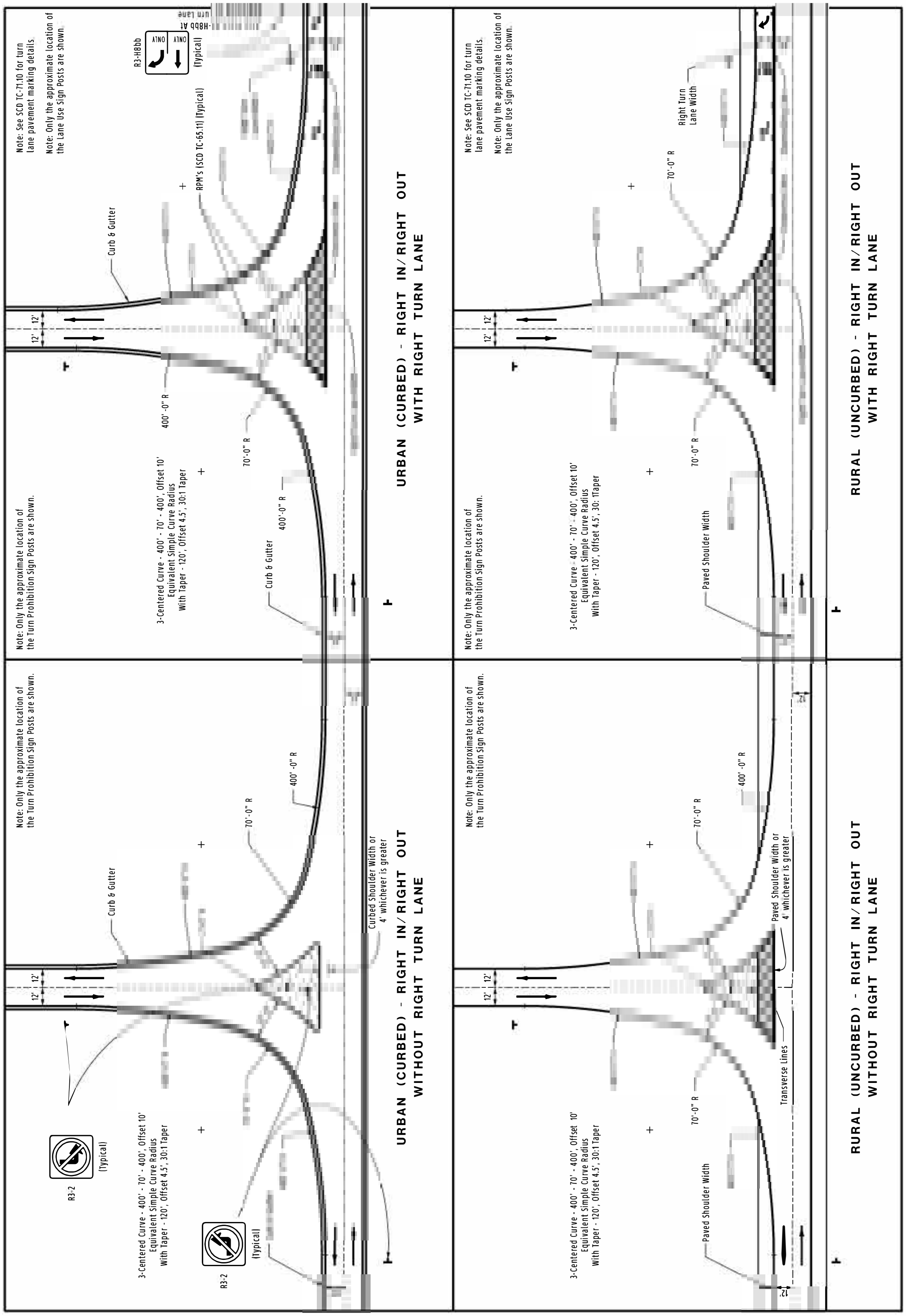
REFERENCE SECTION 401.7.2



CHANNELIZING ISLANDS WB-62 TRUCK DESIGNS

401-13c

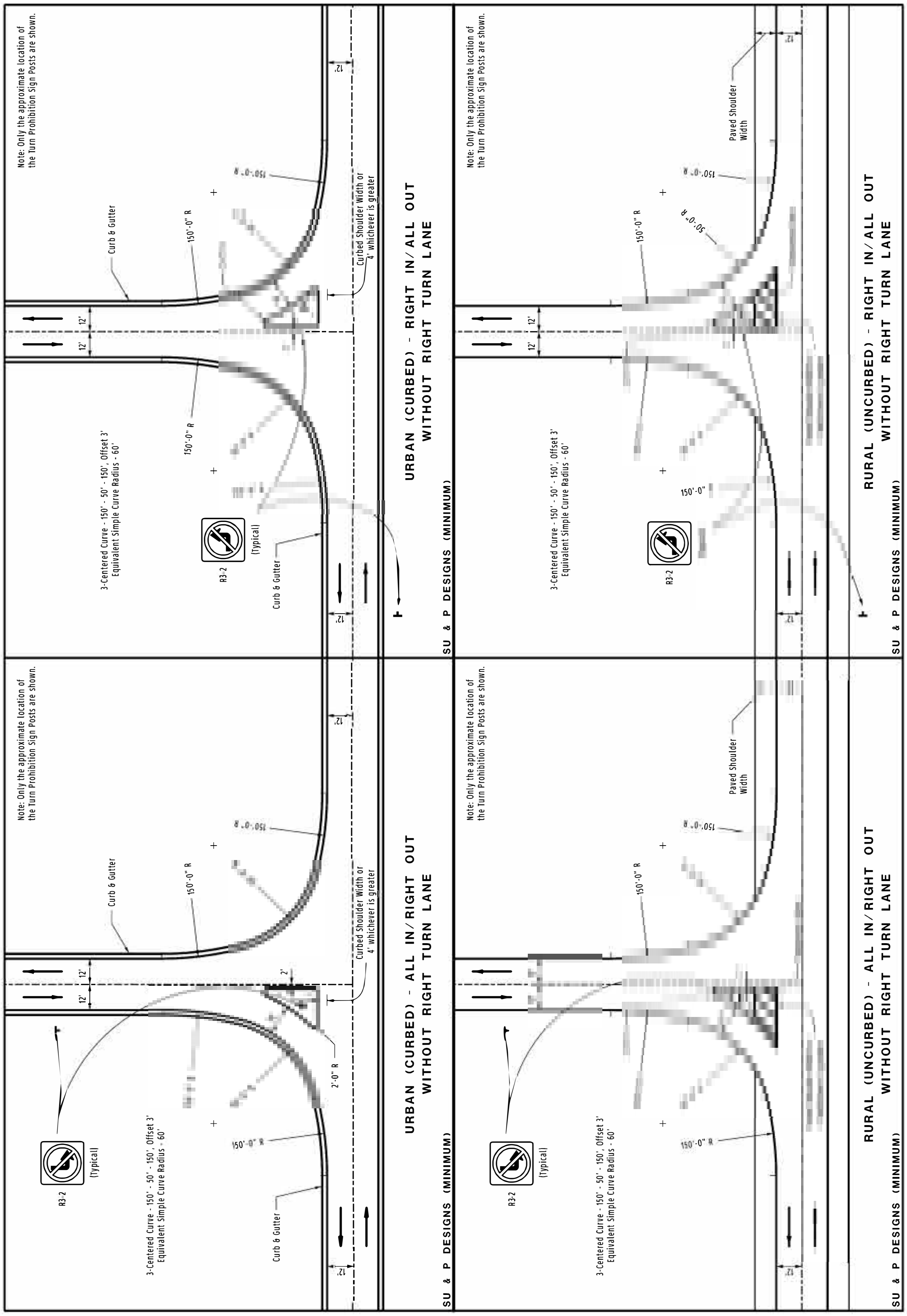
REFERENCE SECTION
401.7.2



CHANNELIZING ISLANDS TYPICAL ISLANDS WITH PERMITTED LEFT TURNS

401-13d

REFERENCE SECTION 401.7.2



TRAFFIC OPERATIONAL ANALYSIS DESIGN SOFTWARE	401-14aE
	REFERENCE SECTION 401.2

Traffic operational analysis used for design purposes (i.e. determination of number and type of lanes) must use the current version of the Highway Capacity Software (HCS) by McTrans. ODOT may allow use of other traffic analysis and modeling programs when HCS is incapable of providing analysis results or when supplemental analysis is desired. Software currently used by ODOT for traffic operations analyses includes Synchro/SimTraffic, SIDRA, and TransModeler. Synchro/SimTraffic may be used for assessing corridor progression. However, Synchro shall not be used for design. While other programs may be considered for unique situations, it is preferred that the above identified programs be used.

Default Values and Guidance

The following section provides information on required input values and default values to be used for traffic analyses and methodologies.

Highway Capacity Software (HCS) – Streets (Signalized Intersection)

Primary Input Data
•Traffic → Storage Length, ft → Enter left/right turn lane and thru lane storage, see figure to right on how values are measured.
•Traffic → RTOR, veh/h → 0 veh/h
•Phasing → Cycle,s → 60-120s, typically
•Phasing → Uncoordinated Intersection → Toggle on
•Phasing → Field-Measured Phase Times → Toggle on
•Timing → Yellow Change, s & Red Clearance, s → Minimum Y+R is 5s. Actual/calculated values may be used.
•Timing → Minimum Green, s → a) 10s for phase that includes a thru movement, b) 7s for a phase that excludes a thru movement. Note, protected lefts must have a minimum contiguous duration of 7s.

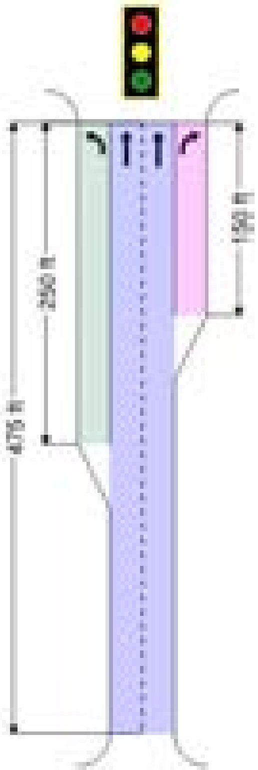
Detailed Input Data
•General → Queue Length Percentile → 95^{th} percentile

Note: A queue storage ratio (QSR) will be calculated using the storage length input and calculated queue length. If QSR exceeds 1.0, HCS will not identify that there will be spillovers into other lanes and cause unforeseen delays because HCS does not currently account for turn lane overflow and queue spillback. If possible, any QSR above 1.0 should be alleviated prior to finalizing the analysis. If not possible, microsimulation (i.e. SimTraffic) may be required.

Highway Capacity Software (HCS) – All Modules
•PHF (peak hour factor) → Use HCS7 default (typically)
Note: A PHF less than the default value may be required to analyze sites where peaked traffic is anticipated (i.e. sports center, school, factory, etc.

Note: Default values shall not be used for readily available information such as acceleration/deceleration length(s), storage length(s), lane width(s), percent heavy vehicles, free-flow speed, etc.

Exhibit for Figure 401-14a TRAFFIC OPERATIONAL ANALYSIS DESIGN SOFTWARE



TRAFFIC OPERATIONAL ANALYSIS DESIGN SOFTWARE (SIDRA)	401-14b REFERENCE SECTION 401.2, 403.3

SIDRA (Limited to roundabout analyses) – The settings below are for SIDRA 8.0

Parameter	Site Input - Tab / Section	Change Setting To
Roundabout Capacity Model	Roundabouts - Options - Roundabout Model Options	*SIDRA Standard*
Site Level of Service Method	Parameter Settings - Options - General Options	*Delay & Degree of Saturation (SIDRA)*
Roundabout LOS Method	Roundabouts - Options - Roundabout Model Options	*Same as Signalized Intersections*
Exclude Geometric Delay	Roundabouts - Options - Roundabout Model Options	*Unchecked*
HCM Delay Formula	Roundabouts - Roundabout Model Options	*Unchecked*
Environmental Factor (Single-Lane)	Roundabouts - Roundabout Data - SIDRA Standard Model Calibration	*1.10 - Opening Day* *1.00 - Design Year*
Environmental Factor (Multi-Lane)	Roundabouts - Roundabout Data - SIDRA Standard Model Calibration	*1.20 - Opening Day* *1.10 - Design Year*
Entry/Circulating Flow Adjustment	Roundabouts - Roundabout Data - SIDRA Standard Model Calibration	*Medium*
Lane Width	Lane Geometry - Lane Configuration - Lane Configuration Data	*14 feet min. (each lane)* *(use design width if known)*
Circulating Width	Roundabouts - Roundabout Data - Geometry	*14 feet min. (each lane)* *(use design width if known)*
Entry Radius	Roundabouts - Roundabout Data - Geometry	*Single Lane - 90 feet Multi-Lane - 150 feet* *(use design width if known)*

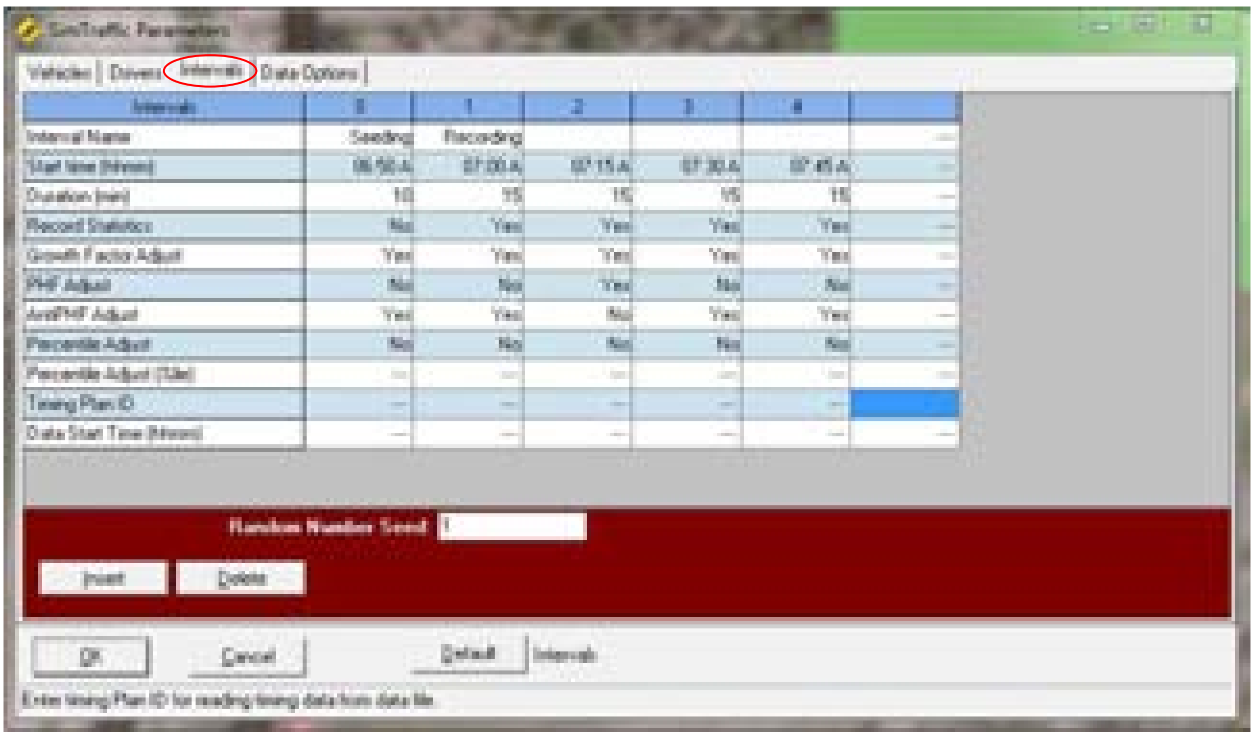
TRAFFIC OPERATIONAL ANALYSIS DESIGN SOFTWARE	401-14cE
	REFERENCE SECTION 401.2

Microsimulation software (SimTraffic)

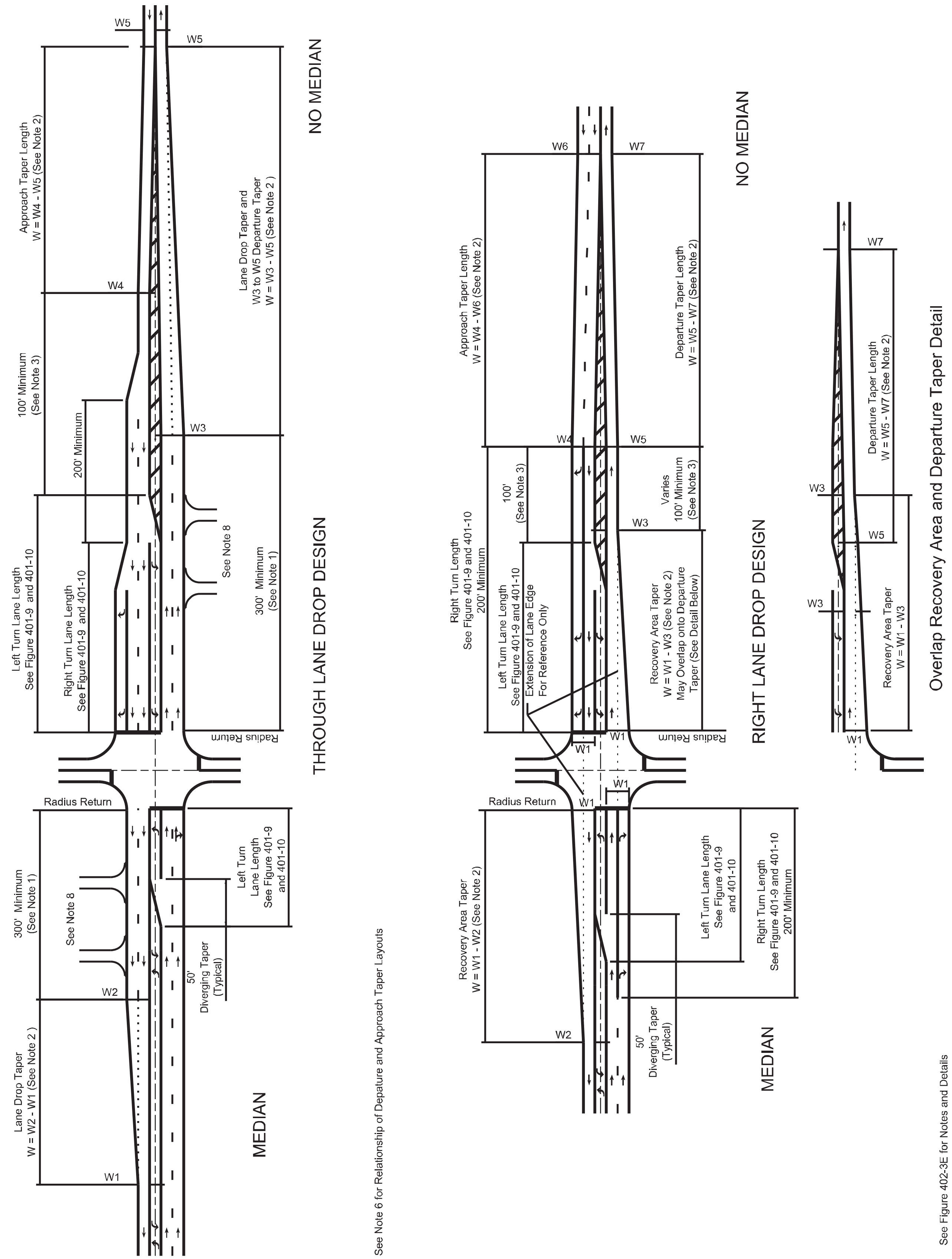
- Analysis should use a minimum of 10 simulation runs with different number seeds.
- Refer to the FHWA Traffic Analysis Tools Program for additional guidance on using simulation software: http://ops.fhwa.dot.gov/trafficanalysistools/index.htm

Synchro (SimTraffic)

Below is a screen capture of the minimum parameters, for the peak period that must be used when running SimTraffic. These are found by selecting “Options” in the menu bar, then “Intervals and Volumes”.



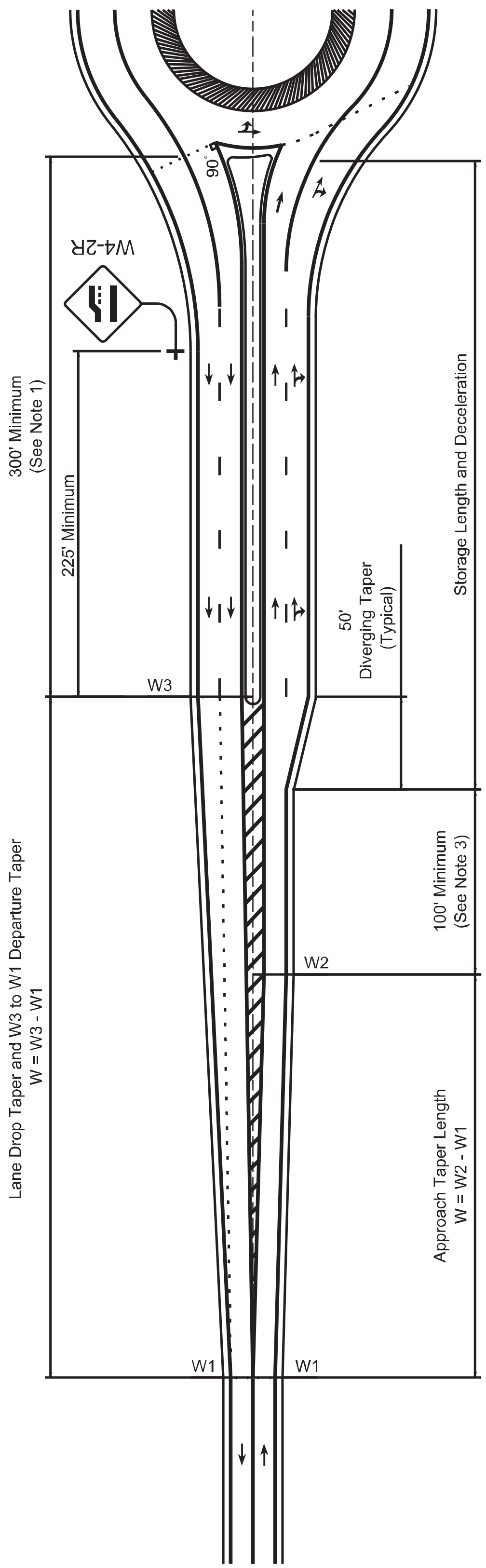
ADD AND DROP LANES AT INTERSECTIONS	402-1
	REFERENCE SEECTION 401.6.5, 401.6.6



See Note 6 for Relationship of Depature and Approach Taper Layouts

See Figure 402-3E for Notes and Details

ADD AND DROP LANES AT MULTILANE ROUNDABOUTS	402-2
	REFERENCE SEECTION 403.4



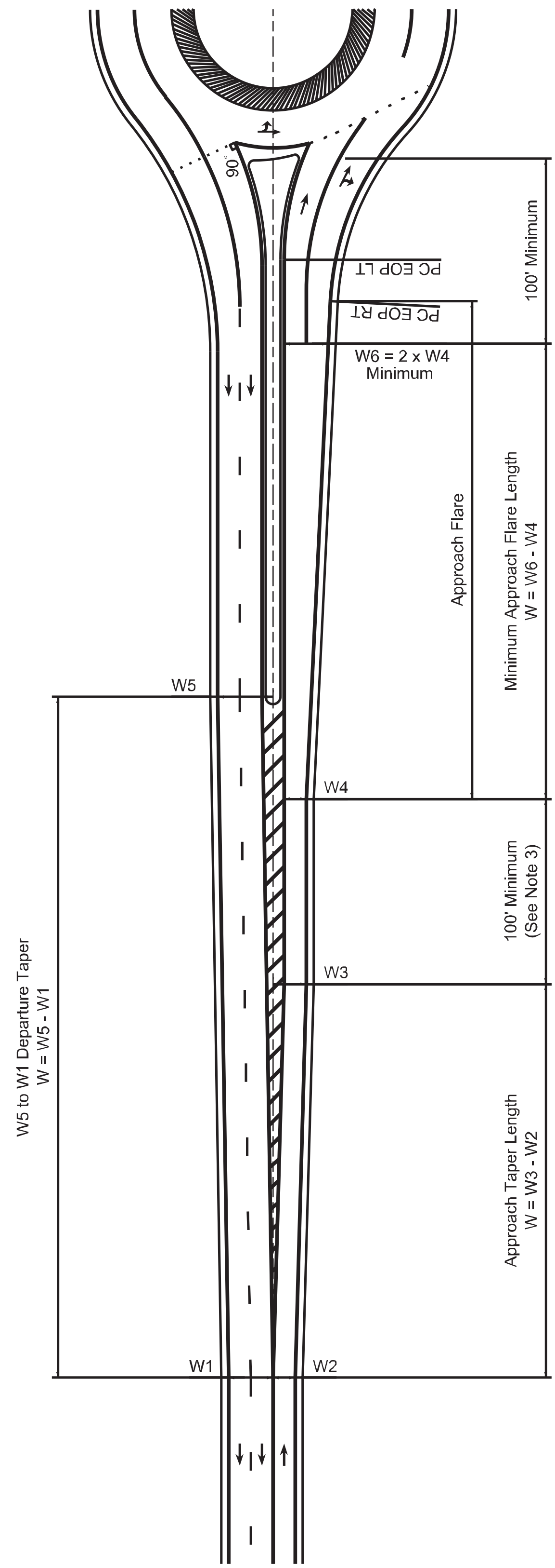
ROUNDABOUT TAPER OPTION

See Figure 402-3 for Notes:

See Note 2 for Calculation of Taper Lengths.

See Note 6 for Relationship of Depature and Approach Taper Layouts

See Figures 401-9 and 401-10 for Calculation of Storage Lengths.



ROUNDABOUT FLARE OPTION

This Option is acceptable for use on Non-State Routes.

ADD AND DROP LANE NOTES AND DETAILS	402-3
	REFERENCE SECTION 401.6.5, 403.4

Notes for Figures 402-1 and 402-2 and Other Details

1. This distance should be at least long enough to allow proper advance placement of warning signs for a typical lane reduction, based on OMUTCD guidelines. The lane is then merged at a taper rate as shown in Note 2 below.

2. The taper distance is calculated from:

   Design Speed of 50 mph or more:
   $L=WS$

   Design Speed of less than 50 mph;
   $L=WS^2/60$

   Where: L=Taper length in feet
   W=Offset width in feet
   S=Design speed

3. May be reduced or eliminated in urban areas if intersection spacing or storage is constraining.

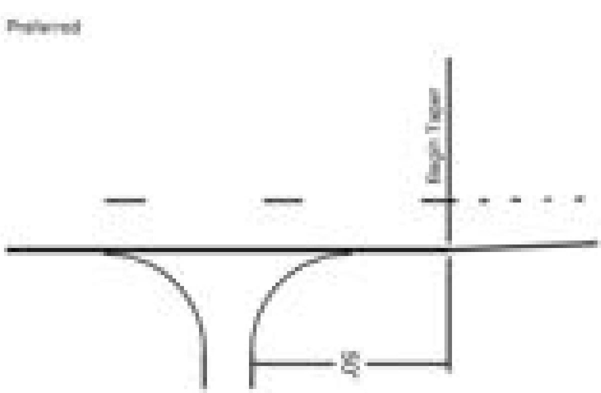
4. Lane addition and removal on 3-lane or more highways is similar. Dropping of additional lanes shall be based on OMUTCD guidelines or at next intersection as shown on Figure 402-1.

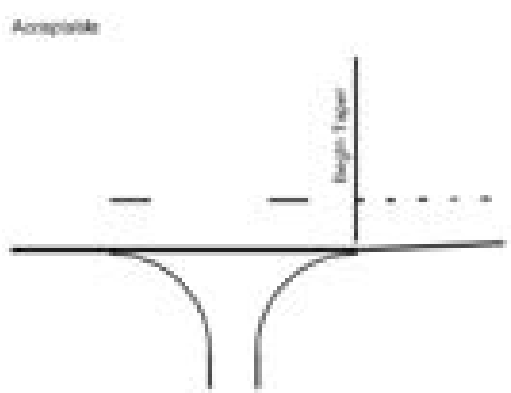
5. Figures are not intended to show pavement striping. Striping is based upon the OMUTCD.

6. For the relationship of departure and approach taper layouts see Figure 401-7.

7. In general, it is desirable to place all tapers on tangents or on curves with normal crown superelevation.

8. If driveways are present, distance to beginning of lane drop taper may need to be increased. See details below for drive and taper configurations.



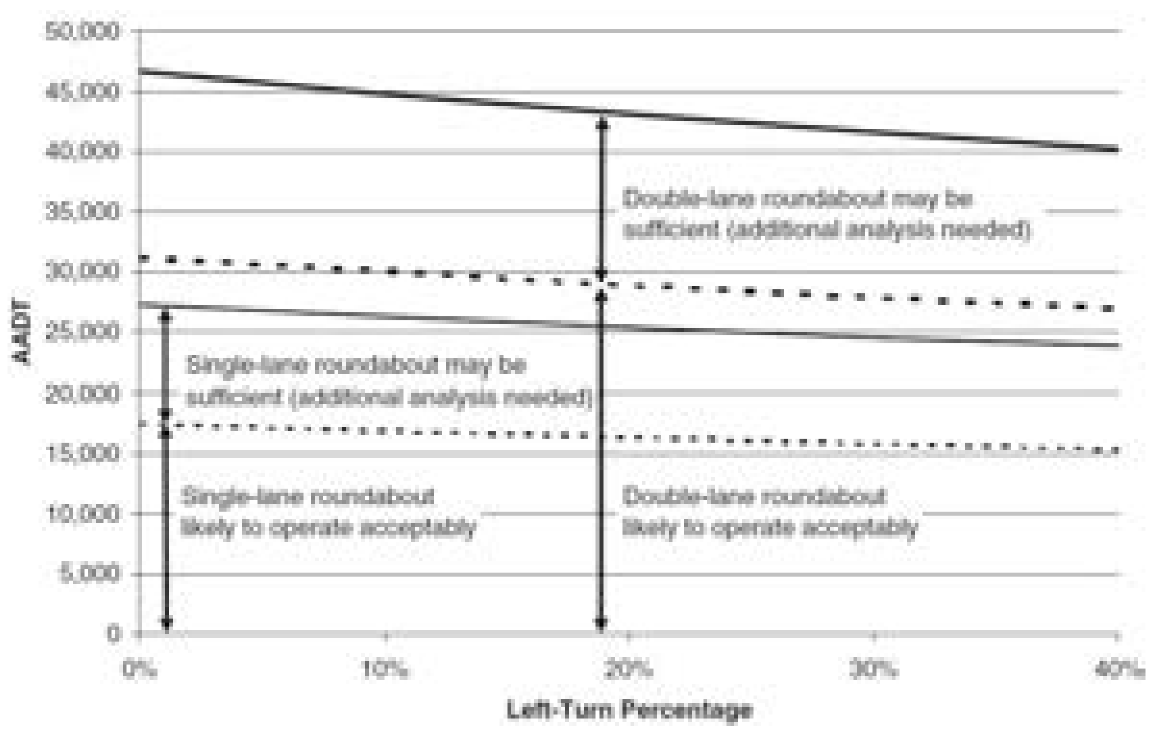


# Roundabout Sizing Thresholds	**403-1**
	REFERENCE SECTION 403.3

NCHRP Report 672 – Exhibit 3-14
Volume Thresholds for Determining the Number of Entry Lanes Required (Planning Level)

Volume Range Entry + Circulating (veh/hr)	Number of Lanes Required
0 – 1,000	• Single-lane entry likely to be sufficient
1,000 – 1,300	• Two lane entry may be needed • Single-lane may be sufficient based upon more detailed analysis
1,300 – 1,800	• Two lane entry is likely to be sufficient
1,800+	• More than two entry lanes may be required • A more detailed capacity evaluation should be conducted to verify lane numbers and arrangements

NCHRP Report 672 – Exhibit 3-12
Planning-Level Daily Intersection Volumes



Roundabout Critical Design Parameters

403-2
REFERENCE SECTION 403.7

Roundabout Critical Design Parameters

Project - County Route Section

PID

Design Parameters	**Leg 1**	**Leg 2**	**Leg 3**	**Leg 4**	**Leg 5**
Inscribed Circle Diameter, FT					
Entry Width, FT					
Entry Angle PHI ϕ, DEG					
Exit Width, FT					
Circulatory Roadway Width Upstream of Entry, FT					

Fastest Path Speed	**Leg 1**	**Leg 2**	**Leg 3**	**Leg 4**	**Leg 5**
R_1, Radius/Speed, FT/MPH					
R_2, Radius/Speed, FT/MPH					
R_3, Radius/Speed, FT/MPH					
R_4, Radius/Speed, FT/MPH					
R_5, Radius/Speed, FT/MPH					
R_5, Bypass Radius/Speed, FT/MPH					

Minimum Sight Parameters	**Leg 1**	**Leg 2**	**Leg 3**	**Leg 4**	**Leg 5**
Approach Design Speed, MPH					
Approach Stopping Sight Distance, FT/MPH					
Circulatory Stopping Sight Distance, FT/MPH					
Exit (Crosswalk) Stopping Sight Distance, FT/MPH					
Intersection Sight Distance, FT/MPH					

General	
Design Vehicle(s)	
Truck Apron Width, FT	

Designer:

Signature:

Date:

Link to editable Excel File

January 2019

500 Interchange Design

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501 INTERCHANGE DESIGN

501.1 General

An interchange is defined as 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 or highways on different levels. Interchanges are utilized on freeways and expressways where access control is important. They are used on other types of facilities only where crossing and turning traffic cannot be accommodated by a normal at-grade intersection. An Interchange Justification Study may be necessary.

501.2 Interchange Type

The most commonly used types of interchanges are the diamond, cloverleaf and directional.

501.2.1 Diamond Interchanges

The diamond interchange is the most common type where a major facility intersects a minor highway. The design allows free-flow operation on the major highway but creates at-grade intersections on the minor highway with the ramps. Traffic control at the at-grade intersections can be stop, signal or roundabout. The capacity of a diamond interchange is limited by the at-grade intersections on the minor highway. Variations of the diamond interchange include but are not limited to the Tight Urban Diamond Interchange (TUDI), the Single Point Urban Interchange (SPUI) and the Diverging Diamond Interchange (DDI).

501.2.1.1 Tight Urban Diamond Interchange (TUDI)

The TUDI, a type of compressed diamond interchange, is used in urban and suburban areas where right of way is limited. This interchange design has two closely spaced signalized ramp intersections. Special signal phasing allows queuing of vehicles outside the ramp intersections and eliminates queuing of vehicles between the ramp intersections. Typical designs provide 250 to 400 feet of separation between the signalized intersections.

The key operational aspect of a TUDI is one controller running both intersections implementing the Texas Diamond 4 Phase operation. The single controller that runs both ramps can be coordinated with adjacent intersections, but the two ramp signals are on one controller. The 4 phase sequence is used when queue storage between the ramp intersections is critical as it tends to not store any vehicles between the two ramp intersections.

A TUDI may not need to implement Texas Diamond 4-Phase operation. Geometric constraints may warrant the construction of a TUDI, but operational benefits of the Texas Diamond Phasing may not be needed and conventional signal phasing can be used.

501.2.1.2 Single Point Urban Interchange (SPUI)

The SPUI, is another variation of the compressed diamond interchange. It was developed to streamline operations by using a single traffic signal to control traffic movements within the interchange area. The SPUI also requires less right-of-way than a conventional diamond interchange.

I-270 & Sawmill Road SPUI (Google)

A SPUI should be considered:

- In areas with limited right-of-way.
- At locations with heavy left turn volumes both on and off the ramps.
- At locations where a wider structure or intersection can be accommodated.

SPUIs offer the following benefits:

- Construction in a relatively narrow right-of-way, resulting in potentially significant cost reductions.
- Since the SPUI has only one intersection, as opposed to two for a diamond interchange, the operation of the single traffic signal on the crossroad may result in reduced delay through the intersection area when compared to a diamond interchange.
- Improved safety since vehicles only cross paths at one intersection instead of two.
- Right turn movements both on and off the ramps are typically free flow or yield control.
- Only the left turns pass through the signalized intersection. As a result, a major source of traffic conflict is eliminated, increasing overall intersection efficiency and reducing the traffic signal phasing needed from four-phase to three-phase operation.
- The turning angle and curve radii for left-turn movements through the intersection are significantly flatter than at conventional intersections and, therefore, the left turns move at higher speeds. The left turn angle is typically 45 to 60 degrees with a minimum radius of 150 to 200 ft. The above-mentioned operations may result in a higher capacity than a conventional tight diamond interchange.

Limitations/disadvantages of a SPUI include:

- Construction costs of SPUIs tend to be much higher than conventional diamond interchanges due to costs associated with the bridges. Overpass SPUIs need long bridges to span the large intersection below. A two-span structure is not a design option because a center column would conflict with traffic movements. Single-span overpass bridges are typically 220 ft in length, while three-span bridges often exceed 400 ft. The SPUI underpass tends to be wide and often is "butterfly" in shape. Rectangular SPUI structures, while resulting in unused deck area, may provide additional area for maintenance of traffic and simplified construction. Where right-of-way is constrained, SPUIs typically utilize extensive retaining walls, further adding to the cost. However, the higher construction cost of SPUIs is often offset by the reduced right-of-way cost.
- A potential disadvantage of SPUIs is the length and geometry of the path for left-turning vehicles through the intersection. Like most typical intersections, left-turning vehicles pass to the left of opposing left-turning vehicles. However, due to the size and distance between opposing approaches, the path of left-turning vehicles does not resemble a quarter of a circle found at typical intersections, but rather resembles a quarter of an ellipse. To provide positive guidance for this non-traditional path, at a minimum, dashed lane lines should be painted through the intersection.
- A skew angle between the two roadway alignments has an adverse effect on SPUIs because it increases clearance distances and adversely affects sight distance. Severe skew in alignments may also increase the length of the bridge and widen the distance between the stop bars on the local streets. Extreme care should be exercised in planning SPUIs when the skew angle approaches 30 degrees. It is important to provide visibility between exit ramp traffic and cross street traffic approaching from the left. For left-turn movements from the mainline's ramp to the cross street, provide a clear cornering sight line with no obstructions from bridge abutments, pilasters, signal/light poles, signing, or landscaping.

Design considerations of a SPUI include:

- It is desirable that the left-turn curve be a single radius. This will, however, typically result in additional right-of-way, a larger bridge structure, or both. Where it is not practical to provide a single radius and curves are compounded from a larger to a smaller radius, the second curve should be at least half the radius of the first.
- Stopping sight distance should be provided on the left-turn movements equal to or exceeding the design speed for the curve radius involved.
- Additional median width on the cross street can improve intersection operation. The stop bar location on the cross street is dependent on the wheel tracks from the opposing ramp left-turn movement. By widening the median, the stop bar on the cross street can be moved forward, thus reducing the size of the intersection and the distance each vehicle travels through the intersection. The results include greater available green time and less potential driver confusion due to an expansive intersection area.
- A minimum clear distance of 10 ft between opposing left turns within the intersection should be provided.
- Pedestrian crossing of the local street at ramp terminals typically adds a signal phase and uses considerable green time, resulting in reduced operational efficiency. Therefore, the overall design should include provision for pedestrian crossings at adjacent intersections instead of at the ramp terminal intersection.
- Right-turn lanes at SPUIs are typically separated from the left-turn lanes, often by a considerable distance. The exit ramp right turn can be a free or controlled movement. The design of free right turns should include an additional lane on the cross street beginning at the free right-turn lane for at least 300 ft before being merged. Free-flow right turns from the exit ramp to the crossroad may or may not be desirable depending on the traffic operational analysis which includes the

downstream intersection(s) of the crossroad. There may be inadequate weaving distance between the exit ramp and the adjacent intersection. Where the right-turn movement is controlled by a stop sign or traffic signal, adequate right-turn storage on the exit ramp should be provided to prevent blockage of vehicles turning left or traveling straight. Free-flow right turns on entrance ramps pose little operational concern, assuming adequate merge length is provided on the entrance ramp. The right-turn lane should extend at least 300 ft beyond the convergence point before beginning the merge.

501.2.1.3 Diamond Interchange with Roundabouts

Roundabout interchanges utilize roundabouts at either one or both ramp intersections. Additional bridge width is usually not necessary at roundabout interchanges due to the elimination of turn lanes. The benefits and costs associated with this type of interchange also follow those for a single roundabout.

Roundabouts may be a good alternative if the interchange has a high proportion of left turn flows from the off ramps and to the on ramps during peak periods, combined with limited queue storage space on the bridge crossing, off-ramps, or arterial approaches. In such circumstances, roundabouts operating within their capacity are particularly useful in solving these problems when compared with other forms of intersection control.

The raindrop roundabout interchange design exhibits very little queuing between the intersections since these movements are almost unopposed. Therefore, the approach lanes across the bridge can be minimized. On the other hand, drivers do not have to yield when approaching from the connecting roadway between the two roundabouts. If the roundabout is designed poorly, drivers may be traveling faster than they should to negotiate the next roundabout safely. The designer should analyze relative speeds to evaluate this alternative.

A potential benefit of roundabout interchanges is that the queue length on the off-ramps may be less than at a signalized intersection.

501.2.1.4 Diverging Diamond Interchange (DDI)

The Diverging Diamond Interchange (DDI) is a variation of a conventional diamond interchange. The DDI uses directional crossover intersections to shift traffic on the cross street to the left-hand side between the ramp terminals within the interchange. Crossing the through movements to the opposite side replaces left-turn conflicts with same-direction merge/diverge movements and eliminates the need for exclusive left-turn signal phases to and from the ramp terminals. All connections from the ramps to and from the cross street are joined outside of the cross-over intersections, and these connections can be controlled by two-phase signals, have stop or yield control, or be free flowing.

A DDI should be considered:

- At locations where there is limited roadway width for left-turns between ramp intersections and limited right-of-way to expand.
- At locations with heavy left turn volumes both on and off the ramps.
- At locations without adjacent traffic signals or nearby driveways.

The DDI offers the following benefits:

- Improved safety due to the reduction in the number of conflict points (locations where vehicle paths cross), vehicle-to-vehicle, vehicle-to-pedestrian and vehicle-to-bike.

- By allowing the ramp-terminal intersections to operate with simple, two-phase signal operations, the design provides flexibility to accommodate a greater volume of traffic and operate with less delay.
- Overall operations of a DDI may be greater compared to a conventional signalized diamond interchange due to shorter cycle lengths, reduced time lost per cycle phase, reduced stops and delay, and shorter queue lengths.
- The DDI also reduces the number and severity of conflict points for both motorized and nonmotorized users. The crossing distances for pedestrians are comparatively shorter, and usually involve traffic approaching from only one direction at a time. The cross-sectional characteristics of a DDI provide multiple options for facilitating convenient pedestrian and bicycle movements, and the geometry of the crossover intersections have an added benefit of reducing motorized vehicle speeds through the interchange, resulting in a traffic calming effect which may reduce crashes.
- Retrofitting an existing conventional diamond interchange to a DDI may be less costly than options involving widening the crossroad near the interchange (including widening the bridge) and adding additional lanes to the ramps.
- For new interchanges, the operational efficiency of a DDI may allow for a smaller structural footprint since fewer lanes are generally needed to accommodate the traffic demands. In some contexts, the DDI may allow for reduced right-of-way needs and construction costs compared to other interchange forms.
- The large channelizing islands help reduce wrong-way movements onto the exit ramps.
- Accommodates either overpass or underpass designs.

I-270 & Roberts Road DDI (District 6)

Limitations/Disadvantages of a DDI include:

- Crossing traffic to the “wrong side” of the road may not meet driver expectations.
- A DDI does not allow free flowing traffic on the crossroad in both directions since the signals cannot be green at both intersections for both directions at the same time.
- The potential operational benefits of the DDI ramp intersections may be overshadowed by poor operational performance of nearby signals on the crossroad. If an adjacent signal is too close and the queue storage length is inadequate, the traffic spillback may inhibit the movement of traffic along the crossroad, and potentially block traffic from the exit ramps.
- The DDI design does not accommodate typical “up and over” exit to entrance movements for oversized vehicles or authorized vehicles during maintenance or emergency situations.
- Crossroads that are heavily skewed to the main facility typically need greater intersection spacing.

Design considerations for a DDI include:

- It may be advantageous to use multiple structures at the grade separation, especially where the skew angle between facilities is significant.
- The spacing between ramp intersections will impact signal design and operations on the crossroad corridor but very tight spacing between ramp intersections may constrain the design of the crossovers and limit queue storage and signal timing options.
- The proximity to a DDI of adjacent signalized intersections along the crossroad may impact the performance of the DDI at a given location. Modifications to adjacent signalized intersections along the crossroad may be necessary to maintain the overall signal progression along the corridor and reduce potential effects of queue spillback.
- The through movement queues need to be checked at the crossovers for blockage of the free-flowing left and right turns to and from the ramps. Auxiliary lanes may be needed for the turn lanes if blockage occurs.
- Design speeds for crossover alignments should be in the range of 20 to 35 mph, resulting in crossover radii in the range of 100 to 500 ft.
- Cross slopes are typically in the range of plus or minus 2 percent.
- Higher approach speeds need to be lowered to the crossover design speeds with advance warning signs and geometric features (reverse curves, etc.).
- 100 ft of tangent between the reverse curves through the crossover should be provided in both directions. The tangent helps to maintain the desired vehicle tracking and the curve-tangent-curve sequence promotes driving at the desired target speed.
- Radii at the exit and entrance ramp movements are like other interchanges and include allowance for the turning path of the design vehicle, sight distance, pedestrians, bicycles and the intersection traffic control type.
- The crossing angle is the acute angle between lanes of opposing traffic within the crossover based on the tangent sections. The greater the crossover angle, the more the crossover will appear like a “normal” intersection of two different cross routes and decrease the likelihood of a driver making a wrong-way movement. However, greater crossing angles generally result in larger footprints. Larger crossing angles in combination with sharp reverse curves can increase the potential for overturning of vehicles with high centers of gravity and excessive driver discomfort through the crossovers. The recommended approach is to attain the largest crossing angle possible that is in balance with the other geometric parameters and site constraints. The crossover angle of a DDI is generally between 40 to 50 degrees. Crossover angles less than 30 degrees may increase the potential for wrong-way movements.

- Lane widths along the crossroad of a DDI typically range from 12 to 15 ft depending on site location and consideration for design vehicles traveling side by side through the crossover area. The wider lane width typically occurs prior to and after the crossover curves. The additional lane width does not need to be continued between the two crossovers.
- The shoulders required by the crossroad should also be provided through the DDI. The outside shoulder becomes the inside shoulder within the interchange between the crossover intersections.
- Sight distance at DDIs is important for both vehicles maneuvering through the crossovers or turning left and right from the ramp terminals onto the cross street, especially when the turning ramp terminal traffic is under yield control.

The FHWA *Diverging Diamond Interchange Informational Guide* provides additional information on diverging diamond interchanges.

501.2.2 Cloverleaf Interchange

Cloverleaf designs may be used in lieu of a diamond when a continuous flow design is required where two major facilities intersect. In this case, a full cloverleaf interchange is the minimum design that can be used. The designer should consider collector-distributor roads in conjunction with cloverleaf interchanges to minimize weaving.

Full Cloverleaf Interchange – I-270/East Main Street (Google)

However, full cloverleafs have deficiencies which need to be addressed before being chosen as the interchange type. Principle disadvantages are:

- The inherent weaving maneuver generated and the short weaving length available.

- Large trucks may not be able to operate efficiently on the smaller curve radii on the associated loop ramps.
- Loop ramps are limited in capacity.

When Collector-Distributor roads are not used, a further disadvantage includes weaving on the main line, the double exit on the main line and problems associated with signing for the second exit.

The full cloverleaf weaving maneuver is not objectionable when the left-turning movements are relatively light, but when the sum of traffic volumes on two adjoining loops approaches about 1,000 vehicles per hour, interference occurs, which results in a reduction in the speed of the mainline traffic. When the weaving volume in a particular section exceeds 1,000 vehicles per hour, the quality of service on the main facility deteriorates, generating a need to transfer the weaving section from the through lanes to a C-D road. For these reasons, full cloverleafs are discouraged.

501.2.2.1 Partial Cloverleaf Interchange

Partial cloverleaf designs may be used in lieu of a diamond when development or other physical conditions prohibit construction in a quadrant, or where heavy left turns are involved. In the design of partial cloverleafs, the site conditions may offer a choice of quadrants to use. However, at a particular interchange site, topography and culture may be the factors that determine the quadrants in which the ramps and loops can be developed. There is considerable operational advantage in certain arrangements of ramps. These are discussed and summarized below.

Ramps should be arranged so that the entrance and exit turns create the least impediment to the traffic flow on the major highway. The following guidelines should be considered in the arrangement of the ramps at partial cloverleafs:

- The ramp arrangement should enable major turning movements to be made by right-turn exits and entrances.

- Where through-traffic volume on the major highway is decidedly greater than that on the intersecting minor road, preference should be for an arrangement that places the right turns (either exit or entrance) on the major highway, even though this results in a direct left turn off the crossroad.

These controls do not always lead to the most direct turning movements. Instead, drivers frequently may need to first turn away from or drive beyond the road that is their intended destination. Such arrangements cannot be avoided if the through-traffic movements, for which the separation is provided, are to be facilitated to the extent practical.

Partial Cloverleaf A Interchange (4 Quadrant ParClo A) I-270/Georgesville Road (Google)

The ParClo A layout has loop ramps that are in diagonally opposite quadrants and the ramps are on the near side of the structure as drivers approach on the major road. Preferred for a high volume of left-turns from the crossroad to the mainline. Weaving between the loop ramps is eliminated.

Partial Cloverleaf B Interchange (2 Quadrant ParClo B) I-71/SR48 (Google)

The ParClo B layout has loop ramps that are in diagonally opposite quadrants and the ramps are on the far side of the structure as drivers approach on the major road. Preferred for a high volume of left-turns from the mainline to the crossroad. Weaving between the loop ramps is eliminated.

For additional information and types of Partial Cloverleaf Interchanges, see the AASHTO Green Book, Chapter 10.

501.2.3 Directional Interchanges

Directional interchanges are the highest type and most expensive. They permit vehicles to move from one major freeway to another major freeway at relatively fast and safe speeds.

502 INTERCHANGE DESIGN CONSIDERATIONS

502.1 Determination of Interchange Configuration

Interchange configurations are covered in two categories, “system interchanges” and “service interchanges.” The term “system interchange” is used to identify interchanges that connect two or more freeways, whereas the term “service interchange” applies to interchanges that connect a freeway to lesser facilities. Generally, interchanges in rural areas are widely spaced and can be designed on an individual basis without any appreciable effect from other interchanges within the system. However, the final configuration of an interchange may be determined by the need for route continuity, uniformity of exit patterns, single exits in advance of the separation structure, and elimination of weaving on the main facility, signing potential, and availability of right-of-way. Selecting an appropriate interchange configuration in an urban environment involves considerable analysis of prevailing conditions so that the most practical interchange configuration alternatives can be developed. Generally, in urban areas, interchanges are so closely spaced that each interchange may be influenced directly by the preceding or following interchange to the extent that additional traffic lanes may be needed to satisfy capacity, weaving and lane balance. Interchanges should provide for all movements, even when an anticipated turning movement volume is low.

Once several alternates have been prepared for the system design, they can be compared on the following principles: (1) capacity, (2) route continuity, (3) uniformity of exit patterns, (4) single exits in advance of the separation structure, (5) with or without weaving, (6) potential for signing, (7) cost, (8) availability of right-of-way, (9) constructability, and (10) compatibility with the environment.

502.2 Approaches to the Structure

502.2.1 Alignment, Profile and Cross Section

Traffic passing through an interchange should be afforded the same degree of utility and safety as that given on the approaching roadways. The design speed, alignment, profile and cross section in the interchange area should be consistent with those on the approaching highways. Four-lane roadways should be divided at interchanges with a non-traversable median to ensure that drivers use the proper ramps for left-turning maneuvers. At-grade left turns preferably should be accommodated within a suitably wide median.

502.2.2 Sight Distance

Sight distance on the roadways through an interchange should be at a minimum the required stopping sight distance and preferably should be Decision Sight Distance (***Figure 201-6***), particularly along entrances and exits.

The horizontal sight distance limitations of piers and abutments at curves usually present a more difficult problem than that of vertical limitations. With the minimum radius for a given design speed, the normal lateral clearances at piers and abutments of underpasses does not provide the minimum stopping sight distance. Similarly, on overpasses with the sharpest curvature for the design speed, sight distance deficiencies result from the usual offset to the bridge railing. Above minimum radii should be used for curvature on roadways through interchanges. If sufficiently flat curvature cannot be used, the clearances to abutments, piers or bridge railing should be increased to obtain the proper sight distance, even though this involves increasing structure spans or widths.

502.3 Interchange Spacing

Interchanges should be located close enough together to properly discharge and receive traffic from other highways or streets, and far enough apart to permit the free flow and safety of traffic on the main facility. In general, more frequent interchange spacing is permitted in urbanized areas. Minimum spacing is determined by weaving requirements, ability to sign, lengths of speed change lanes, and capacity of the main facility.

Interchanges within urban areas should not be spaced closer than an average of 2 miles, in suburban sections an average of not closer than 4 miles, and in rural sections an average of not closer than 8 miles. In consideration of the varying nature of the highway, street or road systems with which the freeway or expressway must connect, the spacing between individual adjacent interchanges must vary considerably. In urban areas, the minimum distance between adjacent interchanges should not be less than 1 mile, and in rural areas not less than 3 miles. Spacing less than this have a detrimental effect on freeway operations.

502.4 Uniformity of Interchange Patterns

Since interchange uniformity and route continuity are interrelated concepts, interchanges along a freeway should be reasonably uniform in geometric layout and general appearance to provide the appropriate level of service and maximum safety in conjunction with freeway operations. Except in highly special cases, all entrance and exit ramps should be on the right.

502.5 Route Continuity

Route continuity is an extension of the principle of operational uniformity coupled with the application of proper lane balance and the principle of maintaining a basic number of lanes. The principle of route continuity simplifies the driving task in that it reduces lane changes, simplifies signing, delineates the through route and reduces the driver’s search for directional signing. Desirably, the through driver should be provided a continuous through route on which changing lanes is not necessary to continue on the through route. In maintaining route continuity, interchange configuration may not always favor the heavy traffic movement, but rather the through route. In this situation, heavy movements can be designed on flat curves with reasonably direct connections and auxiliary lanes.

502.6 Signing and Marking

The safety, efficiency and clarity of paths to be followed at interchanges depend largely on their relative spacing, geometric layout and effective signing and marking. The location of and minimum spacing between ramp terminals depends to a large degree on whether or not effective signing can be provided. Signing and marking should conform to the OMUTCD.

502.7 Basic Number of Lanes

The basic number of lanes is defined as a minimum number of lanes designated and maintained over a significant length of a route, irrespective of changes in traffic volume and lane balance needs. (The basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes, based on capacity needs of the section.)

502.8 Coordination of Lane Balance and Basic Number of Lanes

Design traffic volumes and a capacity analysis determine the basic number of lanes to be used on the freeway and the minimum number of lanes on the ramps. The basic number of lanes should be established for a substantial length of freeway and should not be changed through pairs of interchanges, simply because there are substantial volumes of traffic entering or leaving the freeway. There should be continuity in the basic number of lanes. Auxiliary lanes should be provided for variations in traffic demand.

After the basic number of lanes is determined for each roadway, the balance in the number of lanes should be checked on the basis of the following principles:

1. At entrances, the number of lanes beyond the merging of two traffic streams should not be less than the sum of all traffic lanes on the merging roadways minus one, but may be equal to the sum of all traffic lanes on the merging roadways.

2. At exits, the number of approach lanes on the roadway should be equal to the number of lanes on the roadway beyond the exit, plus the number of lanes on the exit, minus one. Exceptions to this principle occur at cloverleaf loop ramp exits that follow a loop ramp entrance and at exits between closely spaced interchanges. In these cases, the auxiliary lane may be dropped in a single-lane exit with the number of lanes on the approach roadway being equal to the number of through lanes beyond the exit plus the lane on the exit.

3. The traveled way of the highway should be reduced by not more than one traffic lane at a time.

502.9 Auxiliary Lanes

An auxiliary lane is the portion of the roadway adjoining the traveled way for speed change, turning, storage for turning, weaving and other purposes supplementary to through-traffic movement. An auxiliary lane may be provided to comply with the concept of lane balance, to comply with capacity needs or to accommodate speed changes, weaving and maneuvering of entering or exiting traffic.

502.10 Lane Reductions

The basic number of mainline lanes should not be reduced through a “service interchange”. If a reduction in the basic number of lanes is warranted by a substantial decrease in traffic volume over a significant length of freeway, then it should be reduced between interchanges. The reduction should occur 2,000 to 3,000 ft. from the end of the acceleration taper of the previous interchange to allow for adequate signing. The end of the lane reduction should be tapered at a rate of 70:1. The lane reduction should occur on a tangent section of freeway, preferably within a sag vertical curve, and provide Decision Sight Distance, ***Figure 201-6***, where possible. The lane reduction should also be on the right side of the freeway.

502.11 Weaving Sections

Weaving sections are highway segments where the pattern of traffic entering and exiting at contiguous points of access results in vehicle paths crossing each other. Weaving sections may occur within an interchange, between closely spaced interchanges or on segments of overlapping routes.

Because weaving sections cause considerable turbulence which results in a reduction in capacity, interchange designs that eliminate weaving or remove it from the mainline by the use of C-D roads are desirable.

The capacity of weaving sections may be seriously restricted unless the weaving section has adequate length, adequate width and lane balance. Refer to the Highway Capacity Manual for capacity analysis of weaving sections.

503 INTERCHANGE RAMP DESIGN

503.1 General

An interchange ramp is a roadway which connects two legs of an interchange. Ramp cross section elements are discussed in ***Section 303.1***. Elements contributing to horizontal and vertical alignments are designed similar to any roadway (***Section 200***) once the ramp design speed has been determined.

503.2 Ramp Design Speed

In order to design horizontal and vertical alignment features, a design speed must be determined for each ramp. Since the driver expects a speed adjustment on a ramp, the design speed may vary within the ramp limits. ***Figure 503-1*** includes three ranges of ramp design speeds which vary with the design speed of the mainline roadway. The ramp design speed range is determined by engineering judgment based on several conditions:

1. The type of roadways at each end of the ramp and their design speeds,
2. The length of the ramp,

3. The terminal conditions at each end, and

4. The type of ramp (diamond, loop or directional).

Design exceptions will be required for speed related design criteria that do not meet the following:

- For directional ramps (roadways) that do not provide the minimum design speed given in ***Section 503.2.3.***
- For loop ramps on high-speed roadways that do not provide a minimum design speed of 25 mph (150-ft radius).
- For all other ramps that, at a minimum, do not provide the lower range design speed of ***Figure 503-1.***

503.2.1 Diamond Ramp Design Speeds

Diamond ramps normally have a high speed condition at one end and an at-grade intersection with either a stop or slow turn (15 mph) condition at the other. Upper to middle range design speeds in ***Figure 503-1*** are normal near the high speed facility with middle to lower range design speeds usually used closer to the at-grade intersection.

503.2.2 Loop Ramp Design Speeds

Loop ramps may have a high speed condition at one end and, either a slow or high speed condition at the other. Loop ramps, because of their short radius, usually have design speeds in the lower range in the middle and slow speed end of the ramp with middle range design speeds occasionally used nearer the high speed terminal. For design speeds, see ***Figure 503-1***. The minimum loop ramp radius is 150 feet (50 m).

503.2.3 Directional Ramp Design Speeds

Directional ramps (roadways) generally have high speed conditions at both ends. They are normally designed using a design speed falling into the upper range of ***Figure 503-1***. The absolute minimum should be the middle range design speeds.

503.3 Vertical Alignment

Maximum grades for vertical alignment cannot be as definitely expressed as for the highway, but should preferably not exceed 5 percent. General values of limiting upgrades are shown in **Table 503-1**, but for any one ramp the grades to be used are dependent upon a number of factors. These factors include the following:

1. The flatter the gradient on the ramp relative to the freeway grade, the longer the ramp will be.

2. The steepest grades should occur over the center part of the ramp. Grades at the terminal ends of the ramp should be as flat as possible.

3. Short upgrades of 7 to 8 percent permit good operation without unduly slowing down passenger cars. Short upgrades of as much as 5 percent do not unduly affect trucks and buses.

4. Ramp grades and lengths can be significantly impacted by the angle of intersection between the two highways when the angle is 70 degrees or less. The direction and grade on the two highways may also have a significant impact.

5. Adequate sight distance is more important than a specific gradient control and should be favored in design.

Table 503-1

Maximum Ramp Upgrades

Ramp Design Speed	25-30 mph	35-40 mph	45 mph and above
Desirable Grade (%)	5	4	3
Maximum Grade (%)	7	6	5

Note: Downgrades may exceed the table values by 2%, but should not exceed 8%.

503.4 Horizontal Alignment

Horizontal alignment will be largely determined by the selected design speed and type of ramp. The horizontal alignment criteria found in ***Section 202*** shall also apply to ramps. Check that the required horizontal stopping sight distance is provided. Use the allowed skew at the ramp terminal at-grade intersection to minimize curvature.

Depending on the design speed and curvature, curve widening may be required on a two-lane ramp. See ***Section 301.1.3*** and ***Figure 301-5c***. The WB-62 [WB-19] design vehicle should be used for Interstate ramps.

503.5 Ramp Terminals

The terminal of a ramp is that portion adjacent to the through traveled way, including speed change lanes, tapers and islands. Ramp terminals, as opposed to diverging roadways, require speed change lanes. Ramp terminals may be the at-grade type, as at the crossroad terminal of diamond or partial cloverleaf interchanges, or the free-flow type where ramp traffic merges with or diverges from high-speed through traffic at flat angles. Terminals are further classified as either single-lane or multi-lane and as either a taper or parallel type.

503.5.1 General Considerations

While interchanges are custom designed to fit specific site conditions, it is desirable that the overall pattern of exits along the freeway have some degree of uniformity. It is desirable that all interchanges have one point of exit located in advance of the crossroad wherever practical.

Because considerable turbulence occurs throughout weaving sections, interchange designs that eliminate weaving entirely or at least remove it from the mainline are desirable. Weaving sections may be eliminated from the mainline by the incorporation of C-D roadways or grade separating the ramps (braiding).

Interchanges that provide all exit movements before any entrance movements will also eliminate weaving and are highly recommended.

503.5.2 Left-hand Entrances and Exits

Left-hand entrances and exits are contrary to the concept of driver expectancy when intermixed with right-hand entrances and exits. Therefore, extreme care should be exercised to avoid left-hand entrances and exits in the design of interchanges. Because they are contrary to driver expectancy, special attention should be given to signing and the provision for decision sight distances to alert the driver an unusual condition exists.

503.5.3 Distance Between Successive Ramp Terminals

In urban areas ramp terminals are often located in close succession. To provide sufficient weaving length and adequate space for signing, a reasonable distance should be provided between successive ramp terminals. Spacing between successive outer ramp terminals is dependent on the classification of the interchanges involved, the function of the ramp pairs (entrance or exit), and weaving potential. Minimum spacing for various ramp combinations are shown in ***Figure 503-1a***.

Where an entrance ramp is followed by an exit ramp, that absolute minimum distance between the successive noses is governed by weaving considerations. This spacing is not applicable to cloverleaf interchanges as the distances between entrance-exit ramps noses is dependent on loop ramp radii and other factors. When the distance between successive noses is less than 1,500 ft. the speed change lanes should be connected to provide an auxiliary lane to improve traffic flow over a relatively short section of the freeway.

503.6 Single-Lane Ramp Terminals

This discussion is limited to terminals used for single-lane entrance and exit ramps only. See ***Section 505*** for multi-lane transitions. Ohio's standards currently permit a parallel exit terminal and tapered entrance terminal.

503.6.1 Terminal Classification

Ohio uses two basic ramp terminal classifications.

High-Speed Terminals (See ***Figures 503-2a, 503-2b*** and ***503-2c***, along with ***Figures 503-3a, 503-3b*** and ***503-3c***) - High-Speed terminals are intended for use on all Interstate highways and on other limited access freeways or expressways having similar design standards and a minimum mainline design speed of 50 mph.

Low-Speed Terminals (See ***Figures 503-4a*** and ***503-4b***) - Low-Speed terminals (mainline design speeds of 45 mph or less) are intended for use on all other limited access expressways or other highways which have little or no access control except through an interchange area. Many of the features of Low-Speed terminals are applicable to a terminal of one ramp with another ramp. Low Speed terminals are also used with Low-Speed C-D Roads.

503.6.2 Single-Lane Entrance Terminals

503.6.2.1 High-Speed

The typical single-lane entrance terminal consists of two parts, an acceleration lane and a taper. The acceleration lane allows the entering vehicle to accelerate to the freeway speed and evaluate gaps in the freeway traffic. The taper is provided for the entering vehicle to merge into the chosen gap in freeway traffic. The minimum taper rate is 50:1.

The length of the acceleration lane varies depending on the design speed of the last ramp curve on the entrance ramp and the design speed of the mainline. ***Figure 503-2a*** provides the minimum lengths of acceleration lanes for entrance ramp terminals. When the average grade of the acceleration lane exceeds 3%, the acceleration length obtained from ***Figure 503-2a*** should be adjusted by the factor obtained from ***Figure 503-2b***. The acceleration lane length is measured from the last entrance ramp curve point (PT or CS) to the point where the right edge of traveled way of the ramp is 12 feet from the right edge of the through traveled way of the freeway. ***Figure 503-2c*** illustrates the typical design of a single-lane entrance ramp terminal.

If the entrance terminal results in an add-lane (no merge), delete the last 600' of the 50:1 taper of ***Figure-503-2c***. All other entrance terminal dimensions of ***Figure 503-2c*** remain the same.

Referring to ***Figure 503-2c***, when the required acceleration length (L from ***Figure 503-2a***, adjusted to grade, ***Figure 503-2b***) is less than the acceleration length provided by the 200 ft. spiral plus 650 ft. of the 50:1 taper, then a parallel acceleration length is not required and the terminal becomes the minimum acceptable design consisting of the 200 ft. spiral and the 1,250 ft. 50:1 taper.

503.6.2.2 Low-Speed

Figure 503-4a provides the Low-Speed Entrance Terminal designs for mainline design speeds equal to or less than 45 mph.

503.6.3 Single-Lane Exit Terminals

503.6.3.1 High-Speed

The typical single-lane exit terminal consists of two parts, a taper for maneuvering out of the through traffic lane and a deceleration lane to slow to the speed of the first curve on the ramp. All deceleration should occur on the full width deceleration lane and not on the mainline or the taper.

The length of the deceleration lane varies depending on the design speed of the mainline and the design speed of first geometric control on the exit ramp, usually a horizontal curve but could be the stopping sight distance on a vertical curve or the back of an anticipated traffic queue. ***Figure 503-3a*** provides the minimum lengths of deceleration lanes for exit ramp terminals. When the average grade of the deceleration lane exceeds 3 percent, the deceleration length obtained from ***Figure 503-3a*** should be adjusted by the factor obtained from ***Figure 503-3b***. The deceleration lane length is measured from the point where the taper reaches a width of 12 feet to the first point that governs the design speed of the exit ramp, usually the PC of the first curve. ***Figure 503-3c*** illustrates the typical design of a single-lane exit ramp terminal.

The minimum deceleration length (***Figure 503-3a***) adjusted to grade (***Figure 503-3b***) shall be 800 ft.

503.6.3.2 Low-Speed

Figure 503-4b provides the Low-Speed Exit Terminal design for mainline design speeds equal to or less than 45 mph.

503.6.4 Superelevation at Terminals

Superelevation at ramp terminals should be developed using the following guidelines:

The rate of superelevation at the entrance and exit nose shall be selected on the basis of the design speed of the ramp at the nose.

All transverse changes or breaks in superelevation shall be made at joint lines (**See Standard Construction Drawing BP-6.1**). In the case of bituminous pavement, the superelevation breaks should occur in the same locations as they would in concrete pavement.

For High-Speed terminals, the transverse breaks in superelevation cross-slope shall not exceed a differential of 0.032 at the mainline edge of traveled way or 0.050 at other locations. If a double break occurs on longitudinal joints less than 6 ft. apart, it shall not exceed a total differential of 0.032, if adjacent to the mainline, or 0.050 elsewhere. On Low-Speed terminals the transverse breaks in superelevation cross-slope shall not exceed a differential of 0.05 to 0.06.

For High-Speed terminals, the rate of rotation of a superelevated ramp pavement or speed change lane pavement shall be in accordance with ***Section 202.4***.

Where possible, the terminal area pavement and shoulder should slope away from the mainline pavement so that a minimum amount of water drains across the mainline pavement.

503.6.5 Terminals on Crest Vertical Curves

Mainline crest vertical curves in the vicinity of ramp terminals should be designed using decision stopping sight distances. When it is not feasible to provide decision stopping sight distance, at a minimum, 125% SSD **(*Figure 201-1*)** should be provided. Where a crest vertical curve occurs on an exit ramp at or near the nose, the crest vertical curve should be designed using the "upper range" design speeds of ***Figure 503-1***.

503.7 Ramp At-Grade Intersections

Ramp at-grade intersections are designed using much of the same criteria as outlined in ***Section 401*** (the normal design vehicle for Interstate ramps is the WB-62 [WB-19]). However, one of the basic differences is the one-way nature of ramps and the fact that most traffic at ramp intersections is turning. ***Figure***

503-5 shows the design of a typical uncurbed ramp intersection. Curbed returns are normally used in urban areas where space is more restricted. Intersection Sight Distance, ***Section 201.3***, should be provided at all ramp at-grade intersections.

Exit ramps may require multiple lanes at the crossroad intersection to provide additional storage and capacity. ***Figure 503-5a*** illustrates alternate ways to transition from a single lane exit ramp to two lanes. The additional lane is usually provided for the minor movement.

504 COLLECTOR - DISTRIBUTOR (C-D) ROADS

504.1 Use of C-D Roads

The reason for using C-D Roads is to minimize weaving problems and reduce the number of conflict points (merging and diverging) on the mainline. C-D Roads may be used within a single interchange, through two adjacent interchanges, or continuously through several interchanges.

504.2 Design of C-D Roads

When a C-D Road is provided between interchanges, a minimum of two lanes should be used. Either one or two lanes may be used on C-D Roads within a single interchange. The cross section elements for one and two lane C-D Roads should be in accordance with the one lane and two lane directional roadways shown in ***Figure 303-1***. The design speed of a C-D Road should normally be the same as the mainline design speed but may be reduced by not more than 10 mph.

The separation between the mainline and C-D Road pavements should be designed to prevent, or at least discourage, indiscriminate crossovers. As a minimum, the separation should be wide enough to provide normal shoulder widths for both the mainline and C-D Road roadways plus a suitable median. Normally, a standard concrete barrier median is used since C-D Road separation often involves obstructions such as bridge parapets, piers or overhead sign supports. There may be isolated cases where a lesser type median may be used.

504.3 C-D Road Entrance and Exit Terminals

Figure 504-1 shows both Low-Speed and High-Speed C-D Road entrance terminals. Three exit terminal lane conditions are shown on ***Figure 504-2***. These terminal designs are to be applied to highways using High-Speed exit terminals.

Superelevation at C-D Terminals shall be developed similar to that described in ***Section 503.6.4***.

505 MULTI-LANE RAMP & ROADWAY TERMINALS AND TRANSITIONS

When two roadways converge or diverge, the less significant roadway should exit or enter on the right. Left-hand exits or entrances are contrary to driver expectancy and should be avoided wherever possible.

505.1 Multi-lane Entrance Ramps and Converging Roadways

505.1.1 General

Figure 505-1a shows the design to be used for multi-lane entrance ramps and converging roadways. Converging roadways are defined as separate and nearly parallel roadways or ramps which combine into a single continuous roadway or ramp having a greater number of lanes beyond the nose than the number of lanes on either approach roadway. (Single-Lane Entrance Terminals should be used in lieu of Converging Roadway drawings when a speed change lane is required.)

Figure 505-1b shows the specific design to be used for two-lane High-Speed entrance ramps.

High-Speed Converging Roadways should be used when either or both of the Converging Roadways are mainline roadways of an expressway or freeway or if the design speed of converging directional ramps is 50 mph or higher. Low-Speed Converging Roadways should be used at the convergence of directional ramps within an interchange or at the convergence of interchange ramps with non-limited access roads or streets where design speeds are 45 mph or lower.

505.1.2 Lane Balance and Continuity

In order to avoid inside merges, the number of mainline lanes plus converging lanes approaching the nose must be equal to the resultant number of lanes leaving the nose. To make this possible, it is often necessary to carry additional mainline lanes past the nose for an adequate distance prior to tapering back to the desired number of lanes. These details are shown in ***Figure 505-1a.***

505.1.3 Inside Merges

When using a taper type of multilane entrance ramp an “inside merge” is created with traffic traveling on both sides of the merging lanes. If either vehicle involved with the merging movement abandons the merge, traffic in the adjacent lanes could prevent the merging vehicles from escaping to the adjacent lanes. By contrast, the parallel type multilane entrance ramp, as shown in ***Figure 505-1a***, allows the merging vehicle to escape to the right shoulder without any interference. For the above reasons, inside merges are not desirable.

505.1.4 Preferential Flow

On ***Figure 505-1a***, one roadway in each design is labeled PREFERENTIAL FLOW. This indicates the more important of the two approaching traffic flows. In selecting the preferential flow a designer must consider the effect of traffic volumes, number of lanes, sign route continuity and importance, vehicle speeds and roadway alignment. Lanes carrying the preferential flow are given the higher design treatment. When it is necessary to reduce a number of converging lanes or where an angular change in direction must occur, the design should favor the preferential flow.

505.1.5 Horizontal Curvature

Horizontal curves of roadways approaching the terminal nose should conform to mainline roadway criteria in the case of mainline roadways and to ramp entrance terminal criteria in the case of ramps.

505.1.6 Crest Vertical Curves

Crest vertical curves on constant-width roadways approaching the merging nose should be designed to provide sight distance consistent with the design speed of the roadway.

Crest vertical curves from the merging nose forward to a point where pavement convergence ceases and to the converging portion of an approaching roadway where the number of lanes is being reduced in advance of the nose should be designed using the decision stopping sight distance shown in ***Figure 201*-6**. (See ***Figure 505-1a***.) When it is not feasible to provide decision stopping sight distance, at a minimum, 125% SSD (**Figure 201-1**) should be provided.

When design speeds differ on approaching roadways, the higher of the two design speeds shall be used in designing the crest vertical curve beyond the merging nose.

505.1.7 Superelevation and Joint Location

Reference shall be made to ***Section 503.6.4*** for superelevation requirements.

Longitudinal joints should be located so they will coincide with and define the lane lines. Reference should be made to **Standard Construction Drawing BP-6.1** for type and location.

505.2 Multi-lane Exit Ramps and Diverging Roadways

505.2.1 General

Figure 505-2a shows the general design for multi-lane exit ramps and diverging roadways. A diverging roadway is defined as a single roadway which branches or forks into two separate roadways without the need of a speed change lane.

Figure 505-2b shows the specific designs to be used for a two-lane high-speed exit ramp at a system interchange or the exit from the mainline of a two-lane CD-Road. Type I should normally be used. Type II should only be used when queuing in the optional lane does not extend to the physical gore (long ramps, Parclo B, etc.).

Figure 505-2c shows examples of designs for diverging roadways.

Figure 505-2d shows the specific designs to be used for a two-lane high-speed exit ramp at a service interchange. Type I should normally be used. Type II should only be used when queuing in the optional lane does not extend to the physical gore (long ramps, Parclo B, etc.).

High-Speed Diverging Roadways should be used when either or both the diverging roadways are mainline roadways of an expressway or freeway or at the divergence of high-speed directional ramps within an interchange. Low-Speed Diverging Roadways should be used at the divergence of low-speed directional ramps within an interchange or at the divergence of ramps with non-limited access roads or streets.

505.2.2 Lane Balance and Continuity

In order to have lane continuity, the number of mainline lanes leaving the diverging nose must be equal to the number of mainline lanes approaching the nose. The total number of lanes leaving the diverging nose (mainline lanes plus diverging lanes) must be one greater than the total number of lanes approaching the nose to obtain lane balance. The purpose for obtaining lane continuity and lane balance is to avoid a drop lane situation. See ***Figures 505-2a*** and ***505-2b***.

It may be necessary to obtain this lane balance by adding additional lanes upstream from the diverging nose. The length of each additional lane should be 2,500 ft. and should be introduced using a 0 to 12 ft. taper with a length of 100 ft. as shown on ***Figure 505-2b*** for the approach roadway class and design speed.

There may be conditions off the mainline, such as on Collector-Distributor Roads or within interchanges, where lane balance and continuity is less important. In such cases, the non-mainline roadway design on ***Figures 505-2a*** and ***505-2b*** may be used.

505.2.3 Terminal Design

The design of diverging roadway terminals is determined by the class and the design speed of the approach roadway, and is based on the neutral gore length "L" and the nose width "N" (See ***Figure 505-2a***).

Table A on ***Figure 505-2a*** lists length "L" and nose width "N" for various design speeds in diverging roadway classes. The "N" dimension should be exact, but the "L" dimensions may vary slightly from the Table A value.

505.2.4 Horizontal Curvature

Table B on ***Figure 505-2a*** lists recommended values for the curve differential between the outer edges of traveled way of diverging roadways. These values apply only when the alignment between the diverging nose and the PC of the diverging curvature is on tangent or simple curvature.

When compounded or spiral curvature is used in the diverging area, it will be necessary to design diverging roadway alignments individually to provide the proper "L" and "N" for the approach roadway Class and design speed.

505.2.5 Crest Vertical Curves

When a diverging nose is located on a crest vertical curve, this vertical curve shall be designed using the design speed of the approach highway and decision stopping sight distance from ***Figure 201-6***. When it is not feasible to provide decision stopping sight distance, at a minimum, 125% SSD (***Figure 201-1***) should be provided.

505.2.6 Superelevation and Joint Location

The superelevation rate will be based on the design speed of the approach roadway. Reference should be made to ***Section 503.6.4*** for other superelevation requirements.

Longitudinal joints should be located so they will define the lane lines. Reference should be made to **Standard Construction Drawing BP-6.1** for type and location. The joints in the gore area should be located to facilitate superelevation and pavement grading.

505.3 Four Lane Divided to Two Lane Transition

Figure 505-3 shows a reversed curve design (Types A and B) a tapered design (Type C) and a design for a transition on a curve (Type D). The pavement transition should be located in an area where it can easily be seen. Intersections or drives should be avoided in the transition area. Vertical or horizontal curves should provide decision stopping sight distance.

Reverse curve transitions should normally be used for median widths of 20 ft. or wider.

Taper lengths are calculated as shown in ***Section 401.6.1.***

506 SERVICE ROADS

506.1 Use of Service Roads

Service roads (frontage roads) are used to enhance capacity on the mainline, control access, serve adjacent properties, or maintain traffic circulation. They permit development of adjacent properties while preserving the through character of the mainline roadway. Service roads may be either one-way or two-way, depending on where they are located and the purpose they are intended to serve.

506.2 Design of Service Roads

Although the alignment and profile of the mainline may have an influence, service roads are generally designed to meet the specific criteria based on functional classification (usually "local"), traffic volumes, terrain/locale and design speed. Two features, however, are unique to service roads and are further discussed below. They are (1) the separation between the service road and mainline and (2) the design of the crossroad connection.

The further the service road is located from the mainline, the less influence the two facilities will have on each other. A separation width that exceeds the clear zone measurement for each roadway is desirable. However, the separation should be at least wide enough to provide normal shoulder widths on each facility plus accommodate surface drainage and a suitable physical traffic barrier. Glare screen is desirable to screen headlights when the service road is two-way.

At crossroads, the distance between the mainline and service road becomes extremely critical. This distance should be great enough to provide adequate storage on the approaches to both the mainline and service road. The recommended minimum distance between the mainline and service road edges of traveled way is 150 ft. in urban areas and 300 ft. in rural areas. In addition, the designer should check the adequacy of stopping sight distance on the crossroad as well as intersection sight distance at the frontage road.

Since service roads are normally maintained by local governmental agencies, the pavement design should either meet, or exceed, that required by the maintaining agency.

550 REQUESTS FOR NEW OR REVISED ACCESS INTERSTATE HIGHWAYS OR OTHER FREEWAYS

550.1 General

Control of access on the Interstate and other freeway systems is considered critical to providing the highest quality of service in terms of safety and mobility. This section provides guidance for the preparation and processing of access point requests in relation to new and existing interchanges on the Interstate and other freeway systems in accordance to Federal Code 23 U.S.C. 111 and FHWA's Policy on Access to the Interstate System, dated May 22, 2017.

The documentation required depends on the type of change requested - new or revised.

New Access is the addition of a point of access where none previously existed. This includes the construction of an entirely new interchange such that it will result in additional points of access or additional ramps to existing interchanges. As an example, the reconstruction of an existing diamond interchange to a full cloverleaf interchange would add four new points of access.

Revised Access is the major revision of an existing interchange such that the number of access points will remain the same but the operation and/or safety of the Interstate/freeway system may be affected. The changing of a cloverleaf interchange to a fully directional interchange, the conversion of a traditional diamond to a diverging diamond interchange, relocating an existing ramp to terminal to a new roadway, and adding a collector-distributor system are all considered examples of revised points of access.

New or revised access point requests require the preparation and processing of an Access Point Request Document. Generally, a new access requires an Interchange Justification Study (IJS) or an Interchange Modification Study (IMS). A revised access requires an Interchange Modification Study (IMS) or an Interchange Operations Study (IOS).

550.2 Interchange Study (Access Point Request Document)

The degree of complexity of the Interchange Study will vary depending on the character of the location (urban or rural) and/or whether the change involves a revised access point, a new access point at an existing interchange, or an entirely new interchange location. To coincide with FHWA's Policy on Access to the Interstate System, the following is a list of items which must be addressed in the interchange study for a new or revised access on the Interstate/freeway system:

1. Evidence that the proposed new or revised access does not have significant adverse impact on the safety and operation of the Interstate/freeway system. The analysis must address design year traffic with and without the new or revised access point (build vs. no-build). Design year traffic must reflect future land use changes and associated trip generations. Traffic projections must be certified as per ***Section 102.1***.

   Requests involving new access points or revised access points must use 20 year design traffic projected from the opening day of the interchange. Certified Traffic (High Risk Design Traffic Forecasting Procedure) will be required for all projects involving an IMS or IJS, in accordance with the most current version of the Ohio Traffic Forecasting Manual.

   The level-of-service (LOS) of the Interstate/freeway system and the interchange components that are built new or modified should generally provide a LOS C, except certain cases in the MPO's Boundary where LOS D may be acceptable.

   The proposed Interstate/freeway interchange or improvements cannot have a significant adverse impact on the safety and operation of the Interstate/freeway facility based on an analysis of design year traffic. A significant impact occurs when the Build Condition degrades traffic operations, see ***Section 550.3.2*** to determine if degradation occurs.

   The operational analysis shall, particularly in urban areas, include an analysis of sections of Interstate/freeway to and including at least the first adjacent existing or proposed upstream and downstream interchange. Crossroads and other roads and streets shall be included in the analysis to the extent necessary to assure their ability to collect and distribute traffic to and from the interchange with new or revised access points. New interchanges must include analysis of the local street system to the extent that local road system improvements can be compared as an alternative to constructing a new interchange. Maps and/or diagrams should be provided as needed to clearly describe the location and study limits of the proposal.

   For requests involving entirely new interchanges, the study should include a discussion of the distance to, and size of, communities to be served by the new interchange. An examination of proper interchange spacing must also be included.

   Every IMS/IJS should include a conceptual signing plan showing the type and location of signs to support the proposed design.

2. Assurance that the new or revised access connects to a public road and is part of a configuration that provides for all traffic movements. Less than "full interchanges" for special purpose access for transit vehicles, for HOV's, or into park and ride lots may be considered on a case-by-case basis. Proposed design must meet or exceed current design standards.

   In rare instances where all basic movements are not provided by the proposed design, the report must include a full-interchange option with a comparison of the operational and safety analyses to the partial-interchange option. The report must also include the mitigation proposed to compensate for the missing movements, including wayfinding signage, impacts on local intersections, mitigation of driver expectation leading to wrong-way movements on ramps, etc. The report must describe whether future provision of a full interchange is precluded by the proposed design.

The development of an Access Point Request Document should be performed in accordance with the ODOT Project Development Process (PDP). As part of the PDP for all projects that require an IJS/IMS, the relevant PDP submissions (including, but not limited to the Feasibility Study and Alternative Evaluation Report), will include consideration of the following points:

1. Adequate documentation that the existing access points and/or local roads are unable to handle the design year traffic demands while providing the access intended by the proposal, or be improved to do so, if the new or revised access is not provided. If the request involves a new access point, and particularly an interchange at a new location, a comprehensive description of the public need for the access must be included. A justification based on enhanced property values or access to private facilities will not be accepted.

2. Assurance that all reasonable alternatives for design options, location, and transportation system management type improvements (such as ramp metering, mass transit, and HOV facilities) have been assessed and provided for if currently justified, or provisions are included for accommodating such facilities if a future need is identified.

3. The proposal considers and is consistent with local and regional land use and transportation plans. Prior to final approval, all requests for new or revised access must be consistent with the metropolitan and/or statewide transportation plan, as appropriate, the applicable provisions of 23 CFR part 450 and the transportation conformity requirements of 40 CFR parts 51 and 93.

   The request should include a statement and analysis of compatibility with, and the effect on, the local road network. Letters of support and commitment are required from the State and other sponsoring agencies for any required street or road improvements as well as for the access point.

4. In areas where the potential exists for future multiple interchange additions, all requests for new or revised access are supported by a comprehensive Interstate/freeway network study with recommendations that address all proposed and desired access within the context of a long-term plan.

5. Evidence that the request for the new or revised access generated by new or expanded development demonstrates appropriate coordination between the development and the necessary transportation improvements. A discussion of potential funding sources, if known, should be included.

6. The request for new or revised access contains information relative to the planning requirements and the status of the environmental processing of the proposal.

As part of ODOT's Project Development Process, the Office of Roadway Engineering is required to review and approve all Feasibility Studies and Alternative Evaluation Reports involving an Interchange Study (IJS/IMS/IOS). If the FS and/or AER involves an interchange, the study limits must encompass the applicable interchange study (IJS/IMS/IOS) limits.

The Office of Roadway Engineering will not review any Interchange Study (IJS/IMS/IOS) that:

1. Does not have an approved Purpose & Need (if applicable); or
2. Does not have appropriate study limits required to support the approved Purpose & Need; or

3. Has interchanges that ORE did not review and approve in the Feasibility Study and Alternative Evaluation Report (if applicable)

The Access Point Request Document should only be performed for the preferred alternative, however a discussion of feasible alternatives should also be included in the study. The preferred alternative will comply to all State and FHWA design requirements, including but not limited to: interchange spacing, interchanges to provide for all traffic movements to and from the freeway, not allowing lanes to drop into private facilities, not allowing intersections (driveways or streets) to intersect ramps (except in special cases such as facilities for utilities

Interchange Modification and Justification Studies (IMS & IJS) are required to reference and describe how each policy point in FHWA's Policy on Access to the Interstate System is being met (link to latest policy: https://www.fhwa.dot.gov/design/interstate/170522.cfm).

All IJS or IMS documents should follow the Report Format/Outline found in the Traffic Academy Interchange Studies (IJS/IMS/IOS) Course Manual. The Interchange Studies Course Manual can be found on the following website:
http://www.dot.state.oh.us/Divisions/Engineering/Roadway/studies/Pages/IJSandIMS.aspx

A reevaluation of the IJS/IMS may be required by FHWA if the project or a phase of the project has not been constructed within 3 years of the approval date of the document.

An IJS/IMS Addendum is required if any of the following condition(s) apply:

1. A Revised-Build condition is proposed that is different than the Build condition (per the approved IJS/IMS) and is not an Interim-Build condition (a phased condition between the No-Build condition and Build condition, per the approved IJS/IMS).

See ***Section 550.2.1*** if your project does not meet the condition(s) listed above. Contact The Office of Roadway Engineering.

550.2.1 Interchange Operations Study (IOS)

Many minor interchange projects, especially those involving service interchanges, do not fall under the definition of warranting an Access Point Request Document (IJS/IMS) per the FHWA's Policy on Access to the Interstate System, but still require an operational evaluation and approval by the Office of Roadway Engineering. This operational evaluation would be in the form of a report referred to as the Interchange Operations Study, IOS. The IOS is intended to be an abbreviated version of the more comprehensive IMS report, highlighting critical traffic operations that may be affected by the proposed improvement. The IOS will utilize the same analysis methodology and 20 yr. design as the IMS, but the IOS will be more limited with respect to the number of analysis points evaluated and the study narrative. For an IOS, Certified Traffic is typically not required. Instead, the Low Risk Design Traffic Forecasting Procedure, per the Ohio Traffic Forecasting Manual, is considered acceptable. Certified Traffic is required when thru lanes on the freeway/crossroad are increased or decreased. In urban areas with significant congestion and oversaturated conditions, coordinate with the Office of Modeling & Forecasting to discuss if Certified Traffic is needed to capture the full demand volumes. An IOS can be applied to an Interstate or non-Interstate. The following is a list of projects requiring an IOS:

1. Changing lane configurations at a ramp intersection approach, including:
    - Adding/removing a left, thru, or right turn lane along a crossroad

- Adding/removing turn lanes to the exit ramp
- Changing lane assignments without altering the number of lanes
    a. Example: Changing a 2-lane approach from a (Left/Thru-Right) to (Left-Thru/Right)
- Implementing a Road Diet (reducing the number of lanes on the crossroad)
- "Squaring" up a continuous right turn from the ramp/crossroad and regulating the movement with a signal
- Converting a "squared" right turn to/from ramp/crossroad to a slip ramp

2. Changing the exit or entrance ramp terminus point with the freeway mainline by:
    - Creating an optional exit lane
    - Creating a 2-lane exit
    - Creating a 2-lane entrance
3. Shifting a ramp's location within the same interchange configuration
4. Changing traffic control type at a ramp/crossroad intersection from a signalized/unsignalized condition to a roundabout
5. Adding an auxiliary lane between 2 adjacent ramp interchange ramps

An IOS cannot be considered for: A) Major interchange design/revisions; B) New interchanges; or C) Interchange modifications necessitated by large developments or conditions that will significantly increase traffic volumes or revise traffic patterns.

For all other interchange or mainline modifications that result in significant operational changes, not covered above or by an Interchange Modification Study, please contact the Office of Roadway Engineering.

An IOS may be required if an interim (phased) condition does not match the approved build condition in the IJS/IMS. Contact the Office of Roadway Engineering for further guidance.

A reevaluation of the IOS may be required by ODOT if the project or a phase of the project has not been constructed within 3 years of the approval date of the document.

550.2.2 Safety Improvements on Interstate or Other Freeways

Safety improvements eligible for this process are defined as low to medium cost solutions that address an identified "spot" safety problem. The LOS provisions of ***Section 550.2*** do not apply except that the LOS should not be degraded over the no-build condition in the design year. All other provisions of 550.2 still apply, including the IMS or IOS report to support the analyses. To determine degradation, the individual operational components shall be analyzed, but evaluated for acceptance within the context of the overall affected system. Though a single operational component could experience incremental degradation, the overall system should improve or essentially remain the same. For a safety improvement to qualify under this section, the following criteria must be met:

1. The project purpose and need is primarily to address "spot" safety problems. The purpose and need may not include operational performance or economic development objectives.
2. The location has separate independent utility from all other improvements
3. Any potential longer term solution which would provide LOS C would take 5 or more years to implement.

4. No major rehabilitation or reconstruction is planned for 5 or more years. Other work (e.g., routine maintenance or minor rehabilitation) may be done within the 5 year window as long as it does not substantially replace the base pavement and/or reconfigure the facility.
5. The location is a spot location (defined as a ramp, intersection, merge/diverge point, weave, or mainline section not to exceed one mile).
6. The location planning level cost estimate is less than $5 million total (low to medium cost measures) for all phases of project development (i.e. preliminary engineering, detail design, right of way and construction).

550.3 Study Methodology

550.3.1 General

One of the primary objectives of an Interchange Study is to determine if additional traffic enters the Interstate/freeway in the build versus the no-build case, and if traffic does increase, does it degrade the operation of the Interstate/freeway. In cases of new interchanges or new access points, the new roadway and connections will generally result in changed traffic patterns from the no-build case. In the case of revised access projects, the build and no-build traffic volumes may be identical. In these cases, it is important to understand the concept of constrained traffic.

Another primary objective is to ensure vehicles exiting the Interstate/freeway do not negatively impact vehicles remaining on the Interstate/freeway. Providing adequate storage for the off-ramp and optimizing signal progression away from a service interchange are two ways this is accomplished.

550.3.2 Constrained Traffic

In many cases, the purpose of a project is to alleviate traffic congestion at an interchange, possibly due to over saturated ramp terminal intersections or inadequate ramp capacity. In these cases, the proposed solution generally includes capacity improvements such as turn lanes and/or additional through lanes intended to remove the geometric constraint, or “bottleneck”. In order to determine the effect of the proposed improvement on the Interstate/freeway, traffic analysis tools such as the Highway Capacity Manual (HCM) or Highway Capacity Software (HCS) must be used.

The following steps are needed to determine if degradation occurs when comparing the No-Build condition to the Build condition:

1) Referencing the table below, determine if degradation may be occurring by comparing the HCS level of service (LOS) for the No-Build and Build conditions downstream of the on-ramp at a) the merge and downstream freeway segment, or b) the weave.

LOS		Is there Degradation?	
No-Build	Build	Urban/ Suburban	Rural
A, B, C	A, B, C	No	No
C	D	No	Check
D	D	No	Check
	E	Check	Check
E	E	Check	Check
	F	Check	Check
F	F	Check	Check

2) If "No" degradation occurs then nothing else required. If "Check" then prepare/calculate constrained analyses, see Traffic Academy Interchange Studies (IJS/IMS/IOS) Course Manual and Example Problem
550-1.

3) Intersections are to be analyzed per ***Section 401.2*** and ***Figure 401-14a***, using design-year traffic, for both the AM peak hour and PM peak hour, or midday peak hour, if applicable. Degradation occurs when the Build traffic volume is greater than or equal to 2.0% of the No-Build traffic volume, see Example Problem 550-1.

4) If the constrained analyses result in a traffic volume increase greater than 2 percent, the project will not be permitted unless mitigative measures are included to either restrain vehicles from entering the freeway (i.e., ramp metering, geometric constraints), or additional capacity is provided on the freeway to restore the LOS. ODOT and FHWA will decide what mitigative measures, if any, will be allowed.

5) For over saturated movements, the demand volume should be divided by the volume-to-capacity (V/C) ratio of that movement to determine the actual, or constrained, flow volumes to be used in the downstream merge and mainline, or weave LOS calculations. The difference between the no-build constrained traffic flow and the build (typically unconstrained, or less constrained) traffic flow is the increase of traffic volume entering the Interstate/freeway.

550.3.3 Diagrams and Plans

The Access Point Request Document should contain diagrams and plans as needed (as applicable) to indicate: project limits, adjacent interchanges, proposed interchange configuration, travel lanes and shoulder widths, ramps to be added, ramps to be removed, ramp radii, ramp grades, acceleration lane lengths, deceleration lane lengths, taper lengths, auxiliary lane lengths, and collector/distributor roads.

550.4 Submission of Interchange Studies

All request submissions are to be sent to the Office of Roadway Engineering (electronic copy) with two printed copies to the ODOT District Office. The Office of Roadway Engineering will be responsible for coordination with the Federal Highway Administration for studies involving Interstates.

All traffic analysis files must be submitted (HCS, Synchro, SIDRA, etc.)

550.5 Review of Interchange Studies

For Interstates, the Office of Roadway Engineering will review and approve the Access Point Request Document (IJS or IMS), and if acceptable, will forward the request to FHWA for their approval. If the environmental document has not been completed, approval will be conditional on acceptance of the environmental document.

For Access Point Request Documents (IJS or IMS) involving non-Interstate freeways, the Office of Roadway Engineering will review the study and has approval authority.

For Interchange Operations Studies, the Office of Roadway Engineering will review and has approval authority. As a courtesy, all IOS submissions involving the Interstate will be made available electronically to FHWA.

LIST OF FIGURES

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RAMP DESIGN SPEED GUIDE

503-1

REFERENCE SECTIONS
503.2

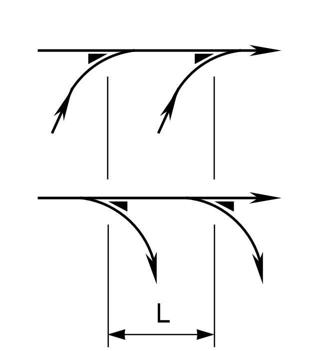
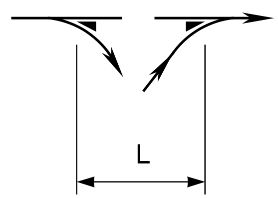
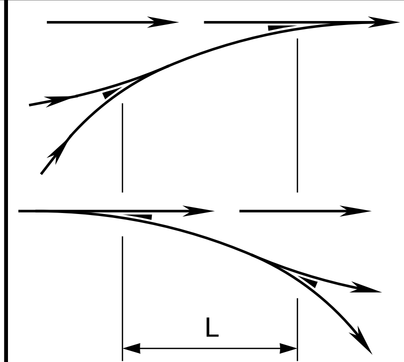
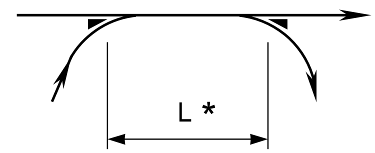
RAMP DESIGN SPEED (mph)	MAINLINE DESIGN SPEED (mph)									
	30	35	40	45	50	55	60	65	70	75
UPPER RANGE	25	30	35	40	45	48	50	55	60	65
MIDDLE RANGE	20	25	30	33	35	40	45	45	50	55
LOWER RANGE	15	18	20	23	25	28	30	30	35	40

Note: Ramp design speeds do not pertain to the ramp terminals.

MINIMUM RAMP TERMINAL SPACING

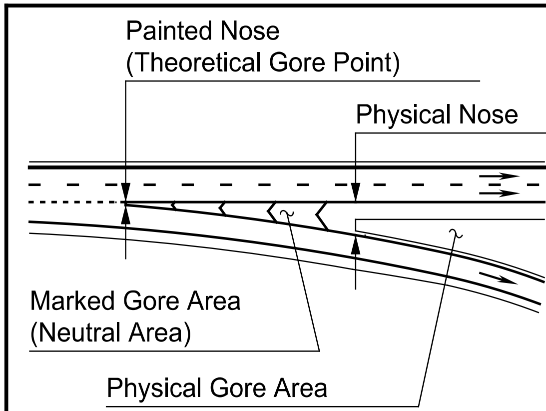
503-1a

REFERENCE SECTIONS
503.5.3

ENTRANCE-ENTRANCE OR EXIT-EXIT		EXIT-ENTRANCE		TURNING ROADWAYS		ENTRANCE-EXIT (WEAVING)			
 L		 L		 L		 L * * NOT APPLICABLE TO CLOVERLEAF LOOP RAMPS			
FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR	SYSTEM INTER-CHANGE	SERVICE INTER-CHANGE	SYSTEM TO SERVICE INTERCHANGE		SERVICE TO SERVICE INTERCHANGE	
						FULL FWY.	CDR OR FDR	FULL FWY.	CDR OR FDR
MINIMUM LENGTHS MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS									
1000 ft	800 ft	500 ft	400 ft	800 ft	600 ft	2000 ft	1600 ft	1600 ft	1000 ft

NOTES: CDR - COLLECTOR DISTRIBUTOR ROAD
FDR - FREEWAY DISTRIBUTOR ROAD

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 (HCM).** Also refer to the HCM for the procedure for measuring the length of the weaving section. The "L" distances noted in the figures above are measured between the painted noses (theoretical gore point). Additionally for EN-EN, a minimum distance of 300 ft. is recommended between the end of the taper for the first entrance ramp and the painted nose for the succeeding entrance ramp (similar for EX-EX except use the physical nose).



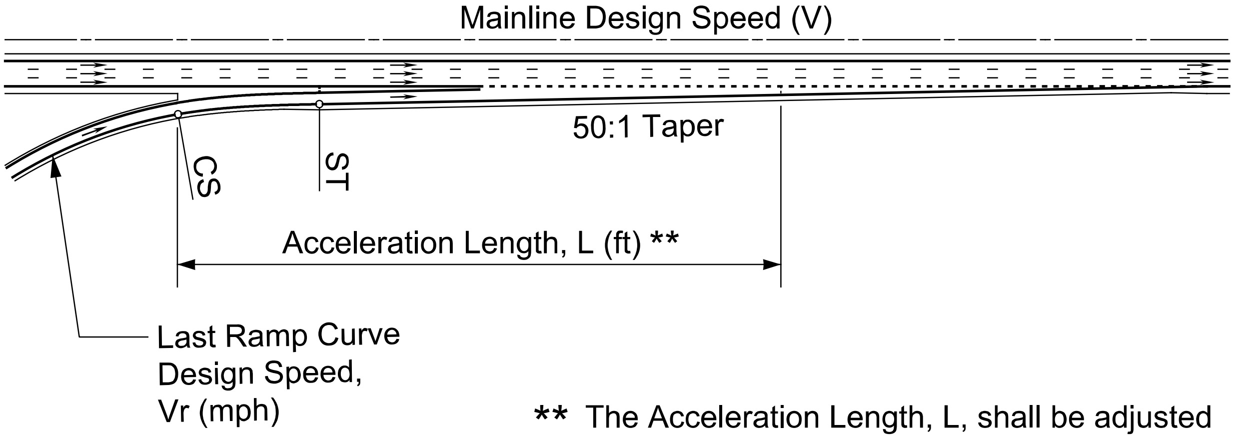
TYPICAL GORE AREA CHARACTERISTICS

MINIMUM ACCELERATION LENGTHS FOR HIGH-SPEED ENTRANCE TERMINALS WITH FLAT GRADES OF 2% OR LESS

503-2a

REFERENCE SECTION 503.6.2

Mainline Design Speed, V (mph)	Acceleration Length, L (ft) for design Speed of Last Ramp Curve, Vr (mph)								
	Stop	15	20	25	30	35	40	45	50
50	720	660	610	550	450	350	130	-	-
55	960	900	810	780	670	550	320	150	-
60	1200	1140	1100	1020	910	800	550	420	180
65	1410	1350	1310	1220	1120	1000	770	600	370
70	1620	1560	1520	1420	1350	1230	1000	820	580
75	1790	1730	1630	1580	1510	1420	1160	1040	780



** The Acceleration Length, L, shall be adjusted for Grade with Figure 503-2b.

HIGH-SPEED ENTRANCE TERMINAL ADJUSTMENT FACTORS AS A FUNCTION OF GRADE

503-2b

REFERENCE SECTION 503.6.2

Mainline Design Speed, V (mph)	Ratio of Length On Grade to Length On Level for Design Speed of Last Ramp Curve, Vr (mph)*							
	Upgrade							Downgrade
	20	25	30	35	40	45	50	All Speeds
	3% to 4%							3% to 4%
50	1.30	1.35	1.40	1.40	1.40	-	-	0.650
55	1.35	1.40	1.45	1.45	1.45	-	-	0.625
60	1.40	1.45	1.50	1.50	1.50	1.55	1.60	0.600
65	1.45	1.50	1.55	1.55	1.60	1.65	1.70	0.600
70	1.50	1.55	1.60	1.65	1.70	1.75	1.80	0.600
	5% to 6%							5% to 6%
50	1.50	1.60	1.70	1.80	1.90	-	-	0.550
55	1.60	1.70	1.80	1.90	2.05	-	-	0.525
60	1.70	1.80	1.90	2.05	2.20	2.35	2.50	0.500
65	1.85	1.95	2.05	2.20	2.40	2.60	2.75	0.500
70	2.00	2.10	2.20	2.40	2.60	2.80	3.00	0.500

No adjustment required for Grades less than 3%.

* Ratio from this table multiplied by acceleration length in Figure 503-2a gives acceleration length on grade.

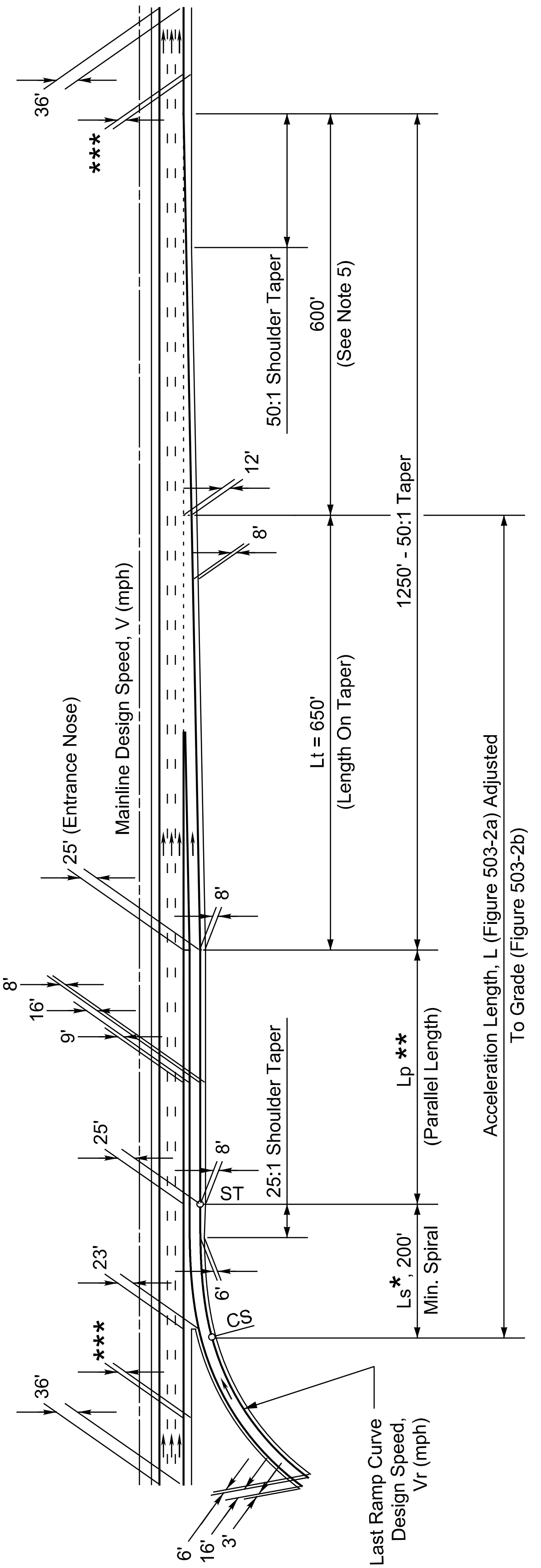
The "grade" in the table is the Average Grade measured over the distance for which the Acceleration Length applies.

For Mainline Design Speeds greater than 70 mph, use 70 mph design speed Adjustment Factors.

HIGH-SPEED SINGLE-LANE ENTRANCE TERMINAL

503-2c

REFERENCE SECTIONS
503.6.2



* Length may be increased for Superelevation Transition

** To determine Lp, subtract Ls and Lt from L.

*** Mainline Paved Shoulder Width as required by Figure 301-3 or 301-4.

Notes For Single Lane Entrance Terminals

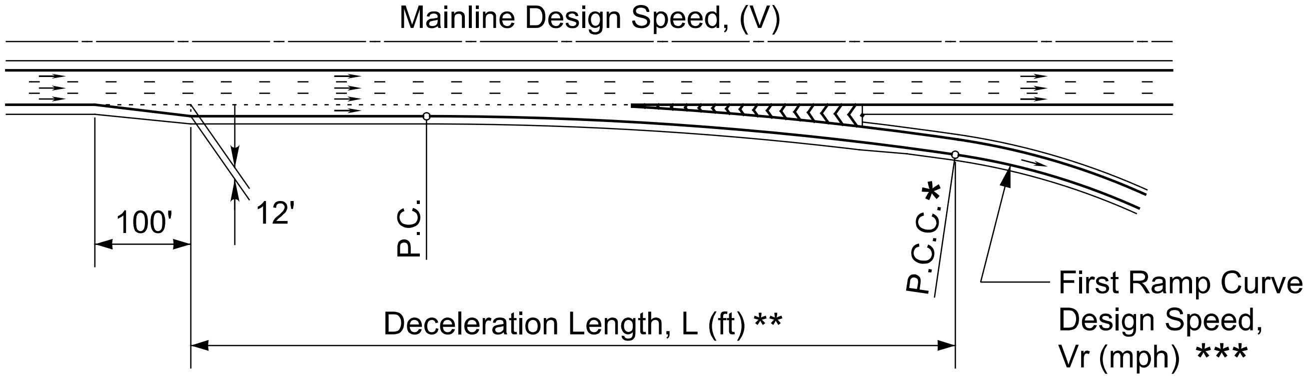
1. The minimum Acceleration Length, L, shall be Ls + Lt.
2. The 9' to 23' variable width of Treated Shoulder of the Entrance Terminal shall be sloped for 12' as required for mainline design (usually ½ in./ft.), except for the last 100' to 200' at the 9' end, which is to be sloped as required for proper terminal grading.
3. Normally Single Lane Ramps will have a Width of 16'. The Width shall be increased to 18' when the Ramp Radius is less than 200'. When an 18' wide ramp is used, the 25' Entrance Terminal Width shall be retained and the 9' width reduced by 2'.
4. If Lp (Parallel Length) is not required (L <= 850'), then the 200' minimum Spiral should be tangent to the 50:1 Taper.
5. If the Entrance Terminal results in an add-lane (no merge), delete the last 600' of the 50:1 Taper.

MINIMUM DECELERATION LENGTHS FOR HIGH-SPEED EXIT TERMINALS WITH FLAT GRADES OF 2% OR LESS

503-3a

REFERENCE SECTION 503.6.3

Mainline Design Speed, V (mph)	Deceleration Length, L (ft) for Design Speed of First Ramp Curve, Vr (mph) ***								
	Stop	15	20	25	30	35	40	45	50
50	435	405	385	355	315	285	225	175	-
55	480	455	440	410	380	350	285	235	-
60	530	500	480	460	430	405	350	300	240
65	570	540	520	500	470	440	390	340	280
70	615	590	570	550	520	490	440	390	340
75	660	635	620	600	575	535	490	440	390



* P.C.C. Or Mid-Point of 200' Spiral

** The minimum Deceleration Length, L, after Adjustment For Grade (Figure 503-3b), shall be 800'

*** Or other Design Speed limiting Geometric Control such as the Stopping Sight Distance for a Vertical Curve or the back of a Traffic Queue.

HIGH-SPEED EXIT TERMINAL ADJUSTMENT FACTORS AS A FUNCTION OF GRADE

503-3b

REFERENCE SECTION 503.6.3

Mainline Design Speed, V (mph)	Ratio of Length On Grade to Length On Level for Design Speed of First Ramp Curve, Vr (mph) *			
	Upgrade		Downgrade	
	All Speeds	All Speeds	All Speeds	All Speeds
	3% to 4%	5% to 6%	3% to 4%	5% to 6%
All Speeds	0.90	0.80	1.20	1.35

No adjustment required for Grades less than 3%.

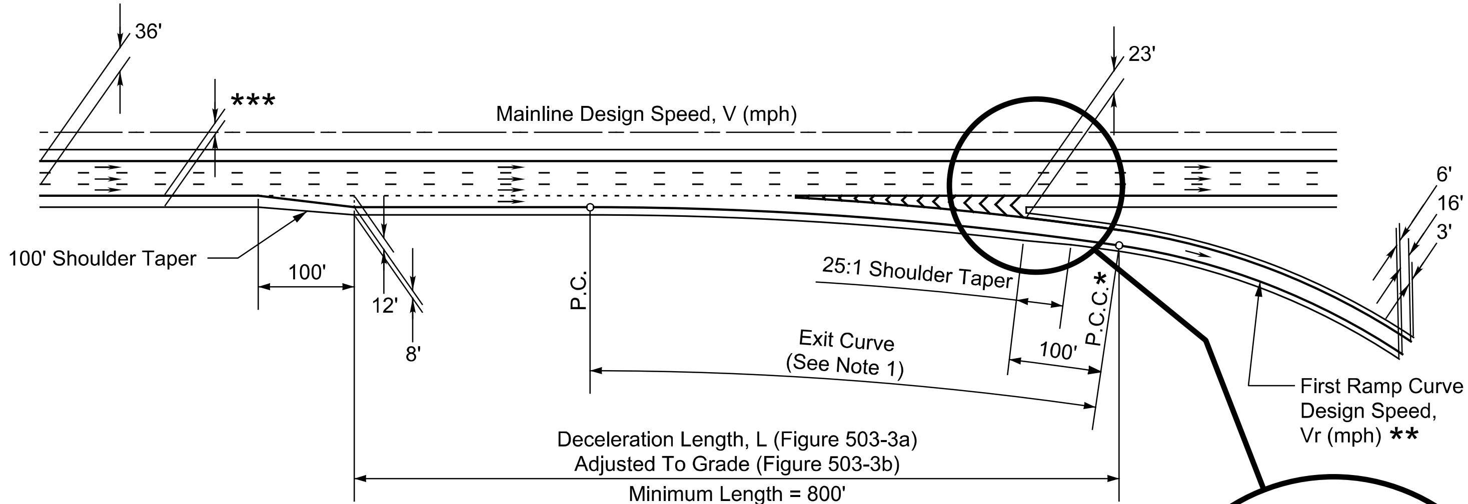
* Ratio from this table multiplied by Deceleration Length in Figure 503-3a gives Deceleration Length On Grade.

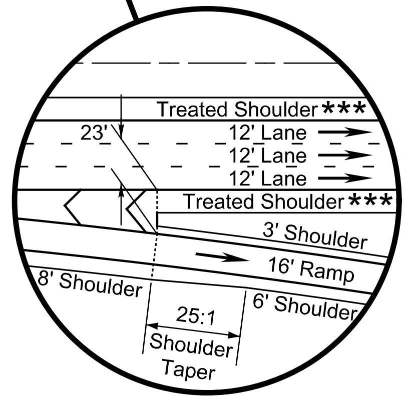
The "grade" in the table is the Average Grade measured over the distance for which the Deceleration Length applies.

HIGH-SPEED SINGLE-LANE EXIT TERMINAL

503-3c

REFERENCE SECTIONS
503.6.3





GORE DETAIL

EXIT CURVE TABLE		
Mainline Design Speed, V (mph)	Maximum Exit Curvature, Dc	
	Rural	Urban
75	1°- 13'	1°- 22'
70	1°- 24'	1°- 35'
65	1°- 35'	1°- 55'
60	1°- 50'	2°- 15'
55	2°- 10'	2°- 45'
50	2°- 40'	3°- 30'

* P..C. Or Mid-Point of 200' Spiral

** Or other Design Speed limiting Geometric Control such as the Stopping Sight Distance for a Vertical Curve or the back of a Traffic Queue.

*** Mainline Paved Shoulder Width as required by Figure 301-3 or 301-4.

Notes For High-Speed Single-Lane Exit Terminals

1. The Exit Curve should normally be according to the Exit Curve Table where the mainline is on tangent. Where the mainline is on curving alignment, the maximum differential between the Exit Curve and the mainline curve should normally be the Exit Curve Table value. This differential, however, may vary by as much as one degree in order to avoid a tangent exit alignment. (See Section 503.6.4 for the allowable Transverse Breaks in Superelevation Cross-Sope.)

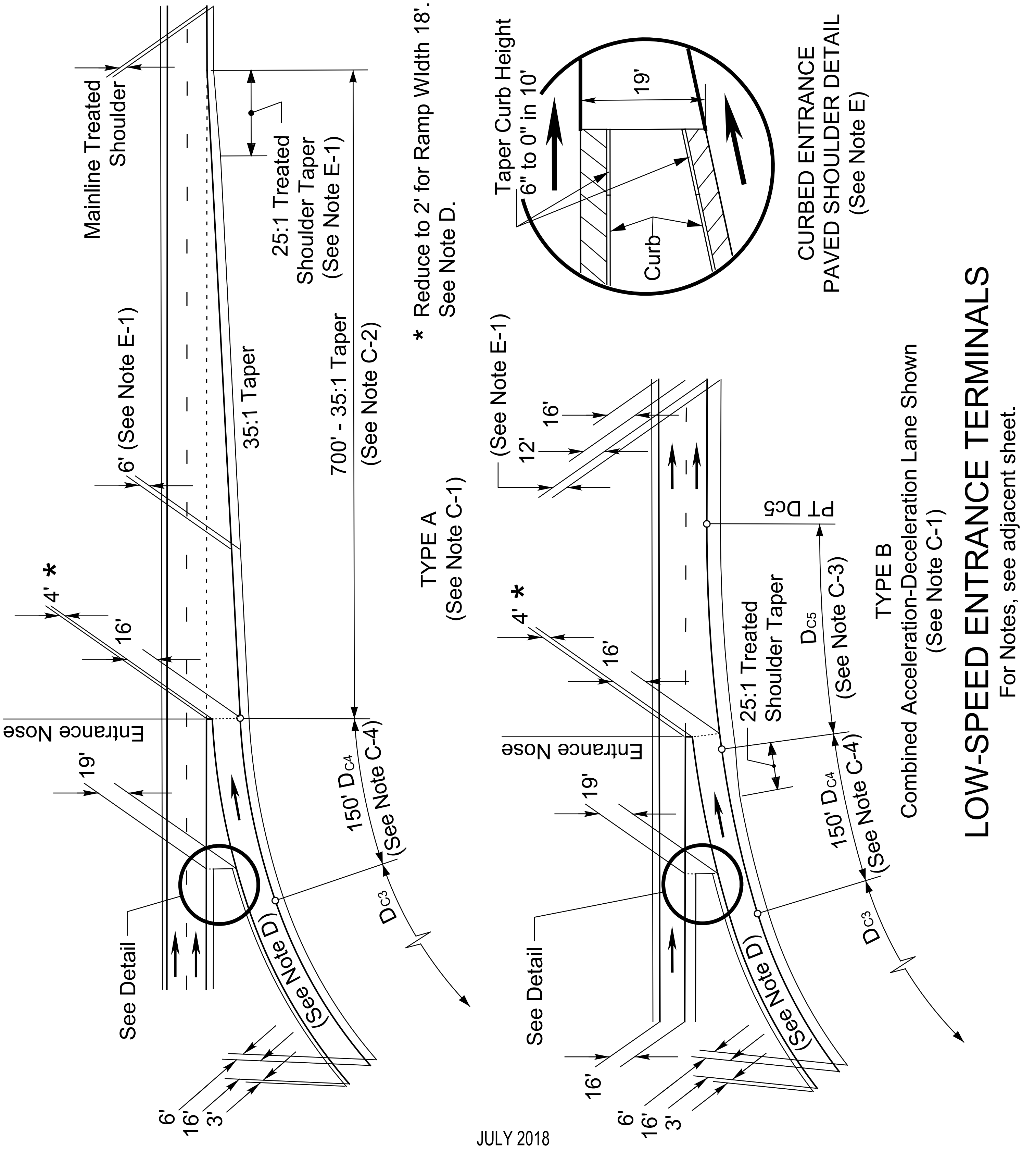
2. When the First Ramp Curve does not exceed 8°, the Exit Curve may be compounded directly with the First Ramp Curve at a PCC 100' beyond the nose. When the First Ramp Curve does exceed 8°, a spiral should be placed between the Exit Curve and the First Ramp Curve and the beginning of the spiral (CS) should be at the nose.

3. Normally Single Lane Ramps will have a Width of 16'. The Width shall be increased to 18' when the Ramp Radius is less than 200'. When an 18' wide ramp is used, the 39' Exit Terminal Width shall be retained and the 23' width reduced by 2'.

LOW-SPEED ENTRANCE TERMINALS

503-4a

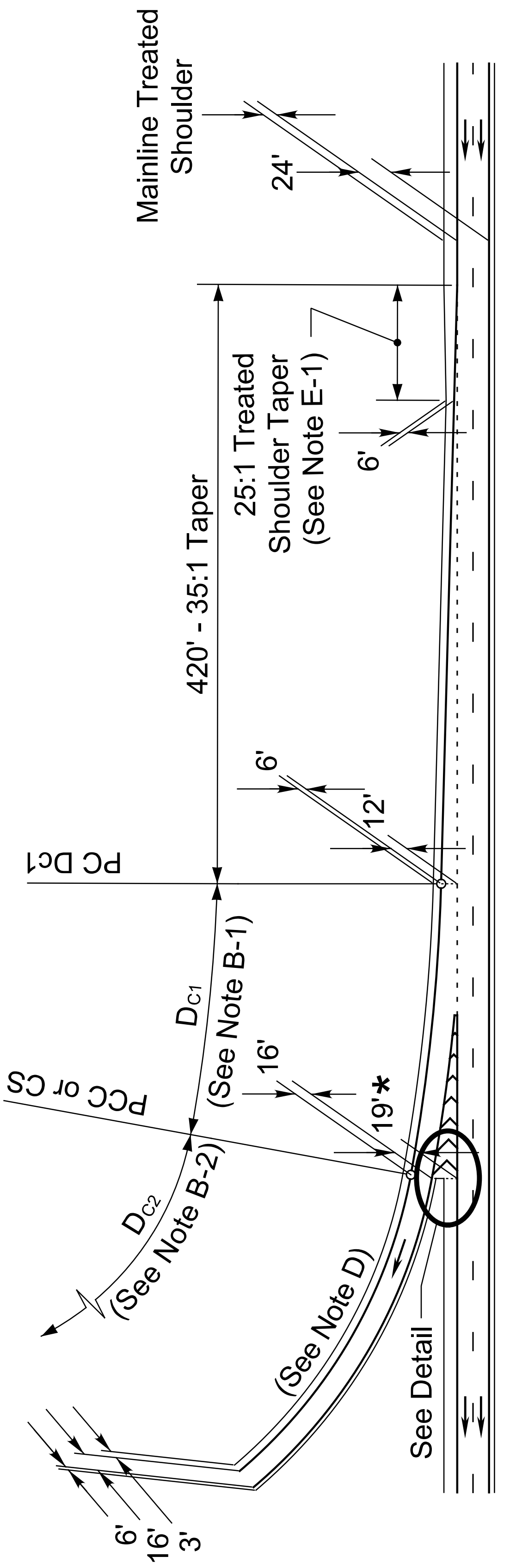
REFERENCE SECTIONS
503.6.2

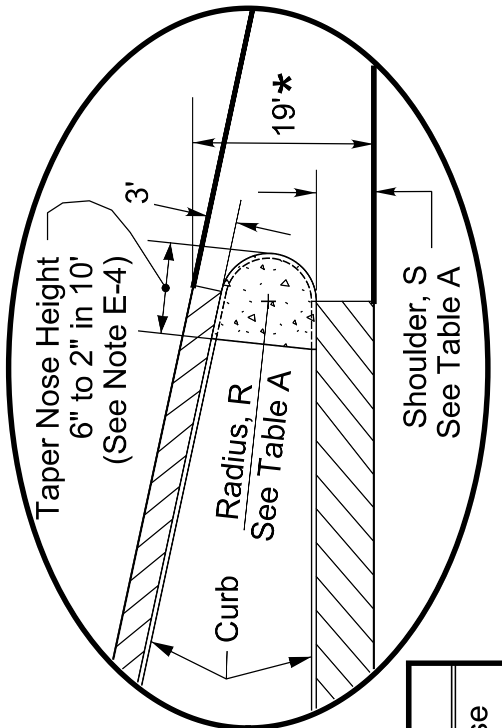


LOW-SPEED EXIT TERMINALS

503-4b

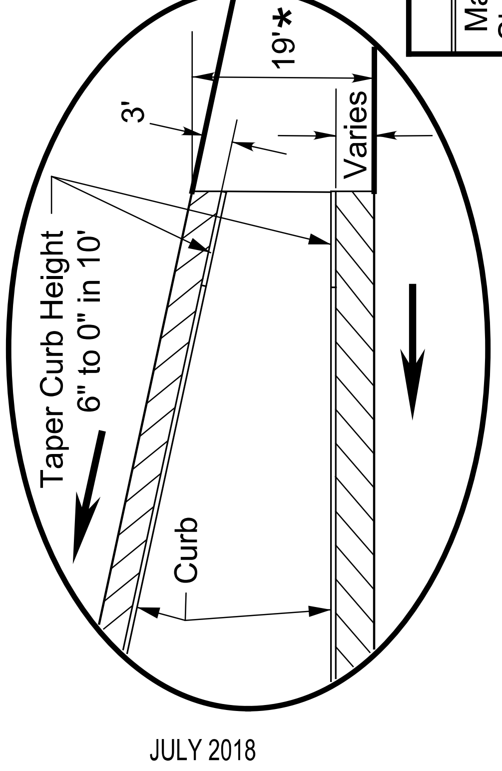
REFERENCE SECTIONS
503.6.3





CURBED PAVED SHOULDER DETAIL OPTION 2 (See Note E)

* Reduce to 17' for Ramp Width 18'. See Note D.



CURBED EXIT PAVED SHOULDER DETAIL OPTION 1 (See Note E)

TABLE A

Mainline Shoulder, S (ft)	Nose Radius, R (ft)
10	3
8	4
6	5
4	6
2	7
1	7.5

LOW-SPEED EXIT TERMINAL

For Notes, see adjacent sheet.

Notes for Low-Speed Entrance and Exit Terminals

Figures 503-4a and 503-4b

A. GENERAL
1. Low-Speed Terminals are intended for used on highways which have little or no access control except through an interchange area. Many of the features of Low-Speed Terminals are applicable to a terminal of one ramp with another ramp in a freeway interchange.

B. EXIT TERMINAL
1. The curve differential between the through roadway and exit curve D_{C1} may vary from a minimum of 4° to the maximum of 8°.
2. Exit Curve D_{C1} may be compounded or spiraled into Ramp Curve D_{C2}. If D_{C2} is greater than 25° then provide a 150 ft. spiral between D_{C1} and D_{C2}.

C. ENTRANCE TERMINAL: TYPE A & TYPE B
1. Type A is preferred and shall normally be used. However, when a ramp enters as an added lane or as a combined acceleration-deceleration lane, Type B may be used if its use would result in a substantial savings in cost (i.e. reduced bridge width).
2. The acceleration lane of Type A shall be a uniform 35:1 taper relative to the through edge of traveled way for either tangent or curving alignment.
3. The curve differential between the through roadway and entrance curve D_{C5} of Type B shall be 4°.
4. The design of the entrance terminal curvature shall be based on the following:

(a) Ramp Curve D_{C3} of 8° or less

When the through roadway tangent or a curve to the right, D_{C4} shall be a 150 ft. long simple curve of a degree such that the differential between it and the through roadway will not exceed 4°. When the through roadway is on a curve to the left, a 150 ft. tangent shall be substituted for D_{C4}.

(b) Ramp Curve D_{C3} greater than 8°

A 150 ft. spiral may be substituted for D_{C4}.

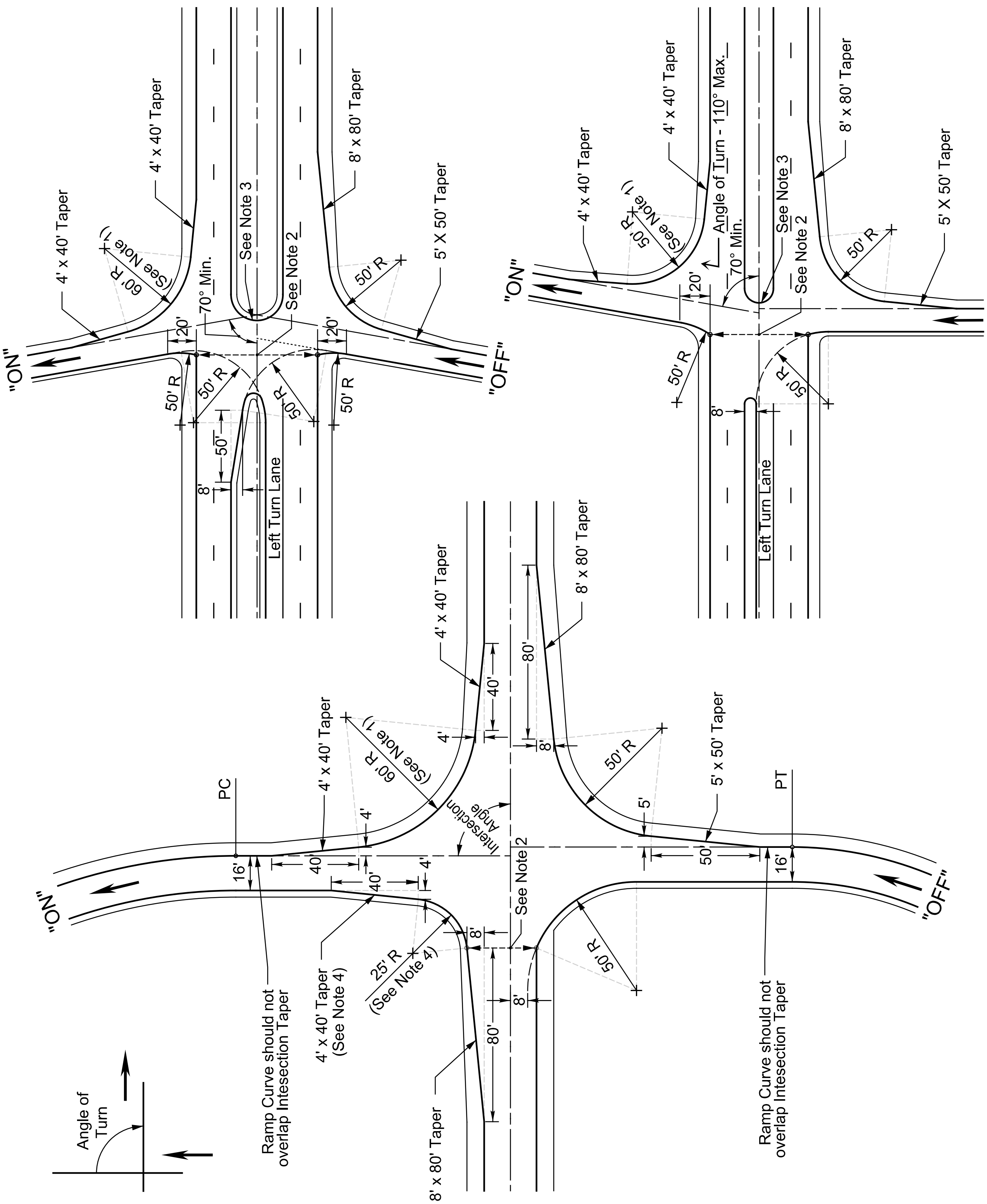
D. RAMP WIDTH
1. Normally single lane ramps will have a width of 16 ft. The width shall be increased to 18 ft. when the ramp radius is less than 200 ft. When an 18 ft. wide ramp is used, the 35 ft. exit and 20 ft. entrance terminal widths shall be retained and the 19 ft. and 4 ft. widths reduced by 2 ft.

E. TREATED SHOULDER
1. The treated shoulder along the speed change lanes shall be as shown on Figure 303-1.
2. If the ramp or through roadway has a curb offset greater than 6 ft. (or 3 ft.), the greater width shall be used at the terminal. Retain the 19 ft. width.
3. The Special Detail drawings shall apply when the through roadway is curbed.

UNCURBED RAMP INTERSECTION

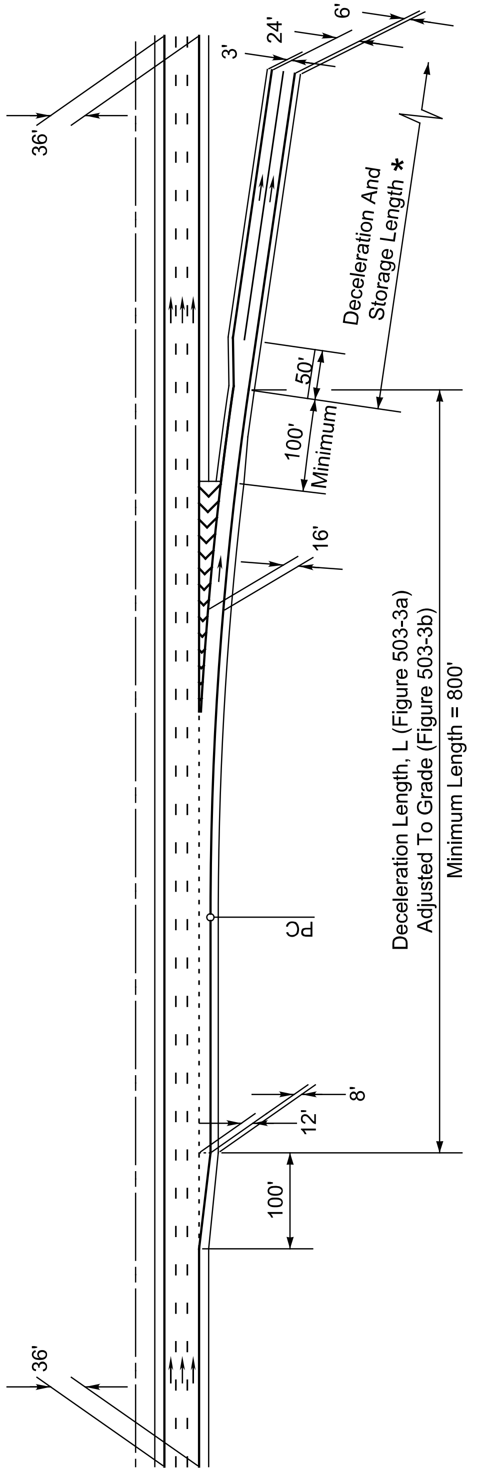
503-5

REFERENCE SECTIONS
503.7

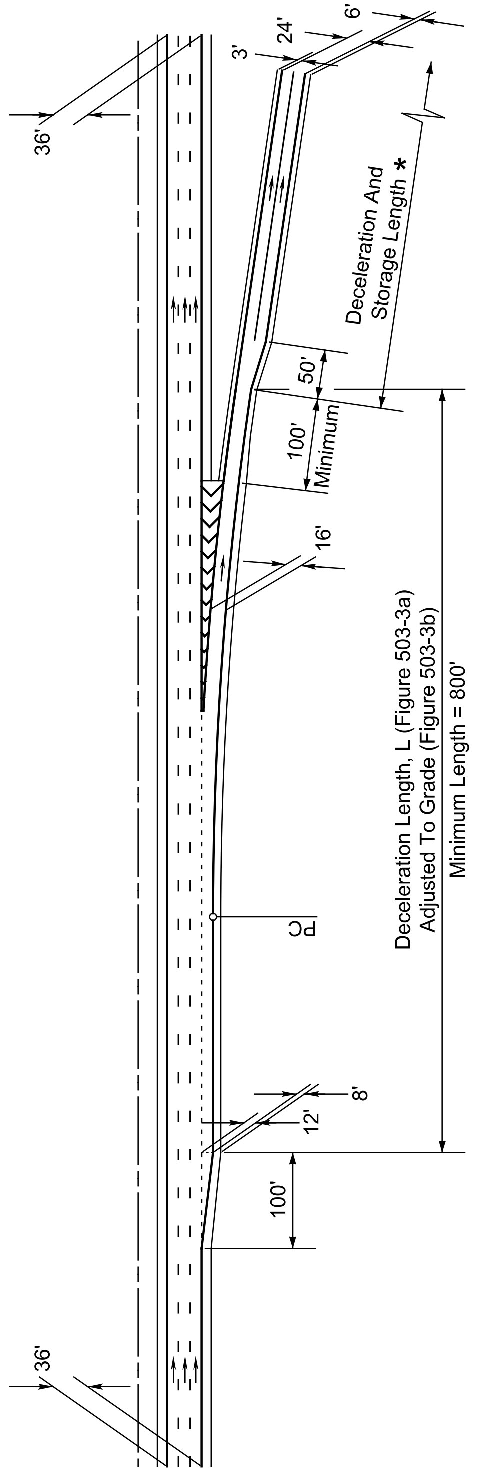


Notes:
1. When the Angle of the Turn into the "ON" Ramp is 100° or greater, reduce the 60' Radius to 50'
2. These points should be at the same centerline station of the crossroad.
3. Where the Ramps intersect a Divided Highway, the Median Opening should be designed to discourage improper turns into the "OFF" Ramps.
4. The 25' Radius and 10:1 Taper may need to be slightly adjusted if the Intersection Angle of the Ramp is less than 90° (verify with turn template software).
5. All of the turn movements at the Intersection are to be analyzed with turning template software and the designated design vehicle.

TRANSITION FROM SINGLE-LANE TO TWO-LANE EXIT RAMP	503-5a
	REFERENCE SECTION 503.7



ALTERNATE A



ALTERNATE B

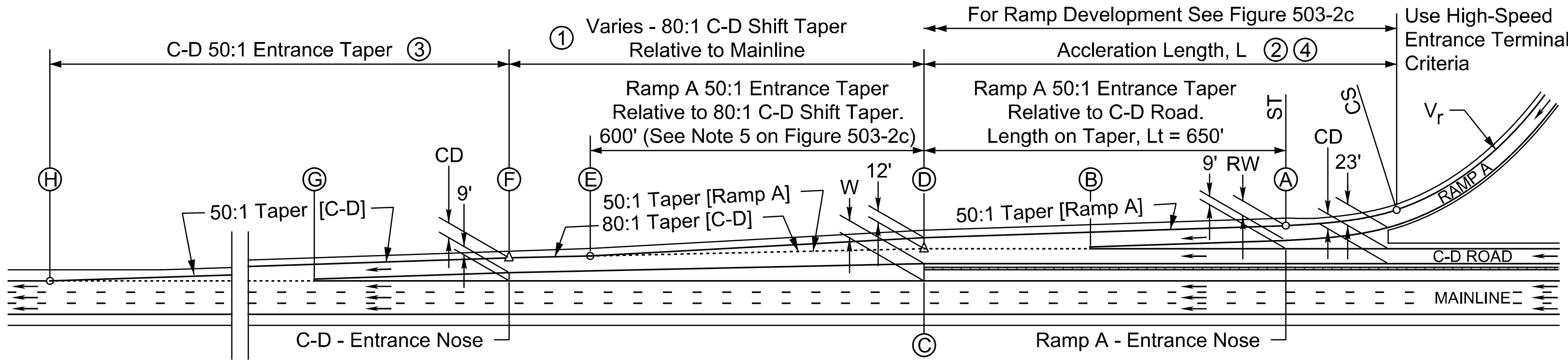
* See Figure 401-9 and 401-10 to compute the length.

Note: The additional lane is usually provided for the minor movement.

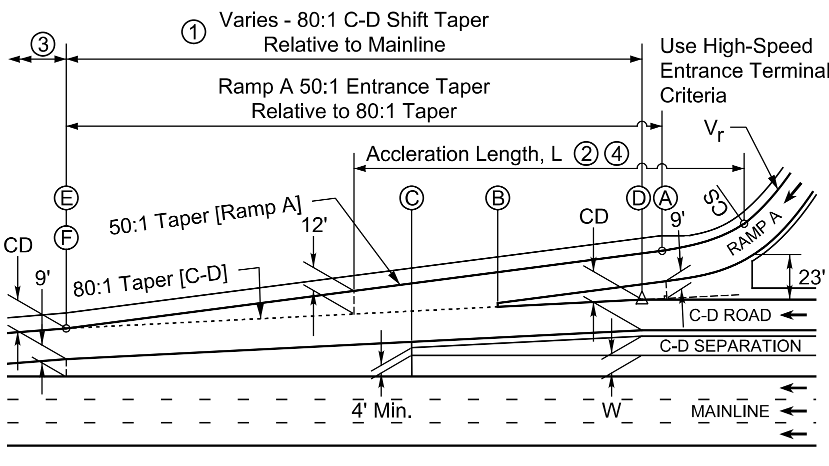
COLLECTOR-DISTRIBUTOR ENTRANCE TERMINALS

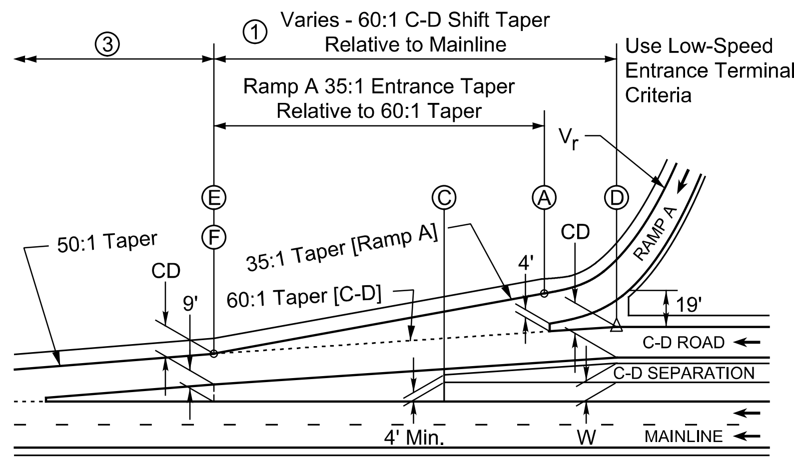
504-1

REFERENCE SECTIONS
504.3



HIGH SPEED (MAINLINE DESIGN SPEED ≥ 50 MPH)
BARRIER C-D SEPARATION





HIGH SPEED (MAINLINE DESIGN SPEED ≥ 50 MPH)
WIDE C-D SEPARATION (NO BARRIER)

LOW SPEED (MAINLINE DESIGN SPEED < 50 MPH)
WIDE C-D SEPARATION (NO BARRIER)

POINT LOCATION INFORMATION

- Ⓐ Begin Ramp A Entrance Taper
- Ⓑ Ramp A Theoretical Gore Point
- Ⓒ End C-D Separation
- Ⓓ Begin C-D Shift Taper
- Ⓔ End Ramp A Entrance Taper
- Ⓕ End C-D Shift Taper
- Ⓖ C-D Theoretical Gore Point
- Ⓗ End C-D Entrance Taper

NOTES:

W = Width of C-D Separation.

CD = C-D Road Width. 16' for Single-Lane. A minimum of two lanes when between interchanges.

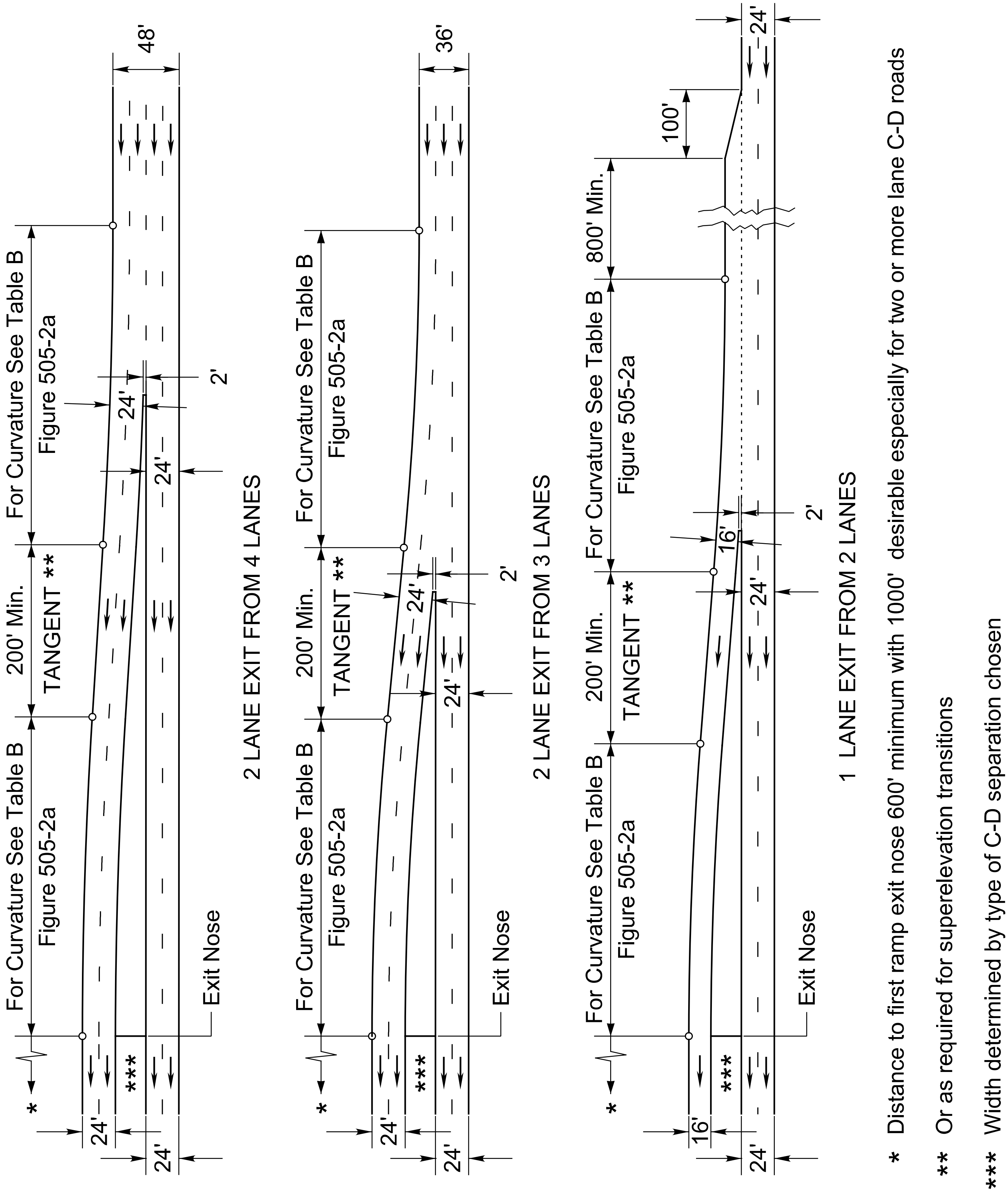
RW = Ramp Width. 16' for Single-Lane, For 2 Or More Lanes Use Multi-Lane Entrance Terminal Criteria.

V_r = Last Ramp Curve Design Speed, (mph)

1. As the Width of C-D Separation increases, the Length of the C-D 80:1 Shift Taper will vary as well. Therefore, the distance between points D and F will vary.
2. It is imperative to provide the necessary adjusted Acceleration Length, L for the Ramp no matter where points B, C or D may be located in relation to the Mainline, C-D and Ramp.
3. Use Single-Lane Entrance Terminal Criteria, For 2 Or More Lanes Use Multi-Lane Entrance Terminal Criteria.
4. See Figures 503 for Minimum Acceleration Lengths, L. If Additional Parallel Acceleration Length, Lp is necessary to meet C-D Design speed, provide Lp as shown in Figure 503-2c.

COLLECTOR-DISTRIBUTOR EXIT TERMINALS

504-2
REFERENCE SECTIONS 504.3



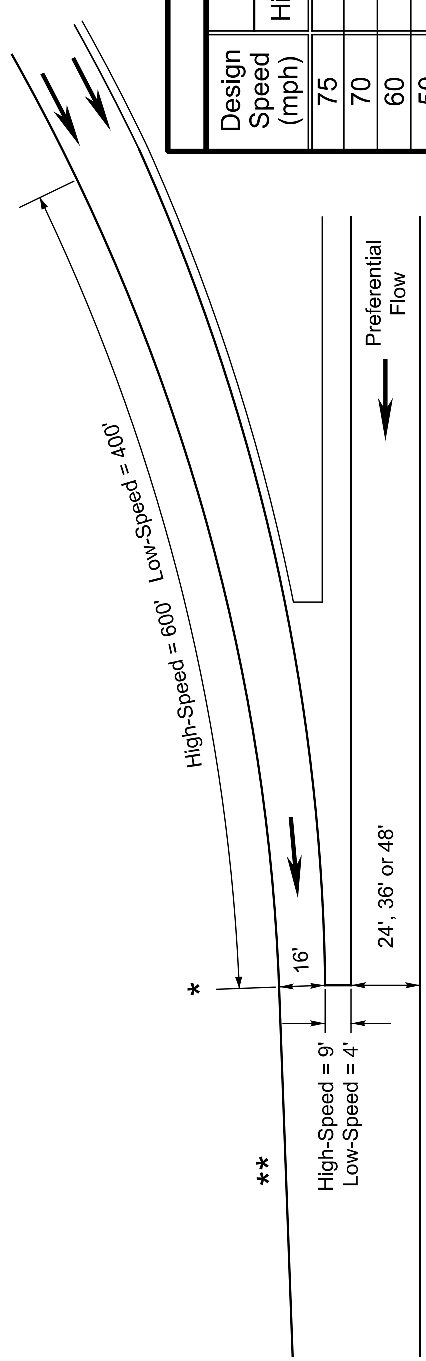
MULTI-LANE ENTRANCE RAMPS AND CONVERGING ROADWAYS

505-1a

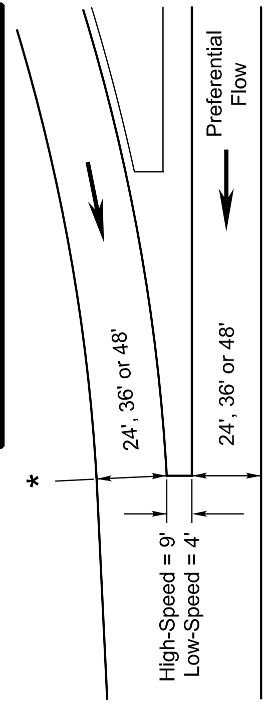
REFERENCE SECTION 505.1

TABLE A

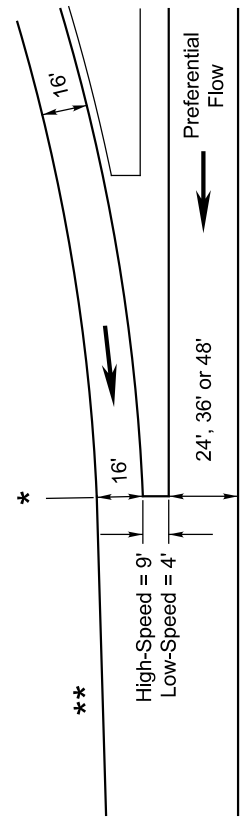
Design Speed (mph)	Taper Rate: High-Speed	Taper Rate: Low-Speed
75	50:1	--
70	50:1	--
60	50:1	--
50	50:1	--
45	--	35:1
40	--	35:1
30	--	35:1



TWO-LANE ROADWAY REDUCED TO SINGLE LANE PRIOR TO ENTRANCE NOSE



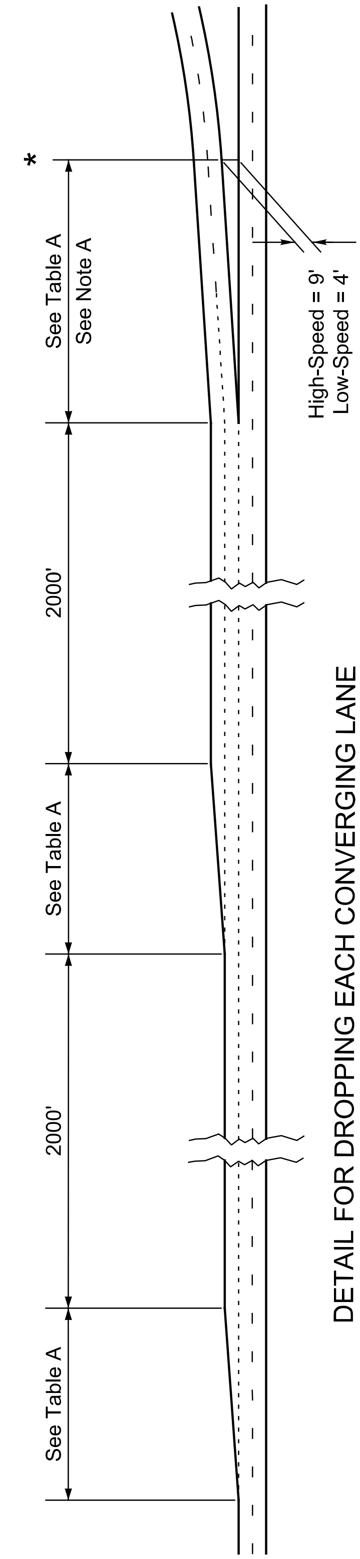
MULTI-LANE CONVERGING WITH MULTI-LANE



SINGLE LANE CONVERGING WITH MULTI-LANE

* Note: the number of lanes leaving the entrance nose must be equal to the total number of lanes (converging plus mainline) approaching the entrance nose.

** See Figure 503-2c or 503-4a for terminal details.



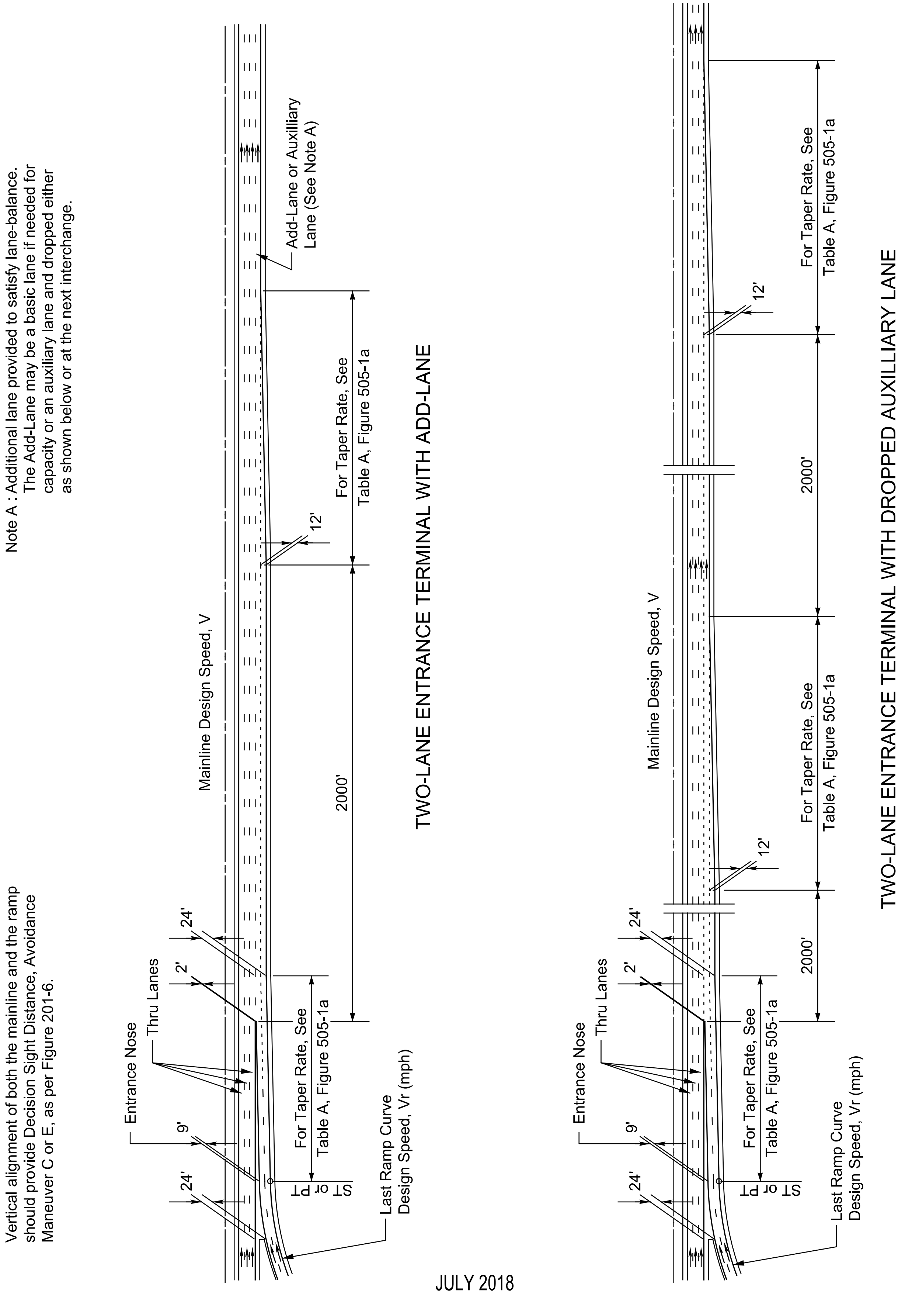
Note A:
Vertical alignment of both the mainline and the ramp should provide Decision Sight Distance, Avoidance Maneuver C or E, as per Figure 201-6.

DETAIL FOR DROPPING EACH CONVERGING LANE

HIGH-SPEED TWO-LANE ENTRANCE TERMINALS

505-1b

REFERENCE SECTIONS
505.1



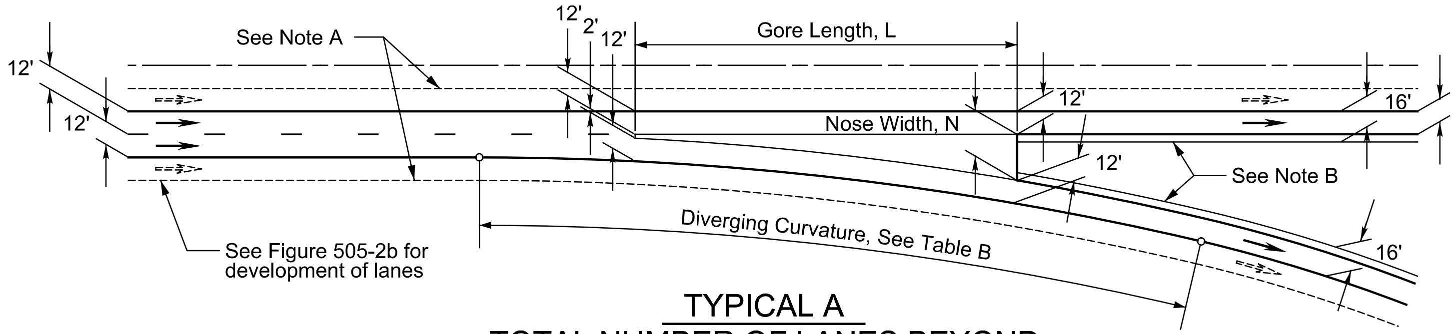
TWO-LANE ENTRANCE TERMINAL WITH ADD-LANE

TWO-LANE ENTRANCE TERMINAL WITH DROPPED AUXILLIARY LANE

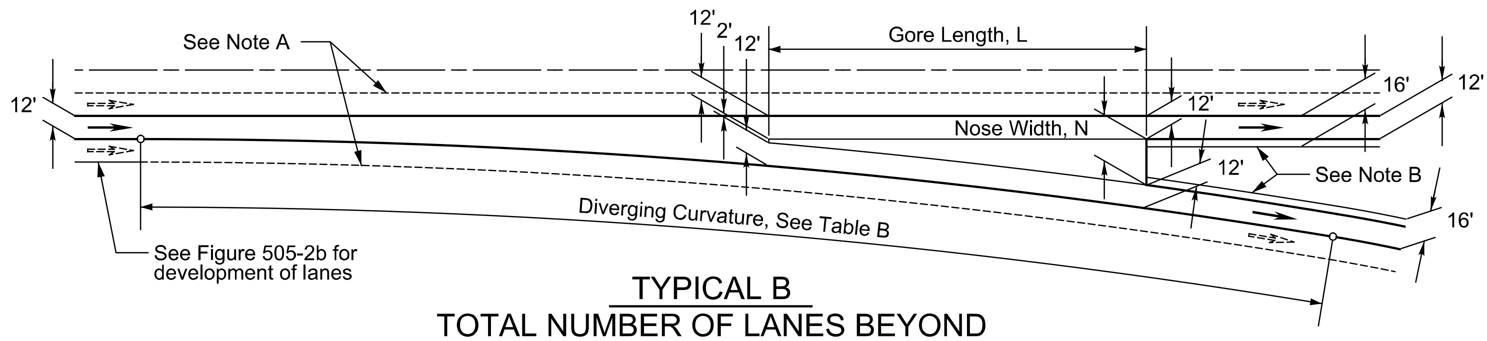
MULTI-LANE EXIT RAMPS AND DIVERGING ROADWAYS

505-2a

REFERENCE SECTION 505.2



TYPICAL A
TOTAL NUMBER OF LANES BEYOND THE NOSE EQUALS APPROACH LANES



TYPICAL B
TOTAL NUMBER OF LANES BEYOND THE NOSE EQUALS APPROACH LANES PLUS ONE

TABLE A

Design Speed, "V" (mph)	"L" Gore Length (ft.)			"N" * Nose Width (ft.)		
	High-Speed		Low Speed	High-Speed		Low Speed
	Rural	Urban		Rural	Urban	
75	525	430		32	24	
70	450	370		32	24	
65	400	340		32	24	
60	370	300		32	24	
55	350	280		32	24	
50	320	260		32	24	
45			180			24
40			160			24
35			140			24
30			120			24

* "N" dimension includes 4' of a 16' lane

TABLE B
RECOMMENDED DIVERGING CURVATURE (See Sec. 505.2.4)

Design Speed of Approaching Roadway "V" (mph)	DIVERGING CURVATURE Total Lanes Beyond Diverging Nose Equals					
	Approach Lanes			Approach Lanes +1		
	High-Speed		Low Speed △	High-Speed		Low Speed △
	Rural	Urban		Rural	Urban	
75	0°- 45'	0°- 45'		0°- 25'	0°- 25'	
70	1°- 00'	1°- 00'		0°- 30'	0°- 30'	
65	1°- 15'	1°- 15'		0°- 35'	0°- 35'	
60	1°- 30'	1°- 30'		0°- 40'	0°- 40'	
55	1°- 45'	1°- 45'		0°- 45'	0°- 45'	
50	2°- 00'	2°- 00'		1°- 00'	1°- 00'	
45			4°- 15'			1°- 45'
40			5°- 30'			2°- 15'
35			7°- 00'			2°- 45'
30			10°- 00'			3°- 30'

△ Based on a Design Speed equal to ("V" - 10 mph)

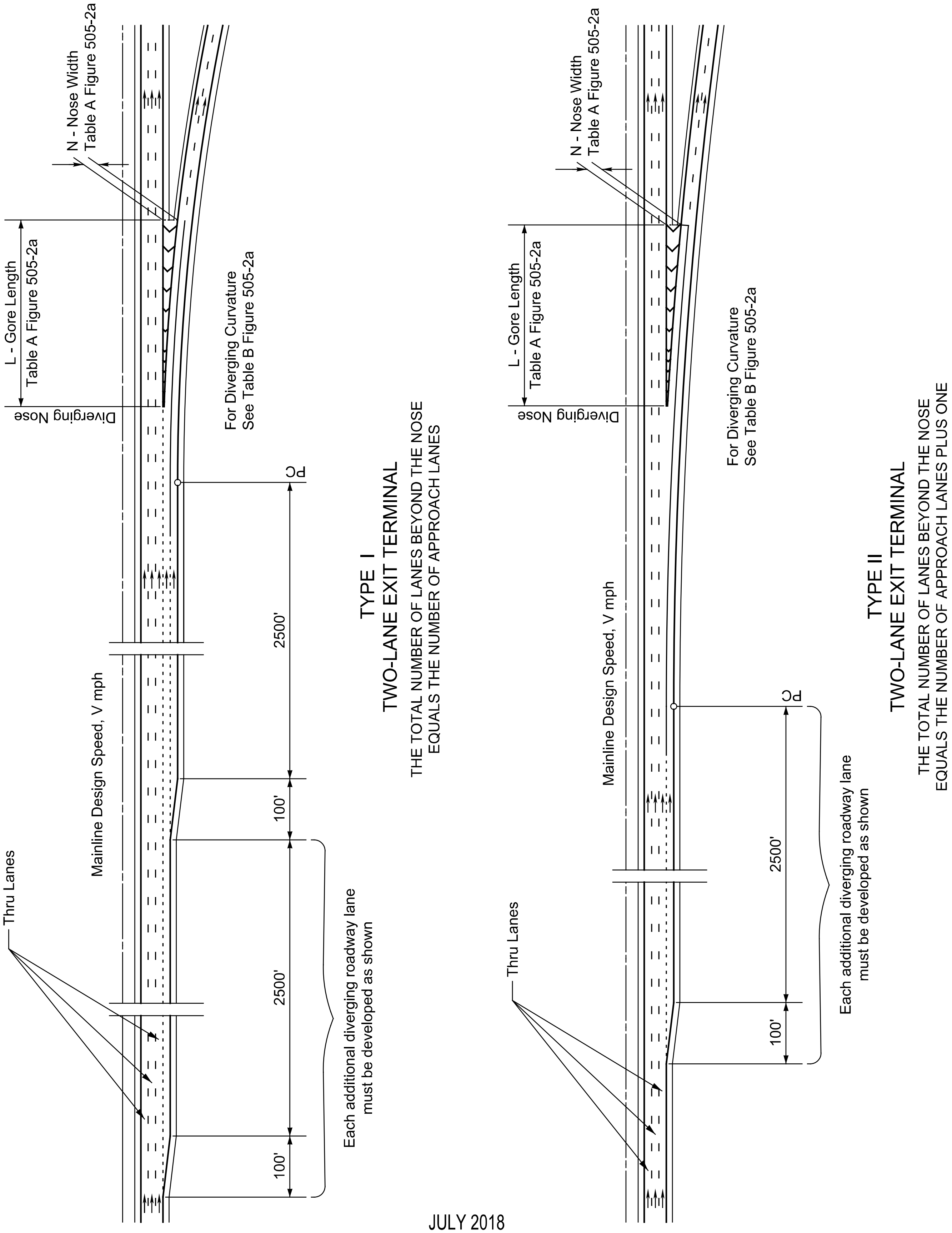
Note A: Any lane combination can be designed from Table A and Typicals A and B by adding one or more lanes to one or both sides of Typical A or adding one or more lanes to both sides of Typical B.

Note B: When a 16 foot lane width is used after the diverging nose, the nose width "N" includes 4 feet of the 16 foot lane width. For two 16 foot lanes, "N" includes 4 feet of each lane.

HIGH-SPEED TWO-LANE EXIT TERMINALS FOR SYSTEM INTERCHANGES

505-2b

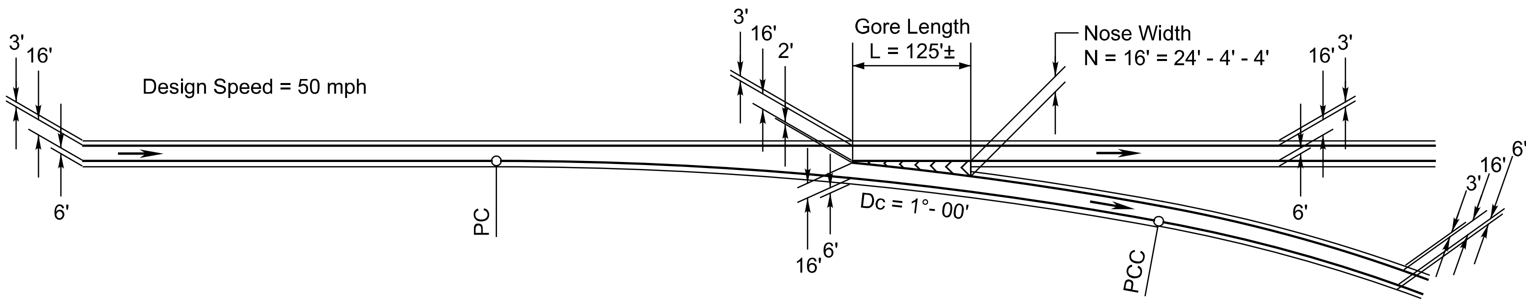
REFERENCE SECTION 505.2



EXAMPLES OF DIVERGING ROADWAYS

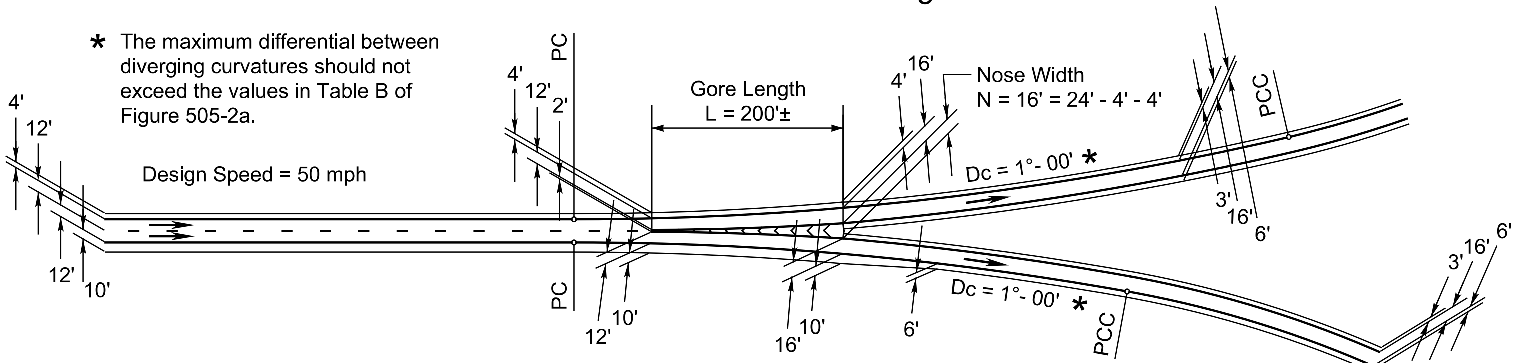
505-2c

REFERENCE SECTIONS 505.2

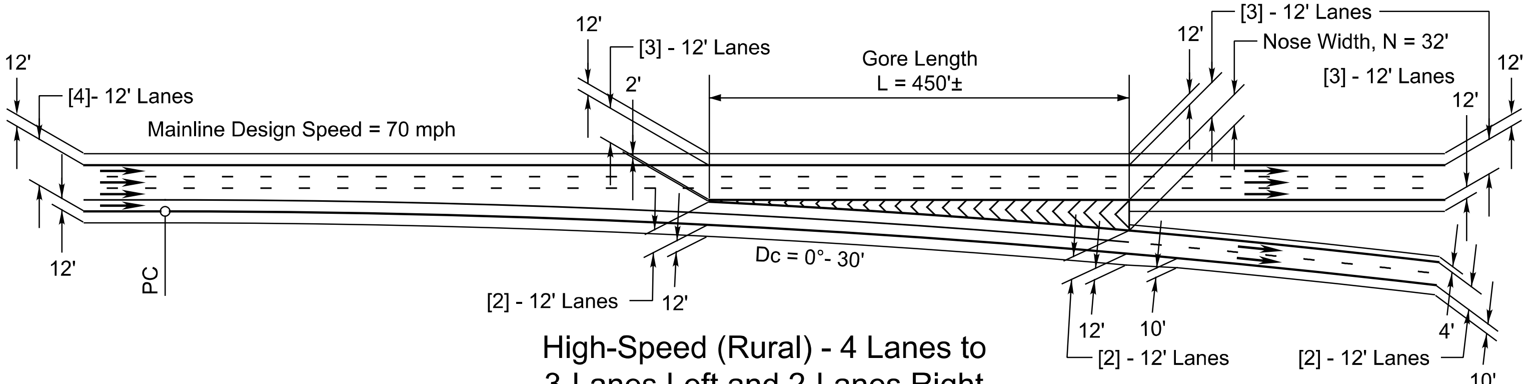


High-Speed (Urban) - 1 Lane Ramp to
1 Lane Left and 1 Lane Right

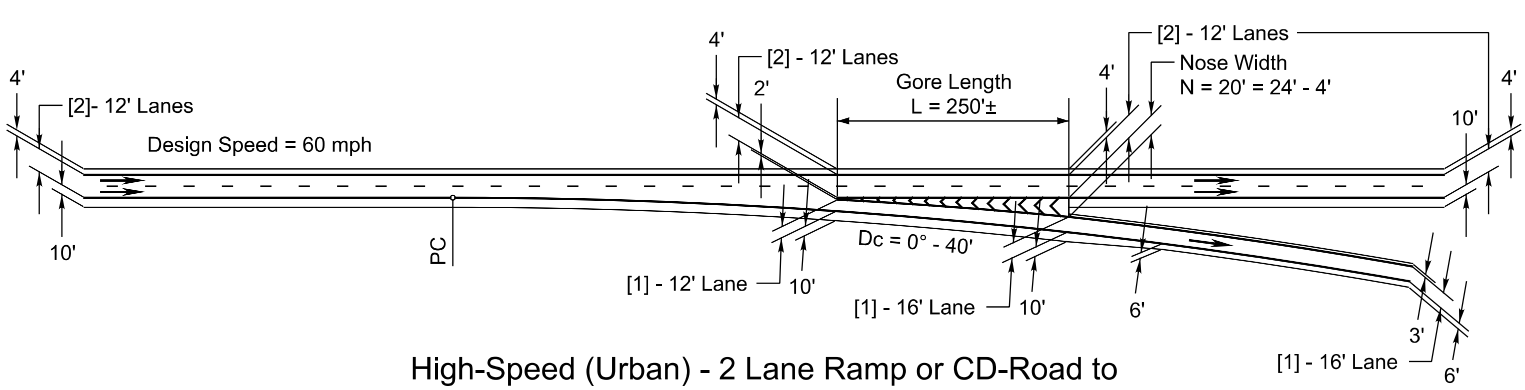
* The maximum differential between diverging curvatures should not exceed the values in Table B of Figure 505-2a.



High-Speed (Urban) - 2 Lane Ramp to
1 Lane Left and 1 Lane Right



High-Speed (Rural) - 4 Lanes to
3-Lanes Left and 2-Lanes Right

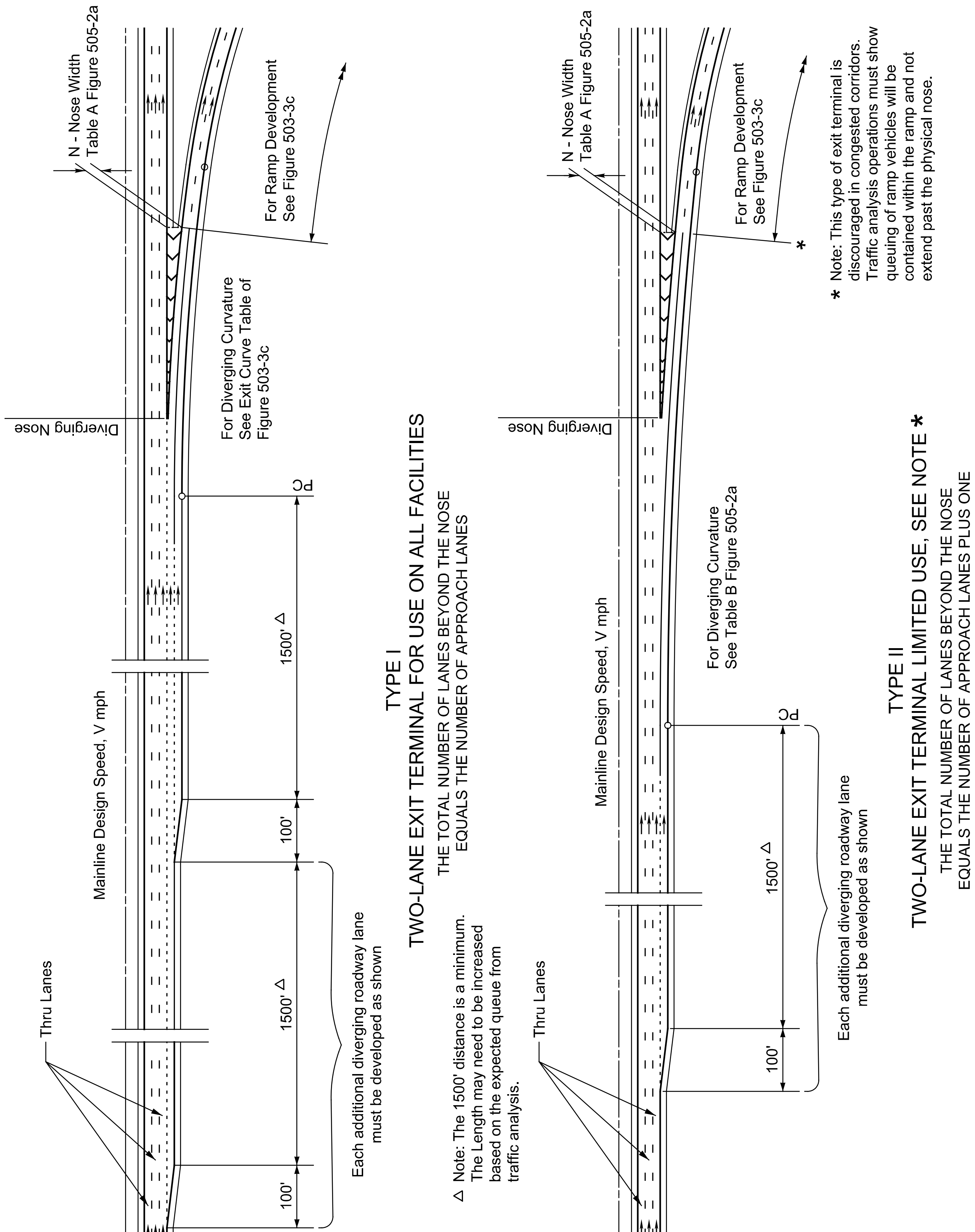


High-Speed (Urban) - 2 Lane Ramp or CD-Road to
2 Lanes Left and 1 Lane Right

HIGH-SPEED TWO-LANE EXIT TERMINALS FOR SERVICE INTERCHANGES

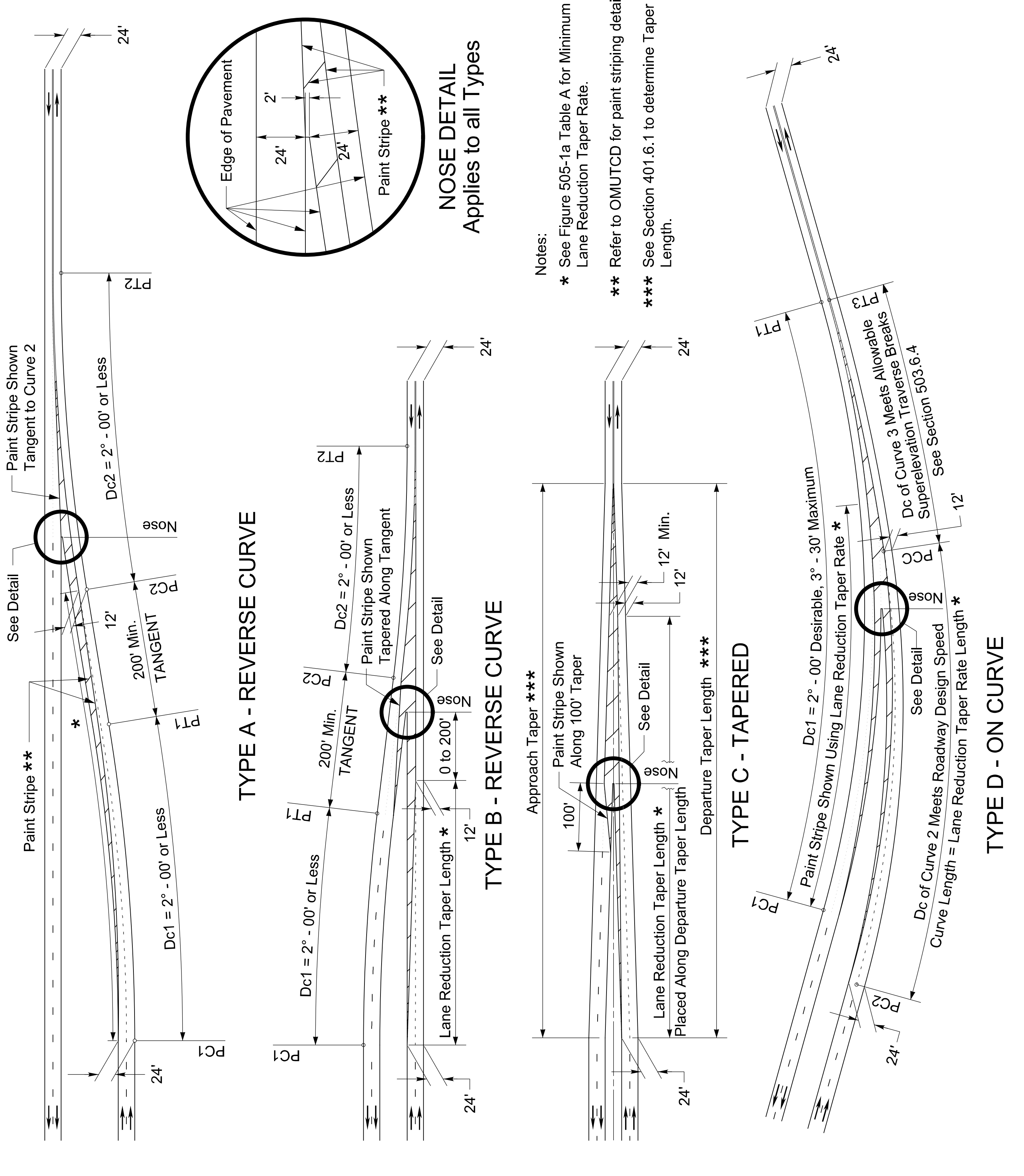
505-2d

REFERENCE SECTION 505.2



TRANSITIONS
FOUR LANE DIVIDED ROADWAY TO TWO LANE ROADWAY

505-3
REFERENCE SECTION 505.3



EXAMPLE PROBLEM	
Ex. 550-1	Constrained Traffic Volumes

Calculation Sheet for Determination of Constrained Traffic Volumes Example

Example: Determine whether construction of an additional eastbound left turn lane from arterial to interstate would degrade the freeway operation.

Step 1: How does the merge downstream of the on-ramp intersection operate? For this example, the merge for the No-Build and Build conditions operates at a LOS F for both, so there is potential degradation. Step 2: Prepare constrained analyses based on results of HCS intersection analyses.

See next page for traffic volumes and lane assignments. (Note: An improvement is deemed to degrade the freeway operation if it increases traffic on freeway mainline by greater than 2.00% when the freeway is operating at LOS F in the No-Build condition.)

NO BUILD CONDITION

•Full demand eastbound left turn DHV onto freeway ramp = 490 vph
•v/c is 1.329 (from HCS analysis), > 1.0 so constrained
•Capacity Constrained volume = vph/(v/c) = 490/1.329 = 369 vph

•Full demand westbound right turn DHV onto freeway ramp = 390 vph
•v/c is 0.613 (from HCS analysis), < 1.0 so not constrained

•Total volume entering freeway ramp = constrained EBL +WBR = 369 + 390
= **759 vph**

BUILD CONDITION

•Full demand eastbound left turn DHV onto freeway ramp = 490 vph
•v/c is 1.136 (from HCS analysis), > 1.0 so constrained
•Capacity Constrained volume = vph/(v/c) = 490/1.136 = 431 vph

•Full demand westbound right turn DHV onto freeway ramp = 390 vph
•v/c is 0.528 (from HCS analysis), < 1.0 so not constrained

•Total volume entering freeway ramp = constrained EBL +WBR = 431 + 390
= **821 vph**

COMPARISON

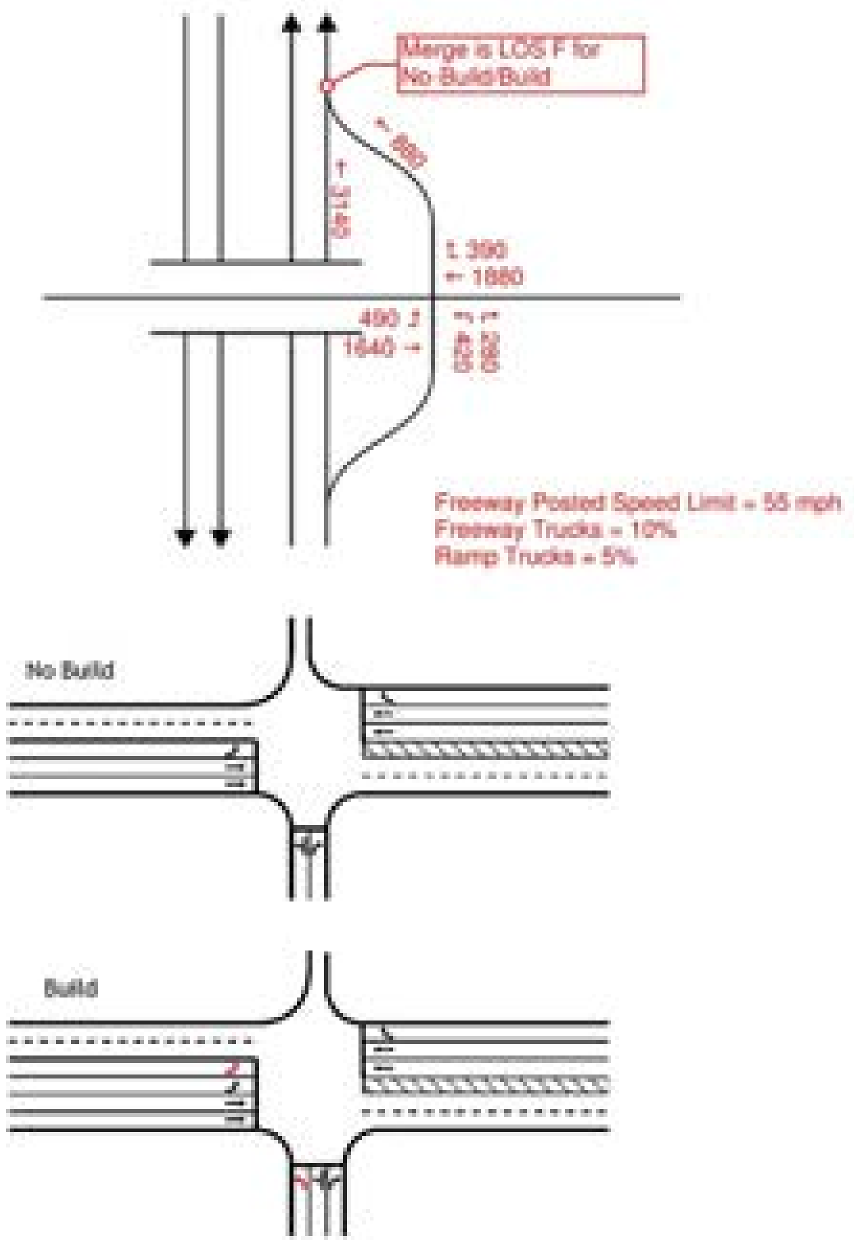
•821-759 = **62** additional vehicles entering the freeway with the improvement is constructed.

•% traffic added to freeway mainline due to improvements = additional vehicles entering freeway after improvements / (trips on mainline + No Build constrained vehicles entering from ramp)

62/(3140+759) = 1.59 % more traffic added to freeway due to improvement

1.59% < 2.00 % Therefore, improvement does not degrade freeway operation

EXAMPLE PROBLEM	
Ex. 550-1	Constrained Traffic Volumes



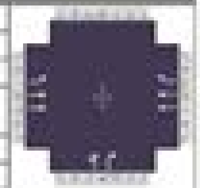
Eastbound left turn lane added
Northbound left turn lane added

EXAMPLE PROBLEM

Ex. 550-1	Constrained Traffic Volumes

HCS7 Signalized Intersection Results Summary

General Information				Intersection Information	
Agency	ODOT			Duration, h	0.25
Analyst	ORE	Analysis Date	May 23, 2018	Area Type	Other
Jurisdiction		Time Period	PM Peak No-Build	PHF	0.92
Urban Street		Analysis Year	Design Year	Analysis Period	1> 17:00
Intersection	Crossroad & Ramp	File Name	PM Peak, No-Build, Intersection.xus		
Project Description	Example Problem 550-1				



Demand Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (v), veh/h	490	1640			1880	390	420	0	280			

Signal Information			
Cycle, s	90.0	Reference Phase	2
Offset, s	0	Reference Point	End
Uncoordinated	Yes	Simult. Gap E/W	On
Force Mode	Fixed	Simult. Gap N/S	On

Green	16.6	40.2	18.2	0.0	0.0	0.0
Yellow	4.0	4.0	4.0	0.0	0.0	0.0
Red	1.0	1.0	1.0	0.0	0.0	0.0

Timer Results	EBL	EBT	WBL	WBT	NBL	NBT	SBL	SBT
Assigned Phase	5	2		6		8		
Case Number	1.0	4.0		7.3		11.0		
Phase Duration, s	21.6	66.8		45.2		23.2		
Change Period, (Y+R c), s	5.0	5.0		5.0		5.0		
Max Allow Headway (MAH), s	3.1	3.1		3.1		3.2		
Queue Clearance Time (g s), s	18.6	31.7		42.2		20.2		
Green Extension Time (g e), s	0.0	7.4		0.0		0.0		
Phase Call Probability	1.00	1.00		1.00		1.00		
Max Out Probability	1.00	0.69		1.00		1.00		

Movement Group Results	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Assigned Movement	5	2			6	16	3	8	18			
Adjusted Flow Rate (v), veh/h	533	1783			2043	424		457	304			
Adjusted Saturation Flow Rate (s), veh/h/ln	1739	1736			1736	1547		1739	1547			
Queue Service Time (g s), s	16.6	29.7			40.2	18.8		18.2	17.6			
Cycle Queue Clearance Time (g c), s	16.6	29.7			40.2	18.8		18.2	17.6			
Green Ratio (g/C)	0.65	0.69			0.45	0.45		0.20	0.20			
Capacity (c), veh/h	401	2387			1553	691		352	313			
Volume-to-Capacity Ratio (X)	1.329	0.747			1.316	0.613		1.298	0.973			
Back of Queue (Q), ft/ln (95 th percentile)	1043.6	363.7			1801.6	277.8		869.5	396.9			
Back of Queue (Q), veh/ln (95 th percentile)	40.1	14.0			69.3	10.7		34.2	15.3			
Queue Storage Ratio (RQ) (95 th percentile)	0.00	0.00			0.00	0.00		0.00	0.00			
Uniform Delay (d 1), s/veh	28.8	9.1			24.9	19.0		35.9	35.7			
Incremental Delay (d 2), s/veh	164.4	1.2			146.9	1.2		153.7	43.1			
Initial Queue Delay (d 3), s/veh	0.0	0.0			0.0	0.0		0.0	0.0			
Control Delay (d), s/veh	193.2	10.2			171.8	20.2		189.6	78.8			
Level of Service (LOS)	F	B			F	C		F	E			
Approach Delay, s/veh / LOS	52.3	D		145.7	F		145.2	F		0.0		
Intersection Delay, s/veh / LOS	106.7						F					

Multimodal Results	EB		WB		NB		SB	
Pedestrian LOS Score / LOS	1.63	B	1.39	A	2.31	B	2.31	B
Bicycle LOS Score / LOS	2.40	B	2.52	C	1.74	B		

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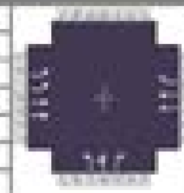
EXAMPLE PROBLEM

Ex. 550-1 Constrained Traffic Volumes

HCS7 Signalized Intersection Results Summary

General Information				Intersection Information	
Agency	ODOT			Duration, h	0.25
Analyst	ORE	Analysis Date	May 23, 2018	Area Type	Other
Jurisdiction		Time Period	PM Peak Build	PHF	0.92
Urban Street		Analysis Year	Design Year	Analysis Period	1> 17:00
Intersection	Crossroad & Ramp	File Name	PM Peak, Build, Intersection.xus		
Project Description	Example Problem 550-1				



Demand Information	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Demand (v), veh/h	490	1640			1880	390	420	1	280			

Signal Information			
Cycle, s	90.0	Reference Phase	2
Offset, s	0	Reference Point	End
Uncoordinated	Yes	Simult. Gap E/W	On
Force Mode	Fixed	Simult. Gap N/S	On

Green	12.5	46.7	15.8	0.0	0.0	0.0
Yellow	4.0	4.0	4.0	0.0	0.0	0.0
Red	1.0	1.0	1.0	0.0	0.0	0.0

Timer Results	EBL	EBT	WBL	WBT	NBL	NBT	SBL	SBT
Assigned Phase	5	2		6		8		
Case Number	2.0	4.0		7.3		9.0		
Phase Duration, s	17.5	69.2		51.7		20.8		
Change Period, (Y+R c), s	5.0	5.0		5.0		5.0		
Max Allow Headway (MAH), s	3.1	3.1		3.1		3.2		
Queue Clearance Time (g s), s	14.5	29.2		48.7		17.8		
Green Extension Time (g e), s	0.0	12.5		0.0		0.0		
Phase Call Probability	1.00	1.00		1.00		1.00		
Max Out Probability	1.00	0.70		1.00		1.00		

Movement Group Results	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Assigned Movement	5	2			6	16	3	8	18			
Adjusted Flow Rate (v), veh/h	533	1783			2043	424	228	229	304			
Adjusted Saturation Flow Rate (s), veh/h/ln	1689	1738			1738	1547	1810	1810	1547			
Queue Service Time (g s), s	12.5	27.2			46.7	16.3	10.7	10.8	15.8			
Cycle Queue Clearance Time (g c), s	12.5	27.2			46.7	16.3	10.7	10.8	15.8			
Green Ratio (g/C)	0.14	0.71			0.52	0.52	0.18	0.18	0.18			
Capacity (c), veh/h	469	2480			1804	803	318	318	272			
Volume-to-Capacity Ratio (X)	1.136	0.719			1.133	0.528	0.719	0.722	1.120			
Back of Queue (Q), ft/ln (95 th percentile)	434	320.5			1232.5	235.6	222.2	232.5	506.7			
Back of Queue (Q), veh/ln (95 th percentile)	16.7	12.3			47.4	9.1	8.9	8.9	19.5			
Queue Storage Ratio (RQ) (95 th percentile)	0.00	0.00			0.00	0.00	0.00	0.00	0.00			
Uniform Delay (d 1), s/veh	38.8	7.6			21.7	14.3	35.0	35.0	37.1			
Incremental Delay (d 2), s/veh	84.3	0.9			67.3	0.3	6.6	6.8	90.9			
Initial Queue Delay (d 3), s/veh	0.0	0.0			0.0	0.0	0.0	0.0	0.0			
Control Delay (d), s/veh	123.0	8.5			89.0	14.7	41.6	41.8	128.0			
Level of Service (LOS)	F	A			F	B	D	D	F			
Approach Delay, s/veh / LOS	34.8	C		76.2	E		78.2	E		0.0		
Intersection Delay, s/veh / LOS	58.9						E					

Multimodal Results	EB		WB		NB		SB	
Pedestrian LOS Score / LOS	1.65	B	1.67	B	2.31	B	2.47	B
Bicycle LOS Score / LOS	2.40	B	2.52	C	1.74	B		

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600 Roadside Design

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600 Roadside Design

600.1 Introduction

This chapter discusses concepts related to roadside safety features which are intended to reduce occurrences of run-off-the-road crashes and reduce the severity of impact when such an incident does occur. The AASHTO Roadside Design Guide contains additional information on roadside design.

Safety devices are themselves fixed objects, and while they may decrease crash severity, they may also increase the total number of impacts. The potential for impacts can be reduced by placing the safety device as close to the hazard being shielded and as far from the traveled lanes as permitted by the following standards. Roadside safety devices are hazards and must result in a less severe crash than the hazard being shielded.

600.2 Clear Zone

Clear Zone The unobstructed, traversable area provided beyond the edge of the through traveled way for the recovery of errant vehicles. The clear zone includes shoulders, bike lanes, or auxiliary lanes, except those auxiliary lanes that function like through lanes. Ideally, there should be no obstructions within the clear zone; however, if an obstruction cannot be removed, then engineering judgment must be used to determine how to treat it.

When a warranting feature cannot be removed, the clear zone distances given in ***Figure 600-1***, may be used as minimum values. These widths are based on design speed, traffic volume, and the combination of foreslopes and backslopes on the typical cross section for the roadway. These minimum values should not erroneously be interpreted as permitting or encouraging the construction of potential hazards immediately outside the clear zone at what may be deemed a “safe” distance from the edge of the through traveled lanes.

Rather, the clear zone width should be increased if a site investigation indicates that doing so would significantly lessen the potential for accidents. For example, if an obstruction exists just outside the required clear zone in an otherwise obstruction-free area, it should be considered for removal or protection.

For curves with a history of run-off-the-road crashes and a Degree of Curve of 2°00' or greater, ***Figure 600-1*** also provides a table of adjustment factors based on design speed that should be used to extend the clear zone. In these cases, the designer should ensure that the roadway has proper superelevation before evaluating the curve’s effect on the clear zone.

The preferred order of corrective treatment for fixed objects and non-traversable hazards located within the clear zone is as follows:

1. Remove the obstacle.
2. Redesign the obstacle so that it can be safely traversed.
3. Relocate the obstacle to a point where it is less likely to be struck.
4. Reduce the impact severity by using an appropriate breakaway device.

5. Shield the obstacle with a longitudinal traffic barrier designed for redirection or use a crash cushion.

6. Delineate the obstacle if the above alternatives are not appropriate.

The overall intent of roadside design is to strive for a forgiving highway. Designing a project exclusively to meet minimum clear zone values may result in a roadside that is not as safe as it could be. On the other hand, the cost of clearing some roadsides may greatly exceed the associated benefits to the traveling public. The optimum solution lies in the judicious application of engineering judgment coupled with a sincere desire to produce safe roadways.

600.2.1 Parallel Embankment Slopes & Ditches

Embankment slopes parallel to the roadway fall into the following categories:

Recoverable Slopes – Slopes on which encroaching motorists can generally stop their vehicles or slow down enough to return safely to the roadway. Slopes 4:1 or flatter are considered recoverable.

Non-recoverable Slopes - Slopes which may be safely negotiated but are generally too steep for most motorists to stop their vehicles or to return easily to the roadway. Slopes steeper than 4:1 up to and including 3:1 are considered traversable but non-recoverable if they are smooth and free of fixed-object hazards. Since a high percentage of encroaching vehicles will reach the toe of these slopes, a clear runout area at the toe is desirable.

Critical Slopes - Slopes steeper than 3:1 on which vehicles are likely to overturn.

Backslopes tend to slow an errant vehicle and are therefore not as critical as foreslopes. They may, under certain conditions, be as steep as 1:1.

Roadside ditches are generally categorized as traversable or non-traversable. ***Figures 307-10*** and ***307-11*** present preferred designs for ditches with gradual and abrupt slope changes, respectively. Ditches that fall within the shaded areas of these figures are considered traversable and are preferred for use within the clear zone. Ditch sections that fall outside the shaded areas are considered non-traversable and should generally be located outside the clear zone. There are certain conditions, however, under which these sections may be considered for use within the clear zone. 3R projects; projects with limited right-of-way or rugged terrain; and low volume or low speed roads (particularly if the channel bottom and backslopes are free of any fixed objects) may utilize non-traversable ditch sections when traversable ditches are impractical.

In determining a clear zone width, only recoverable foreslopes (4:1 or flatter), traversable ditches, and backslopes 3:1 or flatter may be included. The recovery area includes the clear zone width plus any non-recoverable slope (over 4:1 through 3:1). These relationships are shown in ***Figure 600-2***.

Several examples of clear zone calculations are included after the figures.

600.2.2 Urban Lateral Offsets

Research has found that curb has very little effect on errant vehicles and thus the clear zone should be calculated as if the curb was not present (based on speed and traffic, ***Figure 600-1***).

Clear Zone is intended to provide a recovery area for errant vehicles. While designers should always strive to keep hazards as far away from the through traveled way as possible, it may not always be practical to provide the Clear Zone on transportation facilities in urban areas where right-of-way is often constrained. On urban facilities where Clear Zone cannot be provided, a minimum lateral offset to fixed objects of 8 feet from the edge of through traveled way for uncurbed roadways is acceptable. On very low speed curbed facilities (35 mph and less), the Operational Offset as described in ***Section 600.2.3*** is acceptable for design features that are functionally necessary (non-breakaway signs and luminaire supports, utility poles, fire hydrants, bus stops, etc.). Otherwise, low speed curbed facilities, shall utilize a minimum Urban Lateral Offset of 4 feet from face of curb. For higher risk locations such as along the outside of curves, offset to fixed objects should be increased to 6 feet for curbed and 12 feet for uncurbed roadways. Refer to ***Figures 600-3*** and ***600-4*** for additional guidance. Where bike lanes and full-time parking lanes are used, their width can be included as offset to fixed objects, however the Operational Offset is still required. Roadside lateral offset also applies to medians.

Posted Speed	Minimum Urban Lateral Offset Requirement	
	Curbed	Without Curb
35 mph or less	Operational offset (See ***Section 600.2.3***) behind curb acceptable for necessary design features. Landscaping and aesthetic features shall be offset per ***Figures 600-3 & 600-4***	8 feet (12 ft. outside curves)
40 to 45 mph	4 feet from face of curb to all fixed objects (6 feet at higher risk locations) refer to ***Figures 600-3 & 600-4***	8 feet (12 ft. outside curves)
50 mph or greater	High Speed - Clear Zone required	

Additional guidance for placement of aesthetic elements (street trees, park benches, trash receptacles etc.) for both curbed and uncurbed urban facilities are provided in the Landscaping Guidelines in the References Section at the end of this Manual.

600.2.3 Operational Offsets on Urban Streets

A minimum operational offset of 1.5' (3' at intersections) is required to be provided from the face of curb to accommodate turning trucks and improve sight distance. The operational offset to any objects accommodates motor vehicles and is necessary to:

- Avoid adverse impacts on vehicle lane position and encroachments into opposing or adjacent lanes
- Improve driveway and horizontal sight distance
- Reduce the travel lane encroachments from occasional parked and disabled vehicles.
- Improve travel lane capacity
- Minimize contact from vehicle mounted intrusions (e.g., large mirrors), car doors, and the overhang of turning trucks.

This operational offset will typically become the controlling criteria where bike lanes or parking lanes meet the previously described lateral clearances. As an exception to fixed object operational offset, traffic barriers should be located in accordance with ***Section 602.1.5***.

601 WARRANTS

601.1 Roadside Barrier Warrants

A roadside barrier is a longitudinal barrier used to shield motorists from natural or man-made obstacles located on the roadside within the clear zone where impacts are expected on one side of the barrier only. In addition to shielding the motorist from roadside obstacles, some types of roadside barrier are required where foreslopes are excessive, and occasionally for the protection of others from vehicular traffic.

601.1.1 Obstacles

Roadside obstacles may be fixed objects or non-traversable terrain. Roadside obstacles located within the clear zone area may or may not require barrier protection. Barriers should be considered in the following circumstances:

1. At bridges, piers and abutments.

2. At culverts, pipes and headwalls depending on traffic volumes, and the culvert's size, location and end treatment. (See ***Section 602.6*** for additional details.)

3. At non-breakaway sign and light supports.

4. At rough slopes in cut sections.

5. At utility poles that cannot justifiably be relocated.

6. At bodies of water or BMP detention ponds where the normal depth exceeds one foot depending on the location and likelihood of encroachment.

7. At transverse ditches if the likelihood of a head-on impact is high.

8. At retaining walls if the anticipated maximum angle of impact is 15 degrees or where there may be snagging potential. (Estimating an encroaching vehicle's angle of impact is usually done using engineering judgment. In general, higher angles of impact are expected on the outside of curves and at locations where items are flared relative to the roadway.)

   Barriers are required to protect mechanically stabilized earth (MSE) retaining wall within the clear zone.

9. At unprotected Noise Walls.

Accident experience, either at the site or at a comparable site, will often be the deciding factor with respect to the placement or omission of a barrier. In all cases, the preferred alternative is to keep the entire clear zone free of fixed objects wherever economically feasible.

601.1.2 Slopes

Embankment height and steepness of foreslopes are the basic factors to be considered in determining the need for barrier slope protection. ***Figure 601-1*** should be used to determine roadside barrier warrants for embankments.

601.1.3 Protection of Others

Barriers are sometimes required to protect others (schools, residences, businesses, pedestrians, bicyclists, etc.) from vehicular traffic. Barrier criteria for protection of others from errant vehicles are not as defined as in other barrier warrant cases. Such decisions are normally made using accident experience, either at the site or at comparable locations along with engineering judgment.

601.1.4 Protection on Low Speed Roadways

Barrier protection on city streets and urban type facilities with design speeds less than 50 mph is not normally required. However, on roadways where the design speed is greater than 25 and less than 50 mph, the designer should specify protection at locations where geometric conditions, accident experience or other circumstances indicate that protection should be considered.

601.1.5 Protection on Very Low-Volume Local Roads (ADT ≤ 400)

The guidelines presented elsewhere in this section were developed using the AASHTO Roadside Design Guide. Guidelines contained in the AASHTO Guidelines for the Geometric Design of Very Low-Volume Local Roads (less than or equal to 400 ADT) may be used in lieu of those presented here.

On roads with very low traffic volumes, research has found that roadside clear zones provide very little benefit, and that traffic barriers are not generally cost-effective. With no criteria to identify appropriate locations where a clear zone or barrier may be warranted, the very low-volume guidelines provide great flexibility to the designer in exercising engineering judgment to decide when it is appropriate to provide improved roadsides. These guidelines apply to both new construction and existing roads.

A clear zone of any width should provide some contribution to safety, so when feasible to do so at little or no additional cost, it should be considered for very low-volume local roads.

601.1.6 Preservation of Safety Grading

Designers should preserve unobstructed areas on roadway designed and constructed with safety grading (***Section 307.2.1***). Typically, safety grading was part of the original construction and is intended to provide a safe recovery area outside of the required clear zone. These unobstructed areas should not be used to locate hazards, such as camera towers, ITS or WIM equipment, BMP detention ponds or aesthetic landscaping. To ensure driver safety and the financial investment made in safety grading the addition of hazards should be located behind existing barriers or as far away from traveled lanes as possible.

601.2 Median Barrier Warrants

A median barrier is a longitudinal barrier used to separate opposing traffic on divided highways having relatively flat, traversable medians. ***Figure 601-2*** provides barrier warrants for freeways to determine the need for median barriers, based on the width of the median and the volume of traffic on the facility. It may also be used for expressways with full access control. The use of the terms freeway and expressway in this instance apply to the operational characteristics of the highway, not necessarily the functional class designation. The use of median barrier on divided highways that do not have full access control requires engineering judgment and analysis with consideration to such items as right of way constraints, property access needs, sight distance at intersections, barrier end termination, etc.

A median barrier may be high tensioned cable, guardrail, or concrete barrier. If the median is wide enough so that the barrier is outside the clear zone of opposing traffic, then roadside barrier warrants may be used.

601.2.1 Safety Studies

It is recommended that a safety study be conducted to determine if median barrier protection would be beneficial at locations shown in ***Figure 601-2*** as "evaluate need for barrier" or "barrier optional." The study may include the following factors: traffic volumes, vehicle classifications, median crossover history, crash incidents, vertical and horizontal alignment relationships and median-terrain configurations.
If barrier is chosen, see ***Section 602.2.2*** for median barrier design considerations.

601.3 NHS Criteria

Highway safety features, including longitudinal barriers, anchor assemblies, bridge terminal assemblies and impact attenuators installed on the National Highway System (NHS) must demonstrate satisfactory crash worthy performance and be accepted by the FHWA. The AASHTO Manual for Assessing Safety Hardware, (MASH) contains the current recommendations for testing and evaluating the crashworthy performance of barriers and has replaced NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features for the evaluation of new devices. Crashworthiness is currently accepted if either of the following conditions are met:

1. A barrier system has met all of the evaluation criteria listed in MASH or NCHRP Report 350 for each of the required crash tests, or
2. A barrier system has been evaluated and found acceptable as a result of an in-service performance evaluation.

A given feature must be tested to one of six different test levels (TL) defined in Report 350 and MASH. The six test levels correspond to the following crash testing matrix.

All six levels of testing determine if the barrier is structurally adequate to contain the vehicle type, while TL-1 through TL-3, criteria looks at the vehicle occupant survivability.
In general, all permanent devices installed on the NHS in Ohio must meet TL-3 requirements.
Exceptions to this would be allowed in low speed urban situations where a TL-3 protection is not feasible or cost prohibitive; in those locations a TL-2 device may be appropriate.

Crash Testing Matrix		
Test Level	**Vehicle**	**Speed (MASH)**
TL-1	Passenger fleet	31 mph
TL-2	Passenger fleet	44 mph
TL-3	Passenger fleet	62 mph
TL-4	Single unit delivery van	56 mph
TL-5	Tractor Trailer	50 mph
TL-6	Tanker Trailer	50 mph

601.4 Design Considerations for Large Trucks

Designers should consider the catastrophic nature of accidents involving tractor-tanker trucks and other large vehicles, even though such crashes are relatively rare and occur at generally unpredictable locations. Wherever large vehicles comprise a significant percentage of the traffic volume, crash potential or crash histories should be carefully reviewed to determine if higher performance traffic barriers are warranted and likely to be cost-effective.

Although objective warrants for the use of higher performance barriers do not presently exist, subjective factors most often considered for new construction or safety upgrading to TL-5 or TL-6 devices include:

1. High percentage of heavy vehicles (along major corridors, on hazardous material routes, or near hazardous industries),

2. Adverse geometrics (vehicle conflict points, sharp curvature, long downhill grades, poor sight distance, or adverse pavement surfaces like shoulder wedges or reverse superelevation on shoulders), and

3. Severe consequences associated with the penetration of a large vehicle (buildings or transit facilities underneath a bridge or multi-level interchange, sensitive environmental areas, or at critical bridges and tunnels).

602 SITE CONSIDERATIONS

Standards and guidelines are presented in this section for certain general site conditions; however, a site visit is essential to ensure that all design considerations have been addressed.

602.1 Roadside Protection

When a roadside obstacle needs to be shielded, the designer should initially consider the most flexible barrier system installed as far from the traveled way as possible. Subsequent systems should be considered in order of increasing strength and decreasing distance from the roadway. In general, the designer should consider options for roadside protection in the following order:

1. Install flared guardrail and either terminate the end outside the clear zone or bury it into a backslope.

2. Install tangential guardrail and terminate the end with a Type B flared end terminal.

3. Install tangential guardrail and terminate the end with a Type E tangential end terminal.

4. Install concrete barrier according to ***Section 603.1.2*** and terminate the end according to ***Section 603.6***.

602.1.1 Location/Offset

The normal roadside barrier location, with respect to the edge of traveled lanes, is shown in ***Figure 301-3***. Minimum barrier clearances, measured from the face of the barrier to the face of the obstacle, are shown in ***Figure 603-2***. (See ***Section 603.4*** for minimum clearances for impact attenuators.) Although variations from these offsets may occur as a result of reduced graded shoulder width, the face of guardrail should not be located closer than 4 feet to the edge of the traveled lane. See ***Section 602.1.5*** for guidelines concerning the use of curb with guardrail.

602.1.2 Length of Need on Tangent Alignments

Length of need is the total length of a longitudinal barrier that is needed to shield an area of concern (warranting feature). The length of need point in a gating end terminal or impact attenuator determines how much of the end treatment can be contributed to the length of need for the barrier.

If it is determined that barrier protection is required to shield a fixed object, ***Figure 602-1*** should be used to determine the length of need. The primary variables are the Runout Length (L_R) and the Lateral Extent of the Hazard (L_H). The Runout Length is the theoretical distance needed for an errant vehicle leaving the roadway to come to a stop. The Lateral Extent of the Hazard is the distance from the edge of the through traveled way to the far side of the hazard or to the edge of the clear zone if the hazard extends beyond the clear zone. The other three variables are the Tangent Length of barrier (L_I,) the Lateral Distance from edge of the through traveled way (L_2), and the Flare Rate (a:b).

The formula in ***Figure 602-1*** shown for computing the barrier length of need is appropriate where tangent roadways are involved.

Short runs of barrier should be avoided where economically feasible. Gaps of 300 feet or less between adjacent runs of guardrail should be closed.

Sample Calculations for length of need on tangent alignments are included in the ***Examples***.

602.1.3 Length of Need on Curved Alignments

Horizontal curvature of a roadway may have an effect in determining the barrier length of need in roadway design. In general, the length of need for a barrier on the outside of curves with a degree of curvature equal to 2°00' or flatter can be calculated as if the barrier was installed tangentially. However, a vehicle leaving the roadway on the outside of a curve sharper than this will generally follow a tangential runout path.

For those cases involving a horizontal curve sharper than the limiting values given above, rather than using the theoretical LR distance, the tangent line from the curve to the outside edge of the warranting feature (or to the clear zone) should be used to determine the appropriate length of barrier needed (See **Figure 602-2**.) The guardrail should not be flared in these locations, since the potential impact angles would generally exceed acceptable design limits.

Lengths of need should not be adjusted on the inside of horizontal curves. These locations should be treated as if they were on a tangent and LR should be measured along the length of the curve.

Sample Calculations for length of need on curved alignments are included in the ***Examples***.

602.1.4 Grading for Barriers & End Treatments

To function properly, anchor assemblies and impact attenuators need to be installed with proper grading. The grading is designed to ensure that an impacting vehicle strikes the device at the appropriate height and with all four wheels on the ground. It also helps to reduce the potential for snagging and vehicle rollover during and after impact. Adequate earthwork and excavation should be included in the plans to ensure that all devices have proper grading.

Ideally, the area immediately behind and downstream of all gating terminals should be reasonably traversable and free from fixed objects to the extent practical. A 20 feet by 75 foot area with 10:1 maximum slopes is required. When this is not practical, due to possible impacts to streams, and wetlands, the designer should consider alternatives. Also, there may be situations where existing conditions may preclude the acquisition of additional rights-of-way or easements necessary to build fill slopes that accommodate this grading. In these situations, it may be advisable to select a terminal that requires less extensive grading (i.e. non-gating or redirective, see ***Section 603.2.1***) or extend a run of guardrail so that the terminal may be placed on more favorable terrain, or buried in the backslope. The designer should attempt to provide a clear area with recoverable slopes (4:1 or flatter) over the same 20 feet by 75 feet area. If a clear runout path is not attainable, this area should be similar in character to the upstream, unshielded roadside area.

In most cases, longitudinal barriers should not be located on slopes steeper than 10:1. Therefore, where a barrier is located outside the graded shoulder, special grading generally will be required to provide slopes that are 10:1 or flatter. Also, 6:1 slopes are of particular concern due to vehicle ramping effects. Barriers installed on 6:1 slopes should be limited to cases where the barrier is located at least 12 feet or more from the edge of the break point for the 6:1 slope to minimize the potential for an errant vehicle to vault over the guardrail. The Buried-in-Backslope Anchor Assembly is one exception that has been designed specifically for 4:1 or flatter slopes. MGS Guardrail may be used with 10:1 approach slopes (See ***Section 603.3.1*** for additional information.)

602.1.5 Guardrail with Curbs

Curbs are generally classified as mountable or barrier curbs. Vehicles can, and do, safely traverse mountable curbs. Barrier curbs tend to inhibit vehicles from crossing over them at low speeds, but they are not a substitute for longitudinal barriers.

When guardrail must be used in conjunction with a curb, the location of the guardrail relative to the curb should be carefully considered to minimize unacceptable post impact vehicle trajectories. When a vehicle strikes a curb, the resulting trajectory may cause the vehicle to impact the guardrail too high. In some cases the vehicle could clear the guardrail altogether.

If guardrail is warranted and curbs are present, then the face of Type MGS guardrail should be located within 6 inches behind the face of the curb. Because of the vehicle vaulting potential, if the guardrail cannot be placed as described above, then the guardrail should be installed well behind the curb to allow the vehicle suspension to return to a normal state as shown in the following table.

Design Speed	Guardrail at Curb	Guardrail Behind Curb
Up to 45 mph	Maximum of 6 inch sloping faced curb: MGS guardrail up to 6 in. behind curb	No closer than 8 feet.
45 and 50 mph	Maximum of 6 inch sloping faced curb: MGS guardrail up to 6 in. behind curb	No closer than 13 feet.
Over 50 mph	Maximum of 4 inch sloping faced curb: MGS guardrail up to 6 in. behind curb Above 55 mph, the sloping face of the curb should be 3:1 or flatter and 4 inches or smaller.	Guardrail should not be located behind curb.

602.1.5.1 On High Speed Roadways

All guardrail on curbed roadways with a design speed of 50 mph or greater preferably should be located so the face of guardrail is at the face of curb. When curb and gutter is used, the gutter pan width will need to be increased to comply with these guidelines and to maintain a minimum 4 feet guardrail offset from the traveled way.

The curb height should be limited to 4 inches or less when used in conjunction with guardrail on high speed roadways.

602.1.5.2 On Low Speed Roadways

Guardrail is not normally used on curbed roadways having design speeds less than 50 mph (see ***Section 601.1.4***). Where guardrail is deemed necessary on these roadways, the same criteria used for roadways with design speeds of 50 mph or greater is recommended. However, since the risk of vaulting is considerably less on low speed roadways, the designer may give more consideration to the location of the guardrail relative to the edge of traveled way than to its location relative to the curb.

602.1.5.3 End Treatments and Impact Attenuators in Curbed Sections

None of the approved anchor assemblies or impact attenuators listed in ***Sections 603.3*** and ***603.4*** have been designed or tested for use with curbs; consequently, the designer should use the guidelines provided for uncurbed sections in addition to engineering judgment and recommendations from the manufacturer to select end treatments in curbed sections. The current recommendation from product vendors is to ensure curbs are not present (if practical) along the length of the product and for a distance of 50 feet in advance of the product. When terminating or removing curbs in the vicinity of end treatments and impact attenuators remember to taper the curb height from 4 or 6 inches to flush with the pavement over a distance of 10 feet.

602.2 Median Protection

Two types of shielding are necessary in medians. First, shielding of fixed objects is required if located in the clear zone of either direction of traffic. Second, if the median width warrants or a safety study shows a history or potential for Cross Median Crashes some type of barrier system may be needed. See ***Section 602.2.2***.

602.2.1 Shielding of Fixed Objects in the Median

When a median hazard requires protection, the treatment depends upon the available width of the median. For the purposes of installing barrier, a median is considered wide when the barrier installed in the median does not extend into the clear zone of the opposing side of traffic. Conversely, when the guardrail run extends into the clear zone of the opposite side of traffic, the median is considered narrow.

602.2.1.1 Narrow Median Barrier Installations

Refer to **SCD MGS-6.1** and **MGS-6.2** Design A for details.

602.2.1.2 Wide Median Barrier Installations

Refer to **SCD MGS-6.1** and **MGS-6.2** Design B for details.

602.2.1.3 Greatest Offset Method to Shield Center Median Piers

Another design for pier protection (refer to **SCD MGS-6.2**) used by some Districts, is to shield center median piers with concrete barrier. This design uses concrete barrier to encase the pier (**SCD RM-4.4**), and then taper the concrete barrier to the end section (**SCD RM-4.6**). Finally install two narrow Type 2 Impact Attenuators, one at each end. This eliminates the need for perhaps hundreds of feet of guardrail as shown in **SCD MGS-6.2**. Contact the **Office of Roadway Engineering** for more information and design details. Proper grading in advance and alongside of the barrier is crucial in ensuring proper performance.

602.2.2 Mitigation of Cross Median Crashes (CMC)

602.2.2.1 Barrier Selection

If a median barrier is determined to be necessary for shielding of CMC ***(Section 601.2),*** then the selection of the type of barrier to be used in the median is based on several factors, including the Test Level desired, median cross section, and barrier deflection.

<u>Test Level</u> - A safety study should determine the causes of CMC to determine the type of vehicle involved, and barrier selection should be based on the study. Guardrail is rated for TL-3 protection, High Tension Cable products are either rated to TL-3 or TL-4 (single unit truck) depending on the product. Single Slope Concrete Barrier is considered a TL-5 system capable of handling a tractor trailer. See ***Section 601.4*** for further discussion on Large Trucks.

Cross Section Type - Barrier selection also depends on the median configuration, whether or not there is a mounded median, depressed median of 4:1, 6:1, or 10:1 or flatter slopes, the width of the paved shoulder and graded shoulder. Other factors include but are not limited to bifurcation and differential superelevation between the traveled lanes.

Barrier Deflection - The designer also has to be aware of the allowable deflection to appropriately select a median barrier product. On one hand, rigid concrete barriers do not deflect, but may require closed drainage and thus are expensive. High tension cable barrier has large deflections.

602.2.2.2 Cable Barrier Placement in the Median

On 6:1 or flatter depressed median slopes, cable should be placed 8 feet up the slope from the bottom of the ditch to avoid drainage hydraulics, poor soil quality, and vehicle under-ride possibilities. If the median slopes are consistent, placement of cable on the slope outside of this zone is allowed. Another acceptable location for cable placement is at the top slope on one side of the median if the paved shoulder is wide enough to accommodate the minimum offset of 12 feet from the edge of traveled way. This location places the cable at the best grading on the near traffic side and at the farthest point away from opposing traffic and allows for a factor of safety of the cable deflection. This location may result in an increase in nuisance hits. Designers need to understand how cable reacts during crashes before the median locations are selected. Consideration should also be made when ending/beginning cable runs so that staggered placement on either side of the median does not unintentionally leave wide gaps between barrier runs. (Refer to ***Figures 602-3a, 602-3b, 602-4a & 602-4b***.) For more information contact the Standard Engineer in the Office of Roadway Engineering Services. The maximum post spacing allowed is 15 feet.

602.2.2.3 Cable Anchors

If installed in the clear zone, cable systems need to be terminated with crashworthy anchors. The maximum allowable distance between cable anchors is 3000 ft. Most crashworthy designs have breakaway anchors. Breakaway anchors will release the tension in the entire run of cable rendering it ineffective until repaired. If a vehicle is tangled in the cable, tension can be easily dropped out of the system if each run of cable has one set of breakaway anchors. Because of varying system length, the length of need point should be detailed on the plans.

602.2.2.4 Cable Barrier as the Primary Barrier System

When designing a project utilizing cable barrier, designers should continue to use guardrail or concrete barrier to protect existing fixed objects. Cable barrier should not be used as the primary means of shielding fixed objects in highway medians.

602.3 Gore Area Protection

Diverging gores are locations where one or more lanes of a road carrying traffic in the same direction diverge away from each other. (Unidirectional traffic exists on both sides of a gore.) Impact attenuators are typically used to terminate the ends of longitudinal barriers located in diverging gores. (See ***Section 603.4*** for additional information on impact attenuators.)

602.4 Protection at Drives and Side Roads

When normal mainline guardrail is interrupted by a side road or drive, the opening should be designed as shown in ***Figure 603-3***.

The introduction of barriers at drives and side roads may have an adverse effect on both horizontal and intersection sight distances. These sight distances should be investigated when barriers are used at these locations. (See ***Sections 602.6.2 and 602.7*** for additional information.)

602.5 Protection at Bridges and Fixed Objects

Concrete barrier end protection, utilizing guardrail with bridge terminal assemblies, shall be used at the approach end of bridge parapets, and other similar fixed objects, on all facilities where the design speed is 50 mph or greater. (See ***SCD MGS-6.1***)

Pier protection in narrow medians and along the roadside is often accomplished using concrete barrier.

From BDM Section S1.3.4, the columns of single-column and two-column piers shall be considered non-redundant. The columns of cap-and-column piers with three or more columns shall be considered redundant. See BDM Section S36.5.1 for protection requirements.

602.5.1 Guardrail at Bridges & Large Culverts

Figures 602-1 and ***602-2*** should be used to calculate the barrier length of need at all bridges and culverts.

Flared guardrail should be provided at overpasses and on safety and clear zone grading projects according to ***SCD MGS-6.1***.

Flared guardrail should be provided at underpasses or other fixed objects on safety and clear zone grading projects according to ***SCD MGS-6.2***.

Tangent guardrail should be provided on common grading projects.

There are occasionally areas where the calculated lengths of need are impractical. An example would be where a drive or intersection is located too close to a bridge and cannot be relocated. In such cases, the approach guardrail length may be reduced as necessary. In no case shall the minimum treatment be less than shown in ***Figure 603-4.***

On divided highways, guardrail is not required at either of the bridge parapet trailing ends unless it is warranted because of the lack of clear zone distance, the presence of openings between bridges, or where it is required in conjunction with a bridge railing.

602.6 Protection at Drainage Structures

Adequate drainage is one of the most critical elements in roadway design. A comprehensive drainage design requires consideration of roadside safety as well as hydraulic efficiency.

In general, no part of an unshielded drainage feature within a clear zone graded roadway, excluding curbs, should extend more than 4 inches above the surrounding terrain. (Drainage features that do not comply with this criterion are herein referred to as "protruding.")

(See the **Location and Design Manual, Volume Two** for specific drainage requirements.)

602.6.1 Transverse Drainage

For pipes with diameters or spans greater than 36 inches:

1. Extend the exposed pipe ends outside the clear zone when practical.

2. When the above option is impractical, shield the ends of the exposed pipe per ***Section 602.5.1***.

For pipes with diameters or spans less than or equal to 36 inches located in areas where clear zone or safety grading is not provided:

Provide standard half-height headwalls (SCD HW 2.1 or HW 2.2) at exposed pipe ends.

For pipes with diameters or spans less than or equal to 36 inches located in areas where clear zone or safety grading is provided:

Extend the exposed pipe ends outside the clear zone when practical and provide standard half-height headwalls.

When the above option is impractical, use slope tapered pipe end treatments.

602.6.2 Intersecting Embankments & Parallel Drainage

Intersecting embankments are slopes that are transverse to the roadway. They are usually created by median crossovers, intersecting roadways and driveways. These slopes are typically struck head-on by vehicles that have left the traveled way.

Median crossovers on Interstates/Freeways shall use a 12:1 slope.

Embankment slopes for side roads should be as flat as practical, and drainage pipes underneath side roads should be located outside of the mainline clear zone where practical. This can typically be accomplished with minor adjustments to the ditch profiles.

For driveways on projects with clear zone or safety grading, the intersecting embankment slopes should be as flat as practical and:

1. All protruding drainage appurtenances should be placed outside the mainline clear zone, when practical. Standard half-height headwalls should be provided on all pipe ends located outside the clear zone.

2. If a protruding drainage appurtenance cannot be located outside the clear zone then it should be placed as far from the roadway as practical and treated similarly to drive pipes on projects without clear zone or safety grading.

3. An enclosed drainage system (storm sewer) may also be considered.

For driveways on projects without clear zone or safety grading, the intersecting embankment slopes should be as flat as practical and:

1. Exposed ends of pipes with diameters or spans less than or equal to 24 inches should be miter cut to conform to the prevailing slope.

2. Exposed ends of pipes with diameters or spans over 24 inches should be designed with standard half-height headwalls.

3. An enclosed drainage system may also be considered.

602.6.3 Special End Treatments

End treatments that utilize bars or grates designed as safety treatments for exposed pipe ends are commercially available. However, these end treatments reduce hydraulic efficiency and exhibit a high potential for clogging. This type of end treatment should only be used when all other reasonable options have been exhausted.

602.7 Sight Distance Considerations

The introduction of longitudinal barriers may have an adverse effect on both horizontal and intersection sight distances. The effect on both distances should be investigated at all locations where barriers are used. (See ***Sections 201.2.2*** and ***201.3.2*** for additional guidance.)

603 ROADSIDE SAFETY DEVICES

The goal of any highway roadside safety device is simply to assist in providing a forgiving roadside for an errant motorist. The goal is met when the feature does one of the following without causing serious injuries to the occupants of the vehicle or to other motorists, pedestrians or work zone personnel:

1. contains or redirects the vehicle away from the hazard,

2. decelerates the vehicle to a stop over a relatively short distance,

3. readily breaks away, fractures or yields,

4. allows a controlled penetration, or

5. allows the vehicle to safely traverse the feature.

(See ***Section 601.3*** for additional information.)

603.1 Longitudinal Barriers

Longitudinal barriers function by containing and redirecting impacting vehicles. They are typically classified into three types based on relative strength characteristics: flexible, semi-rigid and rigid.

Deflection characteristics of a longitudinal barrier system determine the minimum clearances between the face of the barrier and the face of the object it shields. Minimum barrier clearances are listed in ***Figure 603-2*** along with typical applications for the standard types of barrier described in the following sub-sections.

603.1.1 Flexible Cable Systems

Cable systems are considered flexible systems in that they tend to exhibit large deflections when impacted. Although large deflections can be problematic they produce a relatively soft impact allowing for a gradual deceleration of the vehicle.

603.1.1.1 Generic Low Tension Cable

Generic low tension cable systems are no longer allowed to be constructed on ODOT's system.

603.1.1.2 Proprietary High Tension Cable Systems

Although proprietary in nature, high-tensioned systems consists of the same standard cable mounted under substantial tension between anchors, but each system has its own light weight steel post. Ohio does use high tensioned systems in medians of divided highways as a method of preventing cross median crashes. See the approved products list on the Office of Roadway Engineering's webpage.

603.1.2 Semi Rigid Barriers

ODOT's approved semi rigid barriers include: Type 5 and Type MGS guardrail – both strong post w-beam guardrail systems. Other proprietary guardrail systems are not considered equivalent and are not acceptable for use on ODOT jobs.

- Type MGS guardrail is a MASH TL-3 crashworthy system at a 31 inch installation height (+/-1 in.) New guardrail designs should utilize MGS.

- Type 5 guardrail is an NCHRP 350 TL-3 crashworthy system at a 29 inch installation height (+/-1 in.). Still acceptable on the State System, this system should be limited to repair locations of existing rail. **Refer to Plan Insert Sheets (GR series) and the July 2012 Version of this Manual for Type 5 guardrail design standards**.

The three major components of a strong post barrier are the rail, posts, and blockouts. This ribbon of rail acts to capture impacting vehicles and to dissipate energy up and down the rail length. The tension on the rail from an impact can be transferred a considerable distance. Proper anchoring of the rail at both ends is critical in achieving proper performance.

Guardrail posts are designed to support the rail at the appropriate height and provide lateral support during an impact. For most impacts, the posts are designed to rotate through the soil, rather than bend at or near the ground surface. This rotation helps to contribute considerably to the energy absorbed in the collision and helps to prevent contact between the vehicle and the posts. For this reason, paving around posts is not advisable if the thickness of the pavement would prevent this rotation from occurring. Three inches of asphalt pavement is the maximum allowable thickness for paving under guardrail. See **Sample Plan Note R116** for additional information on paving under guardrail.

For guardrail installations to perform properly during an impact, adequate soil support must be provided for the posts in the guardrail run. To ensure this support, longer posts should be specified at locations where the distance behind the post to the slope break point is less than one foot. These locations should be specifically identified in the plans. See **SCD MGS-1.1** for additional details and proper post length.

The use of blockouts increase the overall performance of a guardrail system. Blockouts minimize the potential for a vehicle's wheels to snag on the posts and reduce the likelihood of a vehicle vaulting over the barrier. This is accomplished by maintaining the height of the rail as the barrier deflects and rotates downward during an impact. The standard Type MGS Guardrail uses a 6" wide x 12" deep x 14" long blockout. Crash testing has also been successfully completed on MGS with reduced and eliminated blockouts. On 2 lane facilities where the overall typical section width is limited by steep foreslopes, drop-offs, or other site constraints, engineering judgment may be used to consider eliminating the blockout -particularly if this will help improve the overall backfill/embedment of the guardrail posts.

603.1.2.1 Type MGS Guardrail

The Midwest Guardrail System, Type MGS, is Ohio's strong post barrier used for roadside protection where 5 feet of barrier clearance is available. Type MGS guardrail uses w-beam rail with a top rail height of 31 inches to accommodate larger vehicles and the blockouts are 12 inches deep. This guardrail system can be placed on foreslopes as steep as 10:1 and may be flared away from the roadside at a rate of 7:1. Type MGS guardrail has passed MASH TL-3 testing. See **SCD MGS-1.1** for additional details.

Half Post and Quarter Post spacing is available for MGS for reduced deflections: 3.5 ft. and 3 ft. respectively. See **SCD MGS-2.1** for additional details. The reduced post spacing should be introduced 25 feet upstream where the reduced deflection is desired for each reduction in deflection. Thus if going from normal post spacing to quarter post spacing, use 25 feet of half post spacing before reducing further to the quarter post spacing which should also be 25 feet upstream of where the actual reduction in guardrail deflection is needed.

The Midwest Guardrail System also performed successfully in a crash test with one omitted post. Designers may note in the plans to leave out one guardrail post at a specific location within a standard run of MGS to avoid utilities or other underground conflicts. Fifty feet of guardrail (which may include the anchor) should be available both upstream and downstream of the omitted post to maintain tension and strength in the system. Posts should not be omitted where curb is present.

603.1.2.3 Barrier Design Guardrail

Barrier Design Guardrail is used primarily in bi-directional median applications on any roadway where a minimum barrier clearance of 5 feet can be provided. Barrier Design Guardrail is identical to standard MGS guardrail with the addition of blockouts and rail on the opposite side of the posts. Type MGS Barrier Guardrail requires a minimum cross slope approach of 10:1 on both sides of the barrier. See **SCD MGS-2.1** for additional details.

603.1.2.5 Long Span Guardrail

A MASH TL-3 long span guardrail design for spanning up to 25 feet across culverts is shown on **SCD MGS-2.3**. This guardrail system with breakaway posts has a deflection of 8 feet from the face of rail and requires 2 feet of grading (8:1 max) behind the post. When possible consider grading up to the back of headwalls that would otherwise protrude more than 4 inches above the slope break point elevation within that 8 ft. deflection area. Otherwise, the culvert should be extended so that the headwall does not become a hazard in the long span guardrail deflection area.

A minimum of 62.5 ft. of Type MGS guardrail is required adjacent to the Breakaway CRT posts to maintain strength in the overall system.

603.1.4 Rigid Concrete Barrier

Concrete barriers are used in locations where barrier deflections cannot be tolerated. Because of its rigidity and shape, it is very effective for small angle impacts and is preferred for use where the chance of impacting it at an angle of 15 degrees or greater is minimal. It also requires less maintenance than steel beam guardrails. Overall impact severities for these barriers are usually greater than the other types of systems.

At locations where a standard barrier cannot be installed, the face of fixed objects within the clear zone should be designed with the concrete barrier shape. Typical locations are along retaining walls and walls that connect pier columns. On upgrading projects where the face of these fixed objects does not have existing protection, the concrete barrier shape should be provided to shield these objects.

Concrete Sealers are not required for concrete barrier.

603.1.4.1 Single Slope Barrier

ODOT changed its standard concrete barrier shape to that of a single slope, from the New Jersey shape in 2003. Single slope barriers have advantages of better crash test performance for TL-3 vehicles, and the capability of being a TL-5 barrier. It is also capable of having multiple pavement overlays placed next to it without having to reset the barrier.

The single slope standard does not require a concrete base outside the end sections, as was required with the previous NJ safety shape. The single slope barrier, however, does need a solid base material (asphalt or aggregate) to support its own weight, and an overlay of material at the toe of the barrier. Single slope barrier does not require horizontal steel rebar except in the end sections and end anchorages. It is used on any roadway in areas where signs, lighting or other unyielding objects are to be mounted on top of the barrier. Concrete barriers are to be terminated with reinforced foundations. Use an End Anchor as shown on **SCD RM-4.3**, unless the barrier end connects to an impact attenuator or guardrail, in which case an End Section as shown on **SCD RM-4.6** should be used in lieu of the End Anchor.

603.1.4.2 Types B & B1

Single Slope Concrete Barrier, Type B, is 28 inches wide at the base and 42 inches tall. Single Slope Concrete Barrier, Type B1, is 33.75 inches wide at the base and 57 inches tall. The additional height of the barrier in excess of the Type B serves as the glare screen. Refer to ***Section 604*** for additional information on glare screens.

603.1.4.3 Type C & C1

Single Slope Concrete Barrier, Types C and C1, are used on any roadway in narrow medians where the difference in elevation on either side of the barrier is less than or equal to 24 inches. The barrier varies in width at the base depending on the height. For Type C, with the height on one side fixed at 42 inches, the other side can vary in height from 42 inches to 66 inches. Type C1 varies from 57 inches to 81 inches on one side while the other side is fixed at a height of 57 inches. Barriers with elevation differences greater than 24 inches are to be individually designed.

603.1.4.4 Type D

Single Slope Concrete Barrier, Type D, is 20 inches wide at the base and 42 inches tall. It has the single slope profile on only one side of the barrier; therefore, it can be used on any roadside where impacts are expected on only one side of the barrier. It is often used for the protection of piers and other fixed obstacles. Two back-to-back Type D barriers should not be used in lieu of a single Type B median barrier as debris collects behind the barrier causing maintenance problems. Nor should Type D barrier be modified to a taller height to accommodate glare screen protection. Separate glare screens attachments should be used. Refer to ***Section 604*** for additional information on glare screens. See **SCD RM-4.5** for barrier and end anchors details and for use at obstructions. See **SCD RM-4.6** for Type D end sections.

603.1.4.5 New Jersey Shape

ODOT’s previous standard was the NJ safety shape barrier. This barrier type has a 3 inches vertical portion at the base which plays no significant role in the performance of the barrier, but provides an allowance for one future pavement overlay. The NJ shape continues to meet at least TL-3 requirements and can be utilized on very short lengths where existing NJ barrier is present. Plan insert sheets of this design are available on the Office of Roadway Engineering’s web page.

603.1.4.6 Portable Concrete Barrier

Refer to **SCD RM 4.1** and **RM 4.2**. All generic portable concrete barrier used on ODOT's system must be constructed using these drawings. For details and additional information on anchoring PCB to bridge decks and asphalt surfaces, see **SCD PCB-91** and the frequently asked questions section of the Office of Roadway Engineering website. For other approved Portable Barriers refer to the Office of Roadway Engineering's website for the approved products list.

603.1.4.7 Zone of Influence

Designers are encouraged to minimize objects on top of and behind concrete barriers because of truck box yaw into the barrier in an impact. Discrete objects such as lighting standards or sign supports could be snagged by a box truck, or continuous objects like sound wall mounted on top of barrier could be damaged by a truck's cargo box rotation. For single slope and NJ shape barriers, a reasonable area to keep as free of objects as reasonable would be 32 inches behind the top face of the barrier to 80 inches above it. Generally, objects placed in this area would not compromise the crashworthiness of the barrier, but incidental damage to the impacting truck's cargo box or the object itself may occur.

603.2 Characteristics of Anchor Assemblies & Impact Attenuators

Originally end terminals were designed simply to anchor the ends of guardrail runs. However, over the years safety at the ends has become a major concern. As a result, guardrail end terminals (anchor assemblies) have taken on additional functions. An anchor assembly can function by:

1. Decelerating a vehicle to a safe stop within a relatively short distance permitting controlled penetration of the vehicle behind the device;

2. Containing and redirecting the vehicle;

3. A combination of the above.

Anchor assemblies must also be capable of developing the full tensile strength of the rail elements.

Impact attenuators (crash cushions) are designed primarily to safely stop a vehicle within a relatively short distance. Some common uses of impact attenuators are at exit gores, on or under bridges where piers require shielding, and at the ends of roadside and median barriers.

Crashworthy anchor assemblies and impact attenuators can be classified as either (1) energy absorbing or not, (2) gating or non-gating and (3) redirective or non-redirective.

603.2.1 Energy Absorbing

When a vehicle impacts an energy absorbing end terminal, energy from the impact is dissipated in a variety of ways through the deformation of the vehicle's crush zone and also from the barrier itself. An energy absorbing system is designed to expend crash energy by crumbling steel or other material so that most of the energy will be dissipated internally within the barrier system. The advantage of an energy absorbing system is that a vehicle and its occupants can be decelerated to a stop within 30 to 50 feet under designed impact.

603.2.2 Gating

A non-gating system will bring an impacting vehicle to a controlled stop or redirect it while a gating system will allow a vehicle impacting the system at an angle to pass through the system along the same general path. Gating guardrail end terminals, will remove very little of the impacting energy, thus vehicles will pass through the system at close to the impacting speed. See ***Section 602.1.4*** for proper grading recommendations with regards to gating end terminals, especially the 20 feet by 75 feet run out area behind and beyond the start of the gating terminal.

The length of need (LON) point in a non-gating system is located at the nose of the system. When using a gating system, the LON point needs to be identified to determine what portion of the system can be used as part of the barrier's LON. See ***Sections 602.1.2*** and ***602.1.3*** for additional information on length of need.

603.2.3 Redirection

A redirective system will redirect an impacting vehicle away from a fixed object when the system is struck at an angle on the side. A non-redirective system will allow a vehicle to continue in approximately the same direction until it comes to a stop.

A non-redirective system is designed to contain and capture a vehicle impacting downstream from the nose of the unit. It provides protection in an end-on collision by absorbing the impacting vehicle's kinetic energy; however, it does not control an angle impact and it may allow pocketing or penetration. (Pocketing is said to have occurred if, upon impact, relatively large lateral displacements happen over a relatively short longitudinal distance.) All non-redirective devices are also gating. LON is established at the rear of the device. Sand barrel arrays are typical non-redirective devices.

A redirective, gating system has redirective capabilities over a portion of its length. The LON point varies from system to system. These devices are almost always anchor assemblies.

A redirective, non-gating system is designed to contain and redirect a vehicle impacting downstream from the nose of the unit. Redirection is provided over the entire length of the device; therefore, the LON is established at the nose of the device.

603.2.4 Proprietary Products

Many of the following devices are proprietary products, which are subject to change at the manufacturer's discretion. The information provided in this manual is accurate and up-to-date at the time of publication and represents the currently approved versions of these products. New products may be introduced and modifications to existing products may occur, which may or may not be approved by ODOT. Shop drawings of all approved proprietary devices are provided with the standard construction drawings. For additional guidance link to Office of Roadway Engineering's web page on Proprietary Roadside Safety Devices or contact the Roadway Standards Engineer.

Each proprietary end terminal and impact attenuator must be installed according to the manufacturer's recommendations.

603.3 Anchor Assemblies

603.3.1 Buried-In-Backslope

The buried-in-backslope anchor assembly is a flared, redirective, non-gating, non-proprietary, end terminal. The length of this terminal varies depending upon field conditions. Its construction is similar to guardrail except the buried-in- backslope terminal uses 8.0 feet long posts and a rubrail. It is installed with 4:1 or flatter foreslopes and backslopes as steep as 1:1. A vehicle impacting this terminal close to the buried end may be able to climb 2:1 or flatter backslopes and encroach behind the guardrail. Consequently, where backslopes are 2:1 or flatter a 75 feet minimum length of guardrail must be provided upstream between the warranting feature and the intersection of the guardrail with the ditch flowline. Where backslopes are steeper than 2:1 this provision is not applicable.

This anchor assembly may be used as an approach end treatment for guardrail on any roadway. **Table 603-1** gives additional information on where to use this anchor assembly. See **MGS-4.5** for additional details.

603.3.2 Type B

The Type B anchor assembly is a flared, redirective, gating end terminal. For the Type B, the first 12.5 feet does not count toward length of need. The SRT-350 is installed with a curved flare while the FLEAT-350 uses a tangent flare, both with an offset of four feet. The Type B may be used as an approach end treatment for guardrail on any roadway. The Type B cannot be used when the back side of the device is in the clear zone of bidirectional traffic. The Type B products require a recovery area immediately behind the terminal detailed on **SCD MGS-5.2**. Designers should check that this grading is present on existing cross-slopes or otherwise revise the cross-slopes to conform. **Table 603-1** provides guidance on where to use this anchor assembly. See **Roadway Sample Plan Note R112a** in Appendix B for additional information. All products listed in this section are gating as described in ***Section 602.1.4***. These end treatments should connect to Type MGS guardrail, but it is acceptable to connect to Standard Bridge Terminal Assemblies.

The pay length and additional details for the Type B anchor assembly can be found under the approved products list on the Office of Roadway Engineering website.

An earlier version of the Type B known as the ELT or MELT depicted on Standard Drawings until 1994 is still found throughout the state highway system. This generic flared end terminal has not meet Report 350 criteria, and should be systematically replaced with the newer version.

603.3.3 Type E

The Type E anchor assembly is a tangent, redirective, gating end terminal. Because of varying system length, the length of need point should be detailed on the plans.

The Type E may be used as an approach end treatment for guardrail on any roadway. The Type E cannot be used when the back side of the device is in the clear zone of bidirectional traffic. The Type E products require a recovery area immediately behind the terminal detailed on **SCD MGS-5.3**. All products listed in this section are gating as described in ***Section 602.1.4***. These end treatments connect to Type MGS guardrail and to Standard Bridge Terminal Assemblies.

The terminal should be offset to minimize the potential for impacts caused by vehicles clipping the portion of the impact head that protrudes in front of the face of the guardrail. The preferred offset method is detailed on **SCD MGS-5.3**. The Type E should not be installed over a radius but may be installed with a 50:1 flare over the full length of the terminal or with a 25:1 flare over the first 25 feet of the device. **Table 603-1** gives guidance on where to use this anchor assembly. See **Roadway Sample Plan Note R113a** in Appendix B for additional information.

Table 603-1

Foreslope	New Construction / Major Reconstruction	3R, HSP and Bridge Replacement
6:1 or flatter	Buried-in- Backslope or Type B	Buried-in- Backslope or Type B
Steeper than 6:1 up to 4:1	Buried in Backslope or Type B	Buried in Backslope or Type E
Steeper than 4:1	Type E	Type E

603.3.4 Type A

The Type A anchor assembly (twisted turned-down end) is a non-proprietary, non-redirective end terminal. It is 25.0 feet long and may be used as an approach or trailing guardrail end treatment where a Type B or Type E anchor assembly is not feasible in any of the following situations:

1. On non-NHS arterials, collectors and local roads with a design average daily traffic volume of 2000 vpd or less.
2. On non-NHS roadway outside the clear zone.
3. On non-NHS roadway with an operating speed of less than 50 mph.

Since the LON point is at the rear of this device, no portion of the Type A can be included within the guardrail length of need. See **SCD MGS-4.1** for additional details.

603.3.5 Type T

The Type T anchor assembly is a non-proprietary, non-redirective end terminal that may be used on any roadway in any of the following situations:

1. On trailing ends of guardrail runs outside the clear zone of opposing traffic. Since the LON point is at the rear of this device, no portion of the Type T can be included within the guardrail length of need.

2. In guardrail runs where directional changes are made using a radius of less than 25 feet (see ***Figures 603-3*** and ***603-4***).

3. On the ends of guardrail runs on drive approaches (see ***Figure 603-3***).

The Type T is 12.5 feet long.
See ***SCD MGS-4.2*** for additional details.

603.4 Impact Attenuators

Impact Attenuators, also known as crash cushions, are generally used to shield motorists from rigid structures like bridge piers and end of concrete barriers. Since impact attenuators can be installed in two sided situations, they are well suited for median or gore applications. Refer to http://www.dot.state.oh.us/Divisions/ProdMgt/Roadway/roadwaystandards/Pages/default.aspx for links and shop drawings of approved proprietary products.

603.4.1 Type 1

Type 1 impact attenuators are redirective, gating, proprietary median guardrail terminal and crash cushion. Type 1's can be installed on any roadway in unidirectional and bidirectional configurations, but they must have at least 10 feet of clearance on both sides of the device. A maximum flare of 20:1 is permissible. Generally Type 1 Impact Attenuators are used in wider medians to safely end barrier design guardrail runs. See **Roadway Sample Plan Note R123** in Appendix B for approved products, specifications, and manufacturer's drawings.

Type 1 impact attenuators are gating systems before the LON of the system, but are redirective after that point. All systems are sacrificial, meaning they absorb impact kinetic energy by deforming the steel rail elements and/or breaking wood posts. Most of these systems are not reusable after an impact and most be replaced with new parts.

603.4.2 Type 2

Type 2 impact attenuators are reusable, redirective, non-gating proprietary systems that can be installed on any roadway in unidirectional and bidirectional configurations. Some of the major components of these crash cushions can be reused after an impact. It is important to note that if any of the components are damaged new parts will need to be installed during the repair in order to make the entire unit crashworthy.

Since the footprint for each product varies the designer should be specific about the available footprint, design speeds, and width of hazard. In some cases when there is a limited footprint available the designer should specify only the appropriate products. If cross slopes are steeper than 8 percent (12:1) or vary by more than 2 percent over the length of the unit, a leveling pad may be used.

Type 2 attenuators are ideal to protect the ends of rigid objects like concrete barrier ends. Some other uses could be for connection to guardrail runs in diverging gores or narrow medians, as well as temporary work zone locations. See **Roadway Sample Plan Note R124** for approved products, specifications, and manufacturer's drawings. Plan notes are in Appendix B.

603.4.3 Type 3

Type 3 impact attenuators are low-maintenance/self-restoring crash cushions typically considered for use at locations where high frequencies of impacts is expected. Maintenance is required with these units after impacts to restore full capacity for design impact conditions. These units could be cost-beneficial at locations with high frequency of impacts despite the higher initial costs because of the lower repair costs over the life of the product. These units are typically restored with minimal labor and replacement parts after a design impact.

Type 3 impact attenuators should be specified in lieu of Type 2 impact attenuators when a higher than normal impact frequency would be expected. Specifically at locations that have a history of being impacted more than once per year and at gores of urban systems interchanges as these high ADT weave areas have the highest potential for crash events. Type 3 impact attenuators are cost-effective when considering the benefits of faster and easier repair. Additionally, the safety benefits for maintenance personnel's exposure while repairing frequently damaged units cannot be discounted.

See **Roadway Sample Plan Note R125** in Appendix B for approved products, specifications, and manufacturer's drawings.

603.4.4 Work Zone Impact Attenuators

All Type 2 and Type 3 impact attenuators are considered acceptable for use in temporary work zones. Additional products specifically listed in this category are approved only for temporary work zones to protect hazards 24" and smaller, and some products can be beneficial in locations where foundations and anchors are not required. Typically considered to be sacrificial units, the impact attenuators that are permitted for work zones only are crashworthy roadside safety devices designed for a single impact, usually protecting the end of temporary barriers. Most of these temporary systems are gating, non-redirective, and absorb impact energy through crushing the product elements. These systems' major components are destroyed in an impact and must be replaced. Refer to the Traffic Engineering Manual Sections 642-30 and 642-31 for additional design requirements.

603.4.5 Sand Barrels

Sand barrel arrays are proprietary sand-filled modules of varying sizes arranged in a pattern designed to protect wide hazards. Sand barrel arrays are appropriate in limited situations for the protection of wide hazards when no other product is acceptable. Because each product is different a specific design layout is required for each location based on the design speed and width of the hazard being shielded. All arrays installed on the NHS must meet NCHRP Report 350 Test Level 3 criteria.

603.5 Bridge Terminal Assemblies

When a less rigid barrier is to be connected to a more rigid barrier, a stiffening transition is needed to make the connection. A transition from a more rigid barrier to a less rigid barrier doesn't require any stiffening unless the barrier can be struck from the opposite direction. Even when the difference in strength is not an issue, a transition is frequently required simply to connect two barrier systems that have different hardware components. Transitions in Ohio are called Bridge Terminal Assemblies because they are typically required where guardrail is warranted in conjunction with bridge parapets/railings. They are also used to connect guardrail to concrete barrier and other similar fixed objects.

603.5.1 Type 1

The Bridge Terminal Assembly, Type 1 is commonly used to connect guardrail to a concrete barrier or a concrete bridge parapet. It uses blocked-out, nested, thrie-beam guardrail panels attached to a vertical concrete surface to transition to the guardrail. The addition of a curb under the stiffer thrie-beam transition panel enables the assembly to meet into TL-3 when connecting to the concrete barrier or parapet. Curb is not required when connecting to Twin Steel Tube Bridge Rail.

It is generally installed at the following locations:

1. At the approach end of a rigid object.
2. At the trailing end of a rigid object if it is located within the clear zone of opposing traffic.
3. To connect Type MGS Guardrail to Twin Steel Tube Bridge Rail

Where designs require the upstream guardrail to be used in conjunction with curb for drainage purposes, the section 25 ft. immediately prior to this transition assembly should be without curb. See **SCD MGS-3.1** for additional details.

603.5.2 Type 1 Barrier Design

The Bridge Terminal Assembly, Type 1: Barrier Design is commonly used to connect barrier design guardrail or a Type 1 Impact Attenuator to a concrete median barrier. It uses blocked-out, nested, thrie-beam, guardrail panels attached to a vertical face on both sides of the barrier to transition to the guardrail or attenuator. As with the Type 1, the curb and stiffer thrie beam transition panels enables the assembly to meet into TL-3.

See **SCD MGS-3.1** for additional details.

603.5.3 Type 2

The Bridge Terminal Assembly, Type 2 is commonly used to connect guardrail to the trailing end of a concrete barrier or bridge parapet located outside the clear zone of opposing traffic. It uses standard w-beam guardrail panels attached to a vertical face on the concrete barrier to transition to the guardrail. When used as a trailing end assembly, it can be used on the NHS. See **MGS-3.2** for additional details.

603.5.4 Previous Types 3 & 4

Refer to the Location & Design Manual dated July 20, 2012 for transitions to Thrie Beam or DBR Bridge Railing (old Types 3 & 4).

603.6 Concrete Barrier End Treatment

The end of a concrete barrier may be a hazard if not treated properly. Since a rigid barrier generally does not require end anchorage to develop its strength, the simplest means of providing impact protection for the barrier end may be to terminate the barrier beyond the clear zone. When this approach is used, the flare rate used to offset the barrier should not exceed the flare rates recommended in ***Figure 602-1***. However, when the end of a concrete barrier is located within the clear zone, a terminal is necessary to protect a vehicle's occupants in an end-on impact.

Acceptable end treatments include the following:

1. Transition to guardrail using a bridge terminal assembly and terminate the end of the guardrail run with an anchor assembly.

2. Use an impact attenuator as discussed in ***Section 603.4***.

3. Terminate the concrete barrier directly into a cut backslope.

4. Use a tapered end section only: (1) when the barrier is terminated outside the clear zone (See ***Figure 603-5***), or (2) when the barrier is on a non-NHS road with a design speed less than or equal to 40 mph (NCHRP Report 350 TL-2) and space is limited by right-of-way constraints or presence of other roadside features that preclude the use of an approved end treatment.

604 GLARE SCREEN

Glare screen is used primarily for the shielding of motorists from headlight glare of opposing traffic. It is normally used in the median of divided highways but may be used in other areas where a specific problem exists or is anticipated.

There are locations, other than in the median, where glare screen may be justified. An example would be between a parallel facility and the mainline where geometrics or unusual sources of light cause a glare problem.

604.1 Median Glare Screen

Glare screens should be provided when concrete barrier is used to separate opposing traffic on interstates and freeways. Median glare screen may also be justified when glare problems are experienced on isolated sharp curves. Median glare screen installation should be as continuous as practical. Gaps of approximately 1 mile or less in length should be avoided.

604.3 Glare Screen Options

Glare screening may be accomplished in a number of ways. These include, but are not limited to, the following options (shown in order of preference):

1. Use a taller standard barrier. For example use Type B1 in lieu of Type B concrete barrier.

2. On a NJ shape barrier, install a concrete cap to extend the height of existing 32 inch concrete barrier where barrier thickness is adequate.

3. Attach a paddle or intermittent type of glare screen to the top of a 42 inch Single Slope or 32 inch tall NJ shape concrete barrier, or on top of steel beam guardrail. These devices shall be designed using a 20-degree cut-off angle measured relative to the centerline of the barrier. They shall be securely fastened to the barrier using the hardware and procedures specified by the manufacturer. Contact the **Office of Materials Management** for a list of approved manufacturers.

Options 1-3 may only be used in locations where barrier is required.

605 RUMBLE STRIPS

605.1 Shoulder Rumble Strips

A shoulder rumble strip is a pattern of grooves or depressions made in paved highway shoulders, by milling or grinding, to produce an audible and/or vibratory warning to drivers whose vehicles have drifted off the traveled way.

SCD BP-9.1 contains design details and options for the placement of rumble strips on shoulders. Shoulder rumble strips have proven to be effective in reducing run-off-the-road accidents due to driver inattention, monotony and fatigue. They also may serve as an audible form of roadway edge delineation in adverse weather conditions. Rumble strips are most appropriate for use on higher speed facilities where access is controlled through interchanges or widely-spaced intersections (several miles apart) and are also appropriate for other roadways with a history of run-off-the-road accidents due to driver inattention.

605.1.1 Locations

Shoulder rumble strips will be installed at the following locations:

1. On new, reconstructed, and resurfaced shoulders of all rural fully access-controlled highways (Interstates, freeways, and expressways).

2. On any other multi-lane roadway with a speed limit greater than 45 mph, at the District's discretion.

Shoulder rumble strips should be considered at the following locations:

1. At certain critical locations, such as: in gore areas, ahead of impact attenuators and next to concrete median barriers.

Shoulder rumble strips may be installed at the following locations:

1. On local roads and streets in Federal-aid projects that are not on the NHS, at the discretion of the responsible local agency. (See **SCD BP-9.1** for additional details on the location of shoulder rumble strips.)

605.1.2 Section Not Used

605.1.3 Lateral Clearances for Machinery

The machinery used in the milling process to construct rumble strips requires a lateral clearance of at least 34 inches from the outside edge of the pattern to any obstruction (guardrail, a barrier, curbs, etc.).

605.1.4 Divided Highways

Rumble strips should be installed on both shoulders (right and left) of divided roadways, but individual circumstances may dictate use on only one shoulder.

605.1.5 Existing Shoulders

Rumble strips should only be installed on existing paved shoulders that are in good condition and have a width of 2.5 feet. Where existing shoulders are resurfaced, the existing rumble strip pattern shall be restored on the new shoulder in accordance with this manual and the Pavement Design Manual.

605.1.6 Bicycle Considerations

Rumble strips generally should not be used on the shoulders of roadways designated as bicycle routes or having substantial volumes of bicycle traffic, unless the shoulder is wide enough to accommodate the rumble strips and still provide a minimum clear path of 4 feet from the rumble strip to the outside edge of the paved shoulder or 5 feet to adjacent guardrail, curb or other obstacle.

In areas designated as bicycle routes or having substantial volumes of bicycle traffic, the rumble strip pattern should not be continuous but should consist of an alternating pattern of gaps and strips, each 12 feet and 48 feet respectively in length. Also, gaps should be provided in the rumble strip pattern ahead of intersections, crosswalks, driveway openings, and at other locations where bicyclists are likely to cross the shoulder.

605.1.7 Residential Areas

In residential areas, noise generated by rumble strips could be objectionable. Rumble strips installed in these areas may be placed further from the edge of the traveled lane than shown on **SCD BP-9.1** to reduce the frequency of contact while still providing some degree of warning to drifting drivers.

The distance from the edge of the traveled lane to the rumble strip pattern should not exceed 2.0 feet on the outside shoulder.

605.1.8 Maintenance of Traffic

Where shoulders are to be used for maintenance of traffic purposes, rumble strips should be positioned to adapt to phased construction sequencing. See **SCD BP-9.1**.

605.2 Rumble Strips Across Traveled Lanes

Rumble strips in traveled lanes are used to alert drivers of unusual or unexpected traffic conditions or geometrics and to bring the driver's attention to other warning devices. They are not intended for traffic calming and they should only be installed after all other appropriate standard traffic control devices have been utilized and have failed to resolve the problem satisfactorily.

Rumble strips are most effective when they surprise motorists enough to catch their attention. For this reason, they should be used sparingly. (See ***Section 605.2.1*** for typical locations.)

605.2.1 Locations

Typical locations for the installation of rumble strips in a traveled lane are at the following:

1. Rural stop approaches with high accident rates.
2. Signalized intersections with high accident rates.
3. Short exit ramp deceleration lanes or hidden intersections.

Other possible locations include:

1. Locations with abrupt changes in horizontal alignment.
2. Intersections with inadequate stopping sight distance caused by vertical or horizontal alignment.
3. Railroad crossings with sight distance restrictions and a history of accidents.
4. Approaches to toll booths and narrow bridges.
5. At the approach to work zones and at other locations within the work zone.

606 FENCE

606.1 Purpose

Highway fences are a part of the highway facility and are placed within the right-of-way limits of highways having controlled or limited access right-of-way. They act as physical barriers to enforce observance of the acquired access rights. The State or other agency responsible for the maintenance of the facility shall assume the responsibility for the maintenance of these fences.

606.2 Types

It is ODOT's policy to construct only the standard types of fence described below in accordance with the current Standard Construction Drawings and Construction and Material Specifications.

TYPE 47 - Woven wire fence with a 47-inch fabric, steel line posts, and one strand of barbed wire on the top. (See **SCD F-2.1**)

TYPE 47RA - Woven wire fence with 47-inch fabric, wood line posts, and no barbed wire. (See **SCD F-2.1**)

TYPE CLT - Chain link fence with 60-inch fabric but with a tension wire in lieu of the top rail. (See **SCD F-1.1**)

606.3 Fence on Freeways

606.3.1 Urban Freeways

Urban freeways shall be continuously fenced. Innerbelts and radials shall use Type CLT. Outerbelts shall use Types CLT or 47 depending upon the adjacent land use.

606.3.2 Rural Freeways

Rural freeways shall be continuously fenced, usually with Type 47 fence; however, Type CLT may be used in areas where there are schools, subdivisions or other developments.

606.3.3 Freeway Fence Design Conditions

1. Where chain link fence is located within the design clear zone, such as along the edge of a roadway shoulder, in a median, or between a frontage road and the mainline, a fence with tension wire, Type CLT, shall be used.

2. Type 47RA fence shall be used to fence rest areas where the highway fence is Type 47. It may also be used in other locations where the aesthetics of the area make this type more desirable.

3. Fence installed across a stream or ditch shall be designed using fence terminals or crossings as shown in SCD F-3.3 and F-3.4, respectively.

4. Where a drainage channel is located parallel to the freeway in a channel easement, the fence shall be located on a bench between the main facility and the channel. Maintenance openings shall be provided at 700 feet maximum intervals where the length of fence between a deep channel and the freeway exceeds 1800 feet, unless access can be provided by another means.

5. Fence shall be provided in the median to connect the abutments of all twin bridges on divided highways.

6. All types of fence shall be grounded where a power line passes over them. Fence shall also be grounded where a parallel power line easement is within 50 feet of the fence. For grounding details see **SCD HL-50.11**.

7. In the vicinity of some airports, fencing should be non-metallic since it sometimes interferes with airport traffic control radar. The **Federal Aviation Administration** should be contacted to ascertain if metallic fencing will be a problem.

8. Fence should normally be continued behind a noise wall. Sufficient distance should be provided between the fence and the noise wall to permit normal maintenance operations. If there is no critical maintenance responsibility between the noise wall and the right-of-way or limited access line (generally in "cut" sections) the fence may be terminated at each end of the noise wall.

606.3.4 Exceptions to Continuous Freeway Fencing

1. Fence shall be terminated with an end post assembly at an existing ½:1 or steeper slope, measured along the fence centerline. However, if the ground approaching the ½:1 slope is too steep to allow proper fence installation, a Type E fence terminal shall be installed at the edge of the slope. (See ***Figure 606-1 (a)*** and ***(b)*** for details.)

2.Fence shall be terminated in a cut section that exceeds 30 feet in vertical height with a backslope of ½:1 or steeper. An End Post Assembly and a Type E fence terminal shall be located as shown in ***Figure 606-1(c).***

3. Where the fence intersects a crossroad right-of-way line at interchanges, it shall be constructed along the crossroad to the limits of the limited access right-of-way.

606.4 Fence on Arterials

606.4.1 Urban Arterials

Fence shall be Type CLT or Type 47 depending upon the adjacent land use. Type CLT should be used where there is any doubt that Type 47 would be adequate to prohibit undesired intrusions.

606.4.2 Rural Arterials

Fence should normally be Type 47.

606.4.3 Arterial Fence Design

Fence shall be provided along the limited access right-of-way line on arterials but shall terminate at the end of limited access right-of-way at crossroads or railroads, and at stream banks and driveways. Fence shall be omitted where the highway right-of-way adjoins lateral features which would prevent vehicular access, such as: railroads, streams, deep ditches, swamps, strip mines or other steep slopes. Type CLT and 47RA shall be used on arterials in the same locations as described for freeways in ***Section 606.3.3*** **(1)** and **(3).**

606.5 Fence on Collectors

Fencing of limited access right-of-way on urban or rural Collectors (or lower classifications) with partial access control will be determined on an individual project basis using arterial requirements as a guide.

606.6 Lateral Location of Fence

Section 607.06 of the **Construction and Material Specifications** gives line post and fence location as related to the right-of-way line. Normally, woven wire fence should be placed 2.0 feet inside the right-of-way line and chain link 1.0 feet

When viewed at a flat angle, chain link fencing restricts sight distance. This should be considered when placing fence in interchange areas and intersections.

606.7 Fence Approval

Determination of the type and extent of fencing will be made during the development of the contract plans and will be completed in time for the Stage 3 review.

606.8 Bridge Vandal Protection Fence

For policy and details of vandal protection fence, see the **Bridge Design Manual** and **SCD VFP-1-90**, both published by the **Office of Structural Engineering.**

LIST OF FIGURES

Sample Calculations

CLEAR ZONE WIDTHS

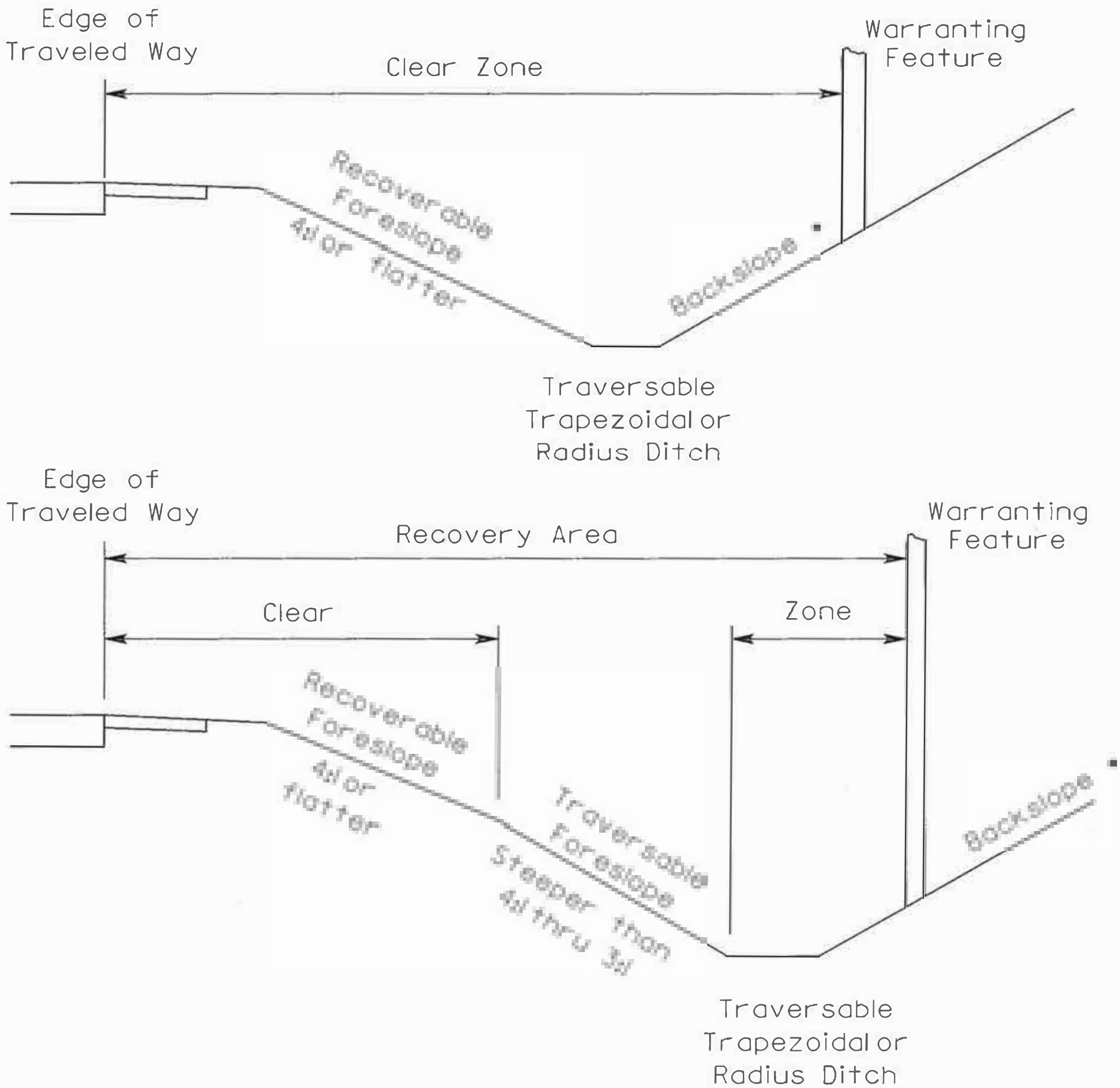
600-1
REFERENCE SECTIONS 600.2

Design Speed	Design ADT	Foreslope		Backslope		
		6:1 or Flatter	Steeper than 6:1 to 4:1	6:1 or Flatter	Steeper than 6:1 to 4:1	Steeper than 4:1
40 mph or less	<750	8 ft	8 ft	8 ft	8 ft	8 ft
	750-1500	11	13	11	11	11
	1501-6000	13	15	13	13	13
	>6000	15	17	15	15	15
45-50 mph	<750	11	13	11	9	9
	750-1500	13	18	15	13	11
	1501-6000	17	23	17	15	13
	>6000	19	26	21	19	15
55 mph	<750	13	16	11	11	9
	750-1500	17	22	17	15	11
	1501-6000	21	27	21	17	15
	>6000	23	29	23	21	17
60 mph	<750	17	22	15	13	11
	750-1500	22	29	21	17	13
	1501-6000	28	36*	25	21	16
	>6000	31*	40*	27	25	21
65-70 mph	<750	19	23	15	15	11
	750-1500	25	32*	21	19	14
	1501-6000	30	38*	27	23	18
	>6000	32*	42*	28	28	23

* Use a **maximum clear zone** of 30 feet unless a site specific investigation or accident history indicates a high potential of continuing accidents. When the potential for continuing accidents is high, the widths in the above chart should be multiplied by the following curve correction factors to extend the clear zone on the outside of curves having a Degree of Curvature of 2 degrees or sharper.

Degree of Curvature	HORIZONTAL CURVE CORRECTION FACTORS						
	Design Speed (mph)						
	40	45	50	55	60	65	70
2.0	1.1	1.1	1.1	1.2	1.2	1.2	1.3
2.5	1.1	1.1	1.2	1.2	1.2	1.3	1.3
3.0	1.1	1.2	1.2	1.2	1.3	1.3	1.4
3.5	1.1	1.2	1.2	1.3	1.3	1.4	1.5
4.0	1.2	1.2	1.2	1.3	1.4	1.4	
4.5	1.2	1.2	1.3	1.3	1.4	1.5	
5.0	1.2	1.2	1.3	1.4	1.5		
6.0	1.2	1.3	1.4	1.4	1.5		
7.0	1.3	1.3	1.4	1.5			
8.0	1.3	1.4	1.5				
9.0	1.3	1.4	1.5				
10.0	1.4	1.5					
15.0	1.5						

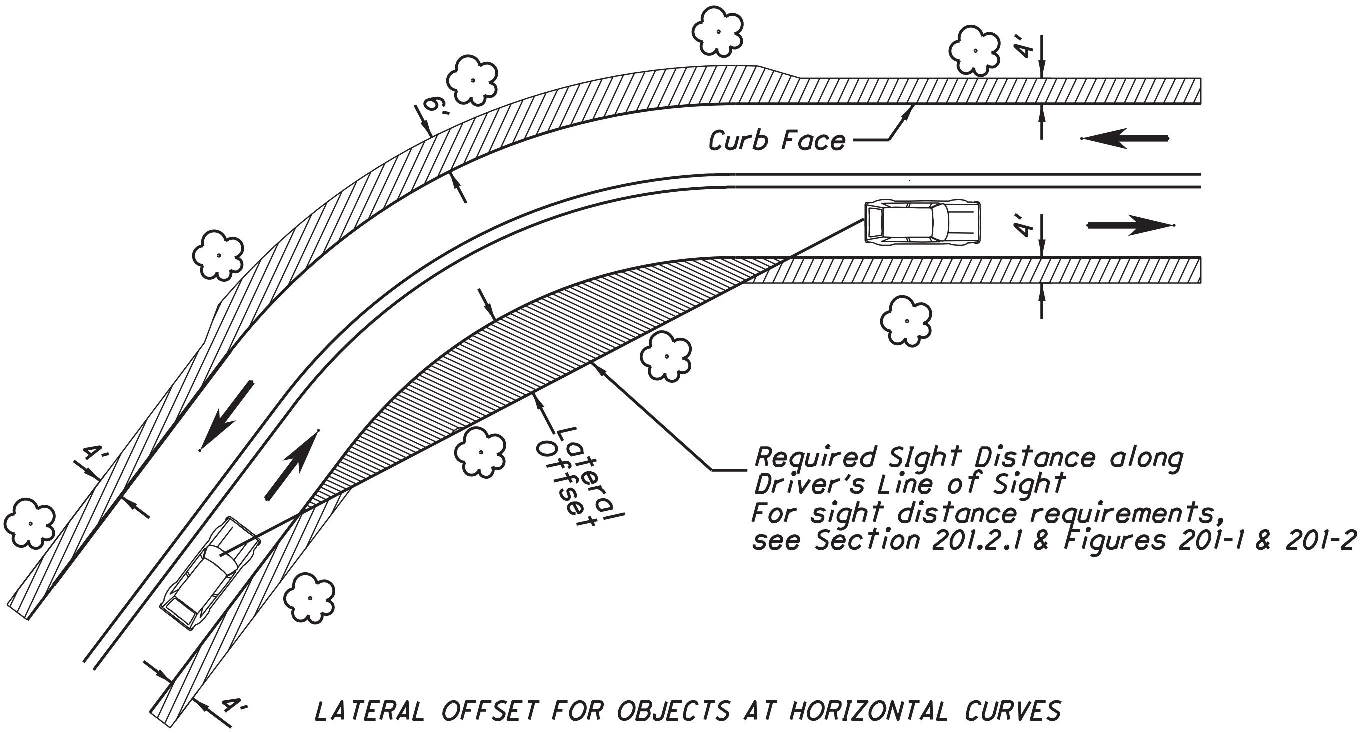
CLEAR ZONE MEASUREMENTS	600-2
	REFERENCE SECTIONS 600.2.1



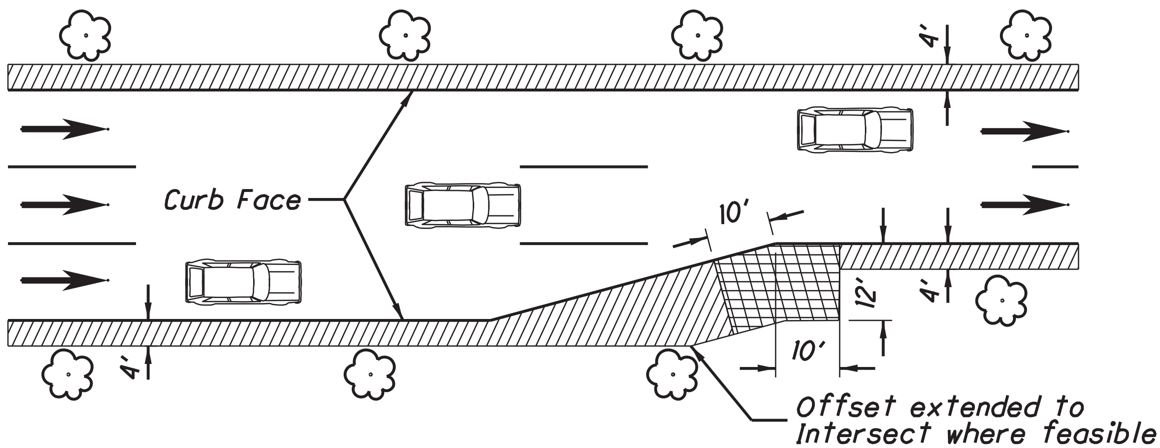
* For acceptable foreslope and backslope combinations that produce traversable trapezoidal and radius ditches, see Figures 307-3 and 307-2, respectively.

For clear zone widths, see Figure 600-1.
For 3R projects, see Section 906.1.

URBAN LATERAL OFFSETS AT HORIZONTAL CURVES AND MERGE POINTS	**600-3**
	REFERENCE SECTION 600.2.2

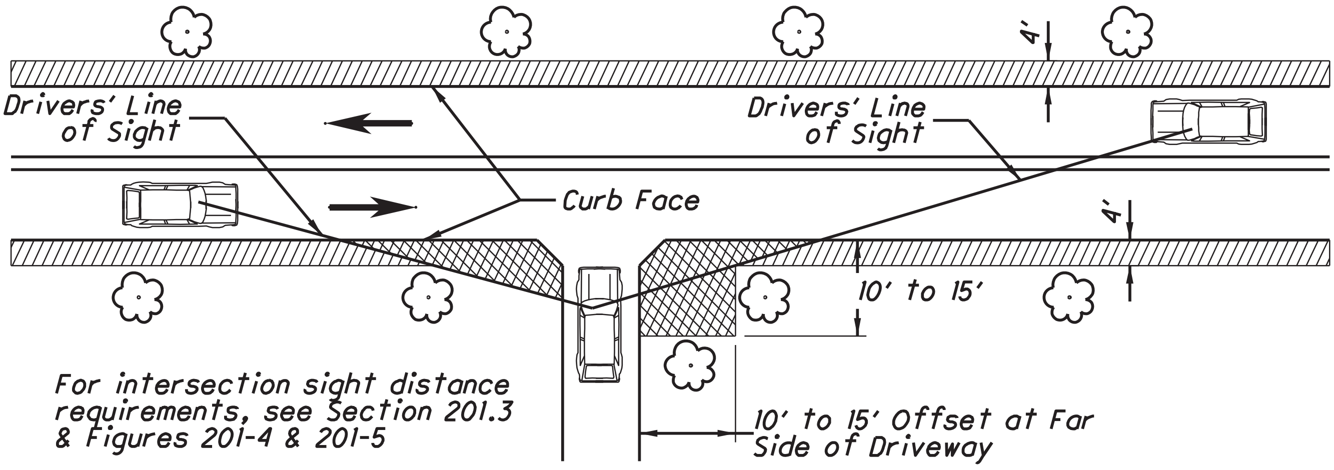


LATERAL OFFSET FOR OBJECTS AT HORIZONTAL CURVES

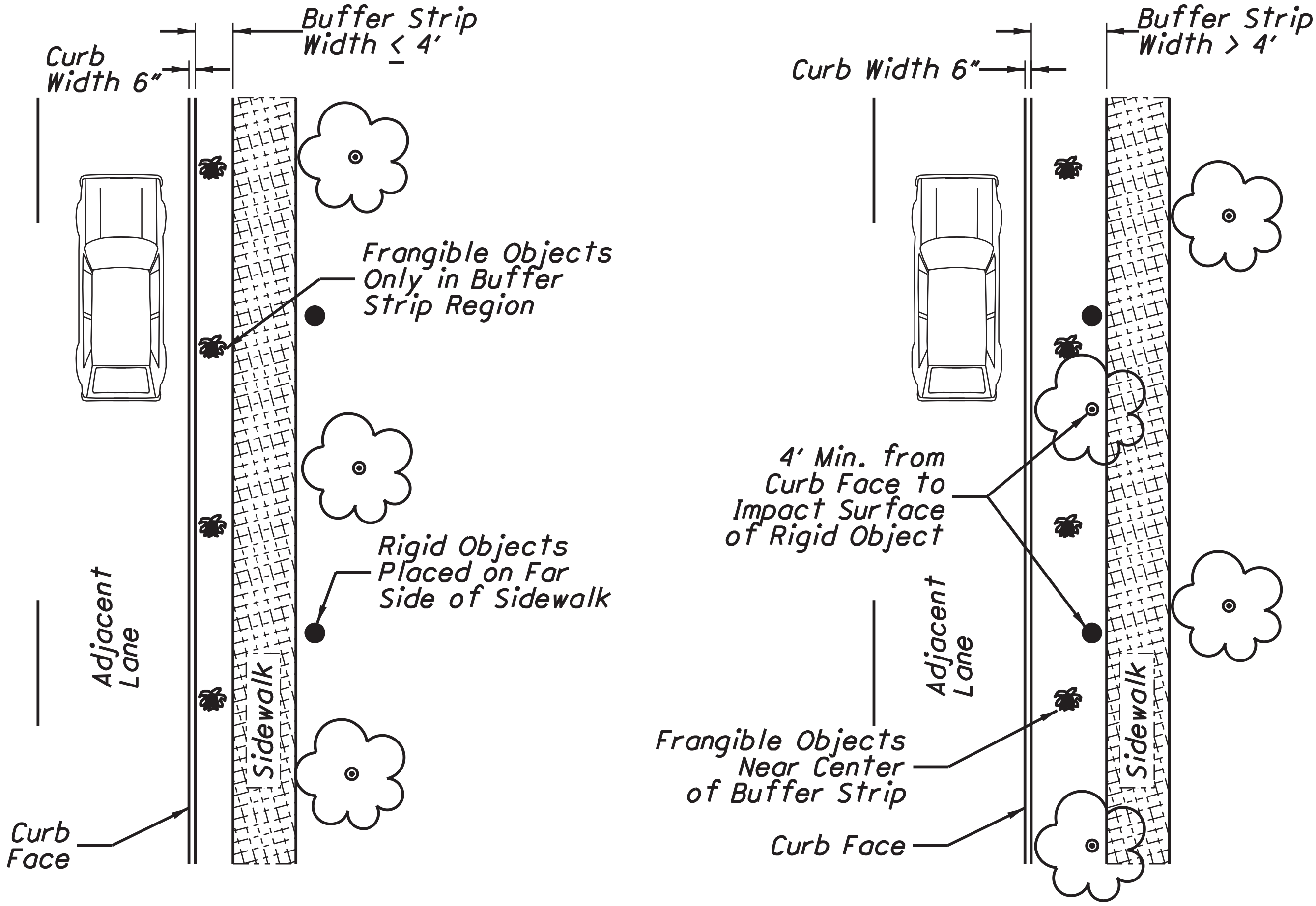


LATERAL OFFSET CONFIGURATION APPLIES TO LANE MERGES, ACCELERATION LANES, AND BUS BAY RETURNS

URBAN LATERAL OFFSETS AT DRIVEWAYS AND SIDEWALK BUFFER STRIPS	600-4
	REFERENCE SECTION 600.2.2

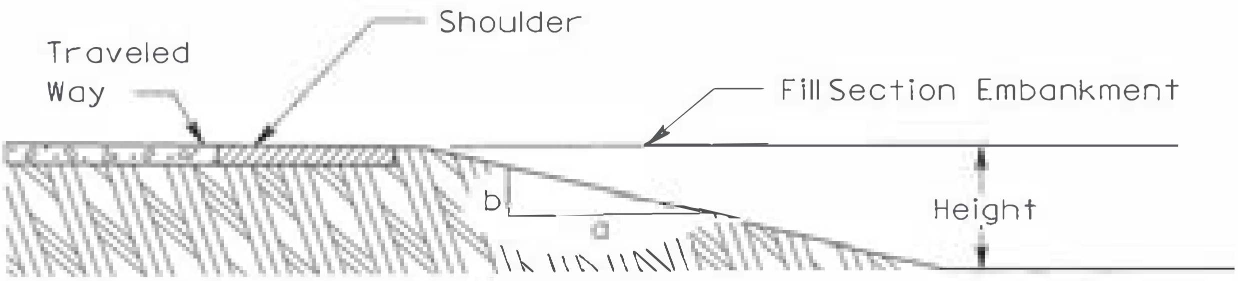


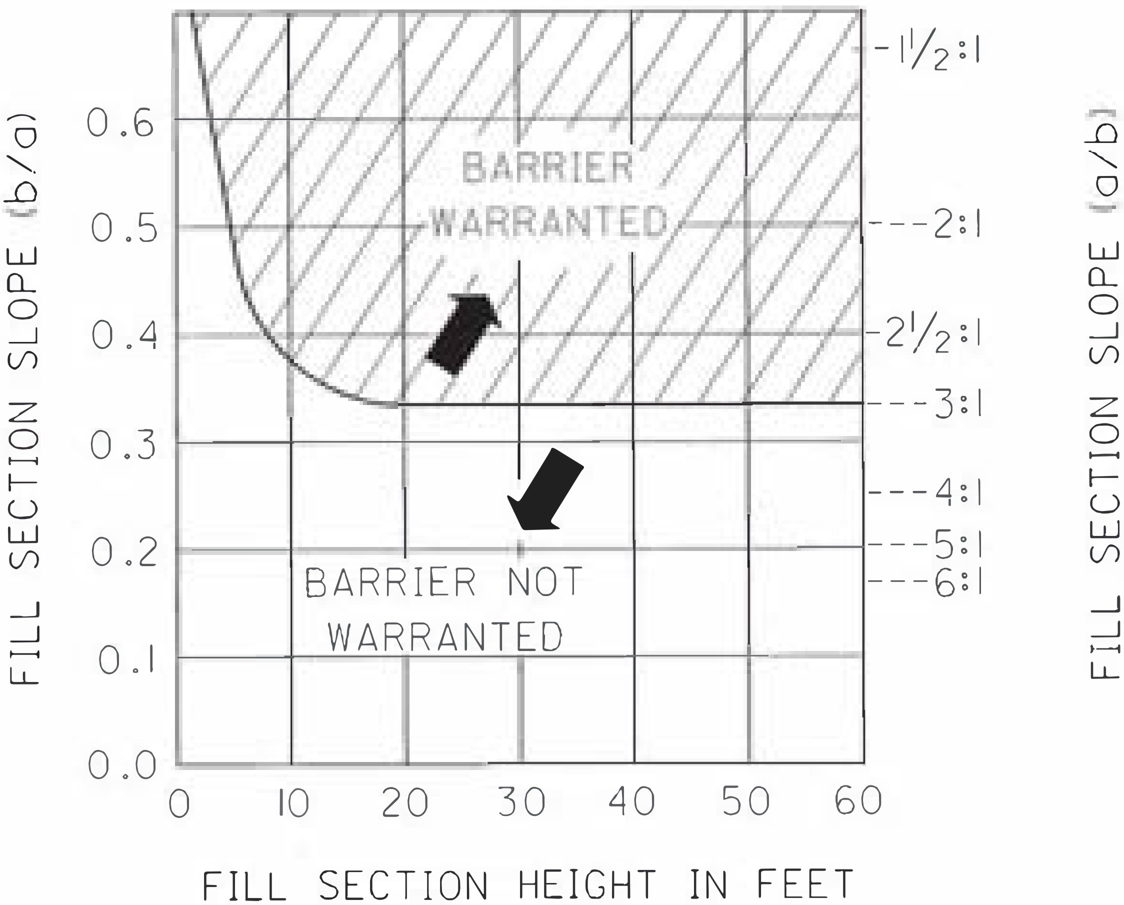
ROADSIDE LATERAL OFFSETS AT DRIVEWAYS



LANDSCAPE AND RIGID OBJECT PLACEMENT FOR BUFFER STRIPS

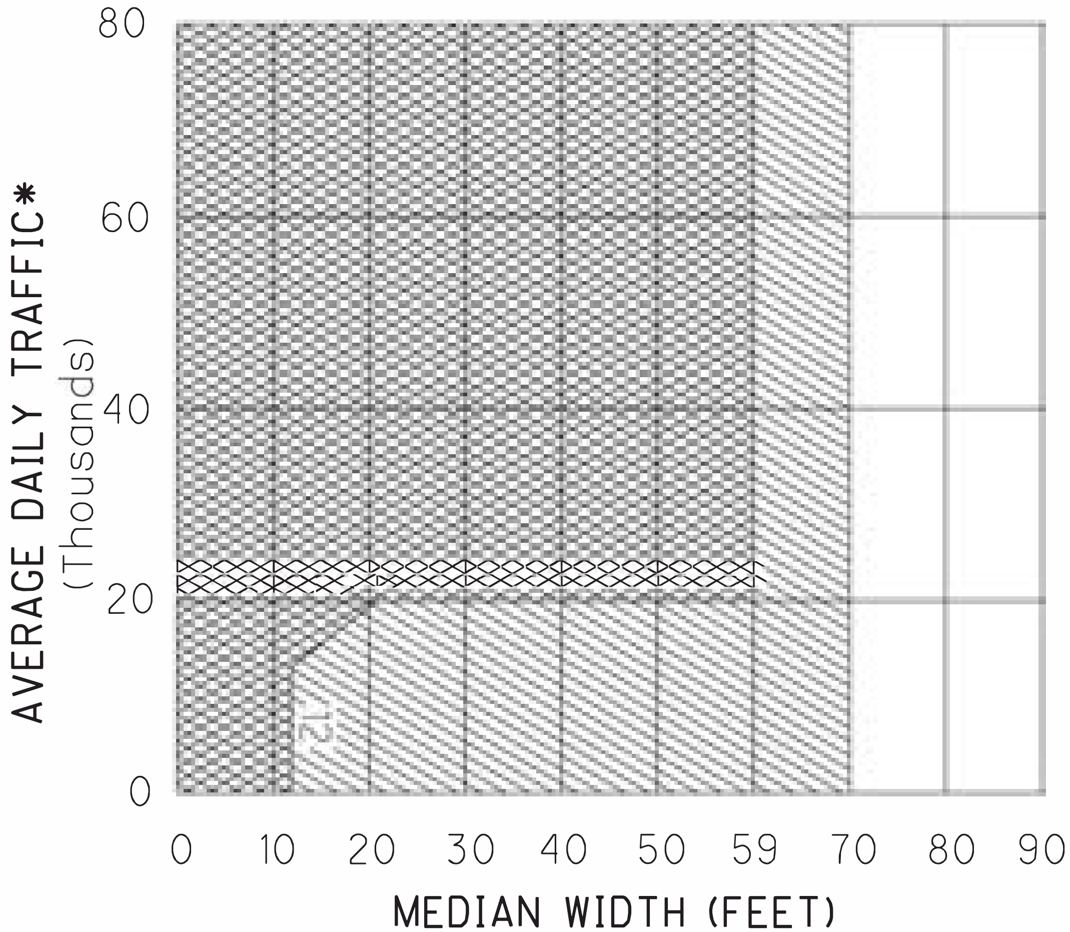
BARRIER WARRANTS FOR EMBANKMENTS





On or below the curve barrier is not warranted for embankment. However, check barrier need for other roadside hazards within the clear zone.

MEDIAN BARRIER WARRANTS	601-2
	REFERENCE SECTIONS 601.2



Warrants for median barriers on freeways

* Based on a 5-year projection

BARRIER OPTIONAL

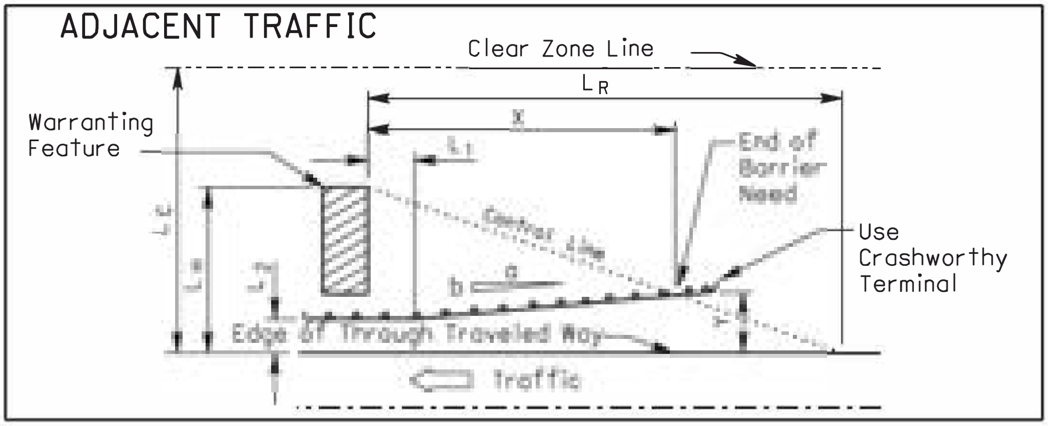
EVALUATE NEED FOR BARRIER

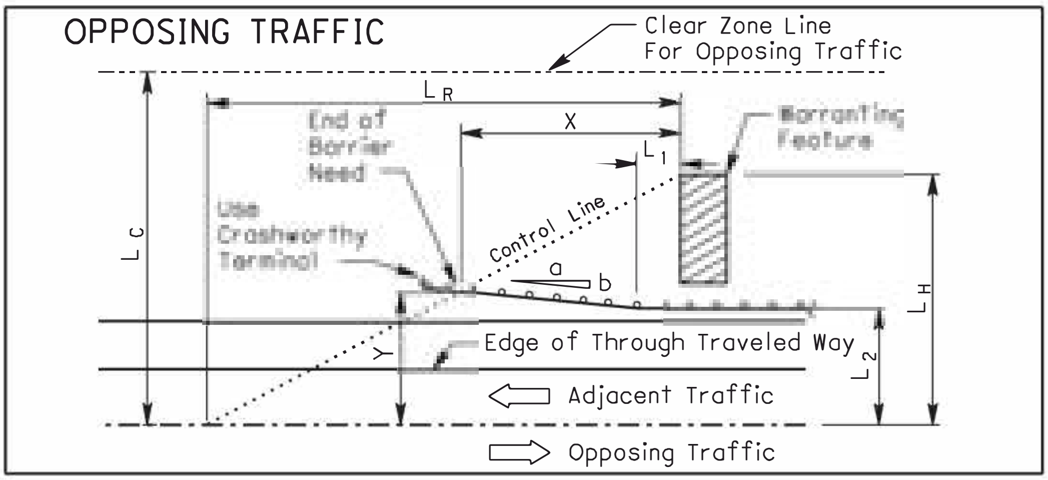
BARRIER NOT NORMALLY CONSIDERED

BARRIER LENGTH OF NEED (TANGENT ALIGNMENT)

602-1

REFERENCE SECTIONS
602.1.2, 602.5.1, 603.6





Design Speed (mph)	Flare Rate (a:b)		Runout Length, L_R (ft) Per Design Year ADT			
	Concrete Barrier	MGS Guardrail	Over 10,000 veh/day	5,000-10,000 veh/day	1,000-5,000 veh/day	Under 1,000 veh/day
75	20:1	7:1	415	380	335	290
70	20:1	7:1	360	330	290	250
65	19:1	7:1	330	290	250	225
60	18:1	7:1	300	250	210	200
55	16:1	7:1	265	220	185	175
50	14:1	7:1	230	190	160	150
45	12:1	7:1	195	160	135	130
40	10:1	7:1	160	130	110	100
35	9:1	7:1	135	110	95	85
30	8:1	7:1	110	90	80	70

Formulas

X = Length of Need
L_R = Runout Length
L_C = Required Clear Zone
L_H = Lateral Offset to Back of Warranting Feature
L_2 = Lateral Offset to Face of Barrier (see Figure 301-3)
L_1 = Varies (Typically measured to the end of a full panel of guardrail)

If $L_H < L_C$: $X = \dfrac{L_H + (b/a)L_1 - L_2}{(b/a) + L_H/L_R}$

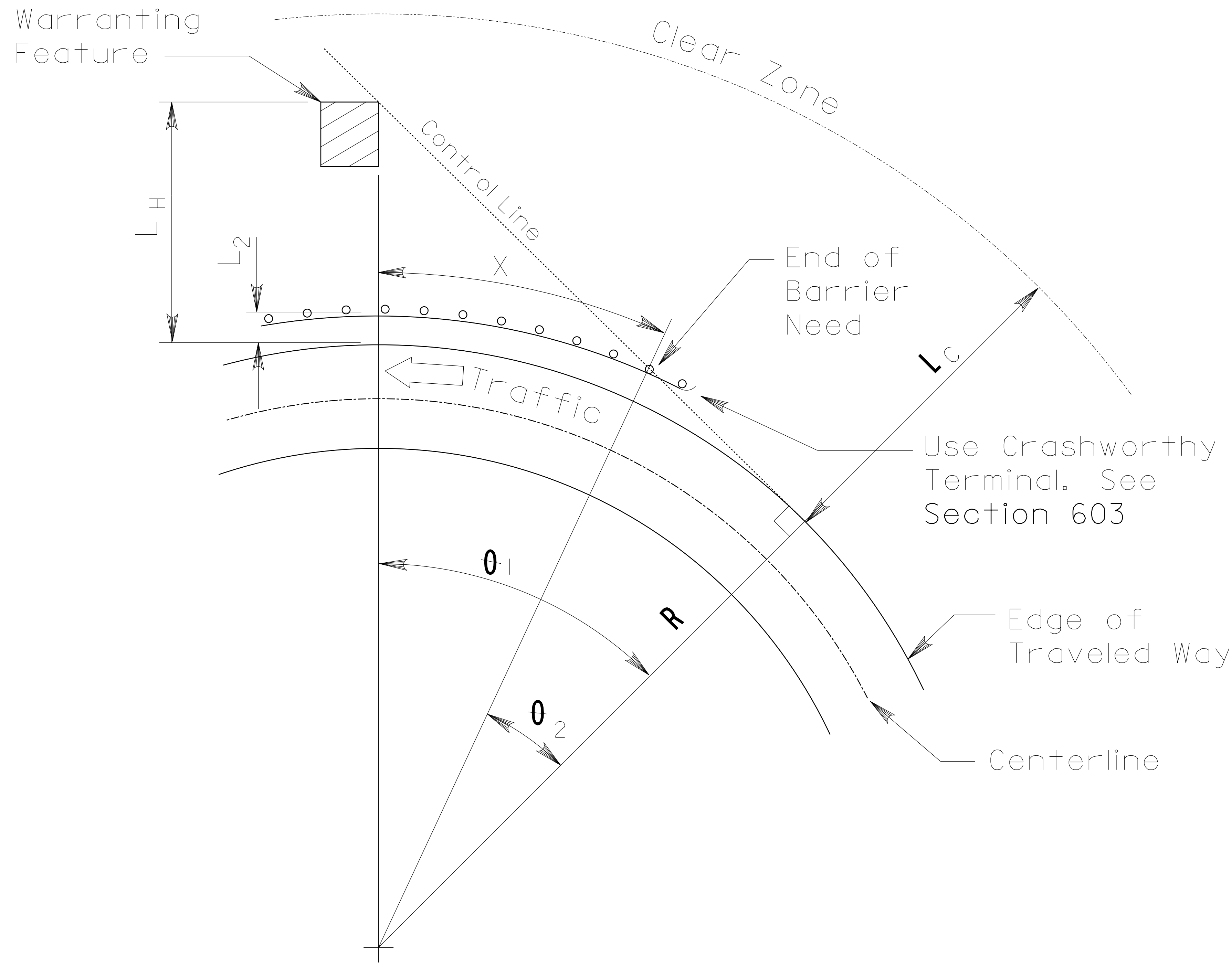
$Y = L_H - X(L_H/L_R)$

If $L_H > L_C$: Substitute L_C in the above formulas.

BARRIER LENGTH OF NEED (CURVED ALIGNMENT)

602-2E

REFERENCE SECTIONS
602.1.3, 602.5.1, 603.6



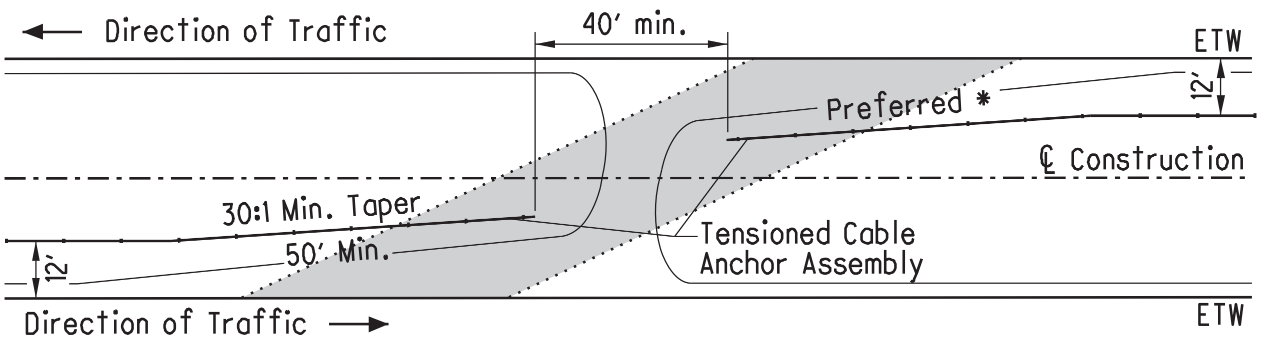
Formulas
X = Length of Need L_C = Required Clear Zone L_H = Lateral Offset to Back of Warranting Feature L_2 = Lateral Offset to Face of Barrier (See Figure 301-3, Guardrail Offset) If $L_H < L_C$: $X = (R+L_2)(\theta_1-\theta_2)$radians, where $\theta_1 = \cos^{-1}(R/(R+L_H))$ and $\theta_1 = \cos^{-1}(R/(R+L_2))$ $R = 5729.58/Dc$, where Dc = decimal degree of curve 1 degree $= \pi/180$ radians If $L_H > L_C$: Substitute L_C in the above formulas

January 2020

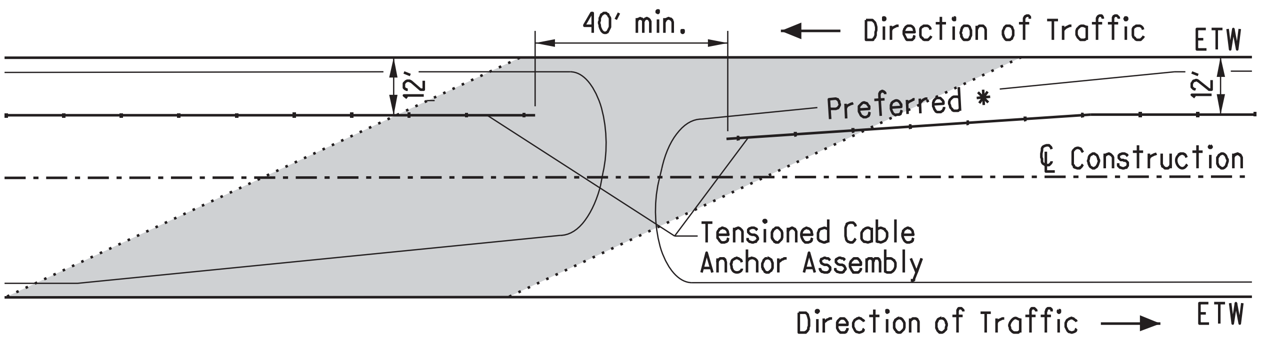
TENSIONED CABLE PLACEMENT AT U-TURNS

602-3a
REFERENCE SECTIONS 602.2.2, 603.1.1

Edge of Traveled Way : ETW

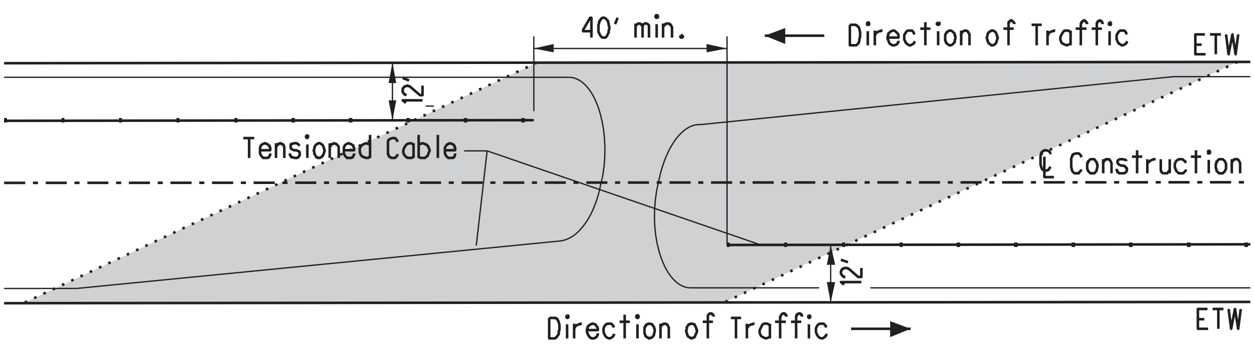


OPTION A - PREFERRED



OPTION B - ACCEPTABLE

Shaded area represents the effective gap between cable ends at the U-Turn Locations Option C Should be avoided where possible unless site constraints prohibit Options A & B



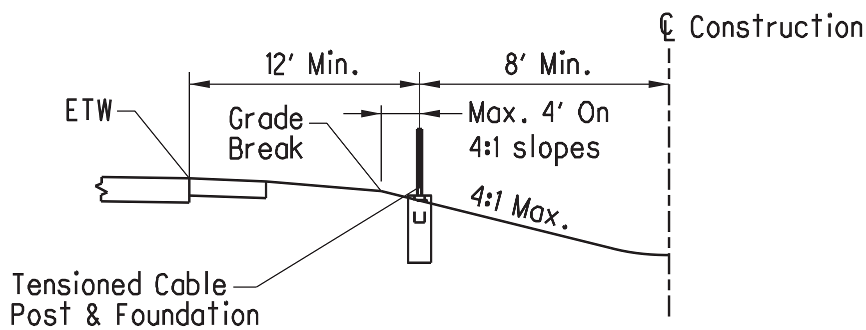
OPTION C - NOT PREFERRED

TENSIONED CABLE MEDIAN PLACEMENT

602-3b

REFERENCE SECTIONS
602.2.2, 603.1.1

Edge of Traveled Way : ETW

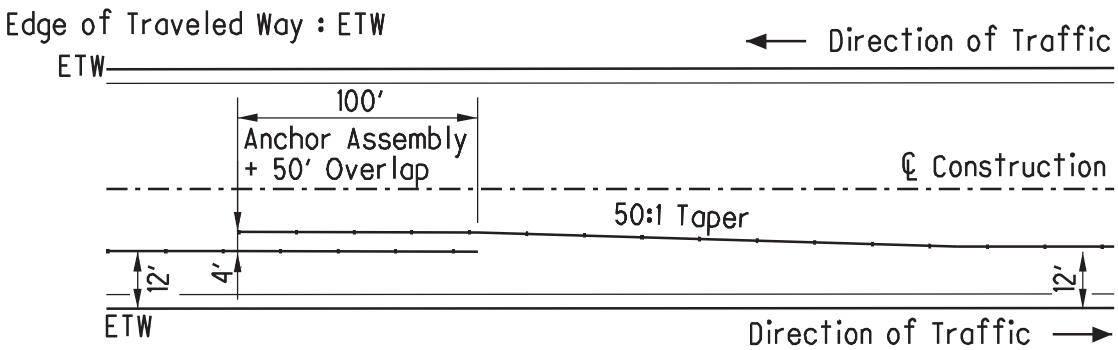


TENSIONED CABLE TYPICAL CROSS-SECTION PLACEMENT

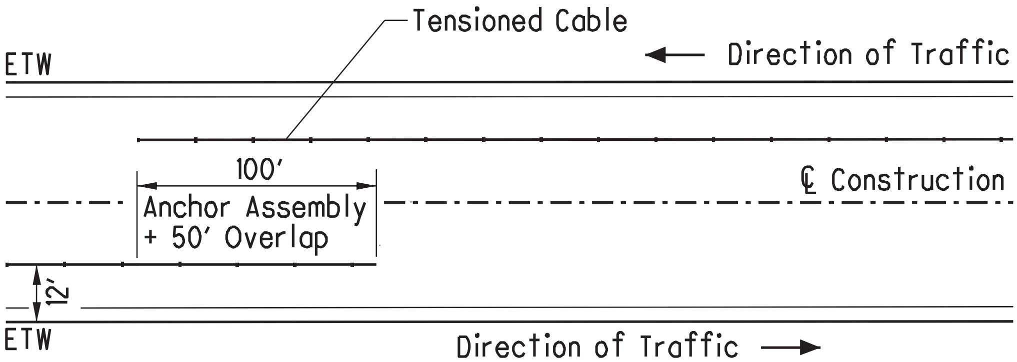
OVERLAPPING RUNS OF TENSIONED CABLE

602-4a

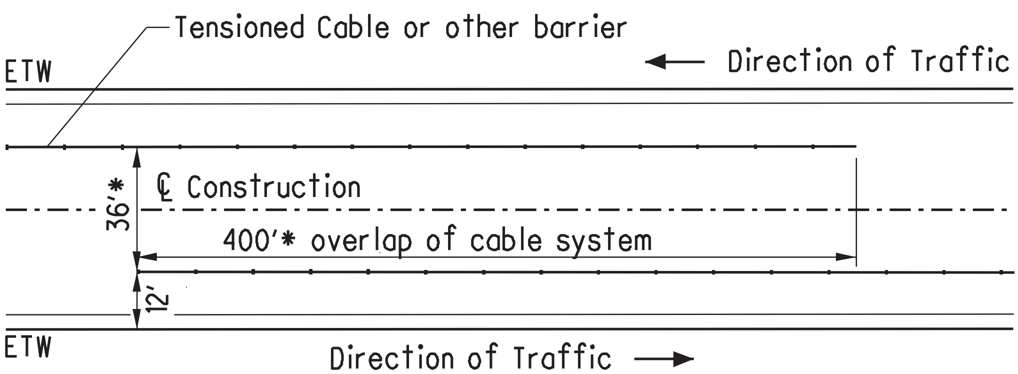
REFERENCE SECTIONS 602.2.2, 603.1.1



TENSIONED CABLE ANCHOR ASSEMBLY OVERLAP (on same side of median)



TENSIONED CABLE ANCHOR ASSEMBLY OVERLAP
(barrier runs on opposite sides of median)



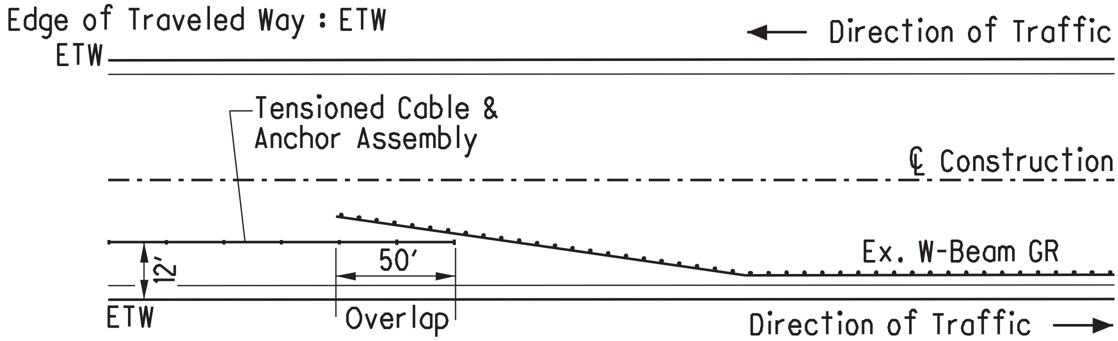
TENSIONED CABLE ANCHOR ASSEMBLY OVERLAP

* Where transverse space between cable barrier and the opposing traffic barrier exists such that a gap in cross-median protection would be created, additional length of overlap between the two barriers should be considered.

TENSIONED CABLE OVERLAPPING OTHER BARRIER

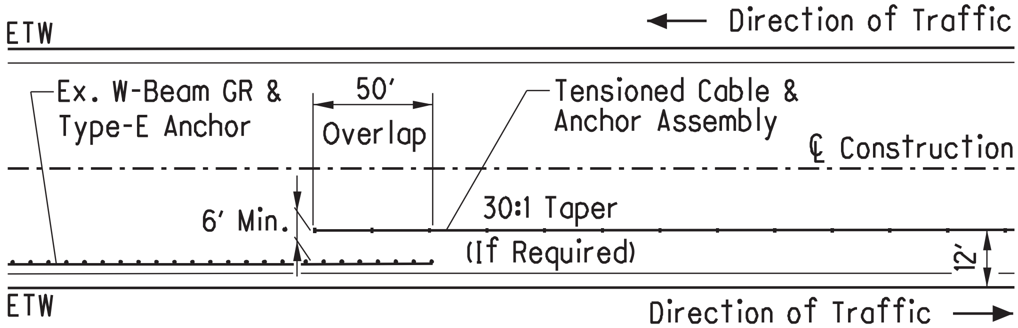
602-4b

REFERENCE SECTIONS
602.2.2, 603.1.1



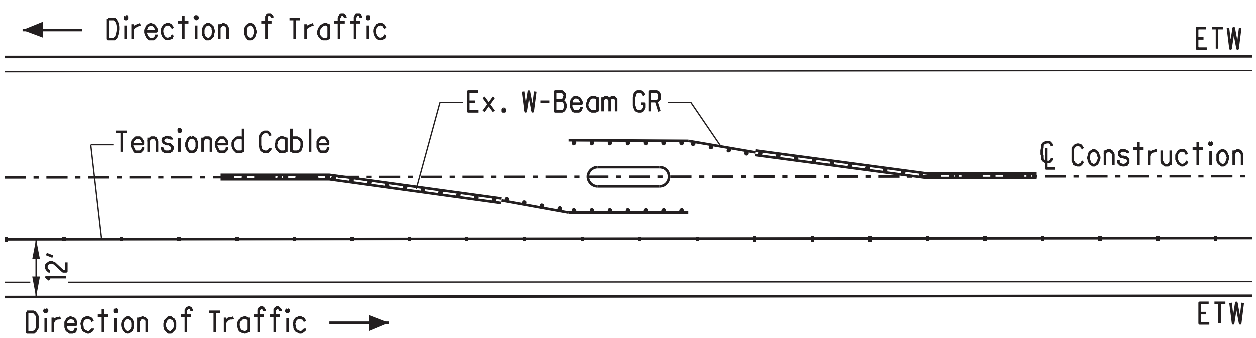
ENDING TENSIONED CABLE AT EX. FLARED GUARDRAIL

Where possible, preference is to overlap the trailing barrier end in front of / closer to the roadway than the beginning / approach end of the second barrier.



BEGIN/END TENSIONED CABLE AT EX. TYPE-E ANCHOR ASSEMBLY

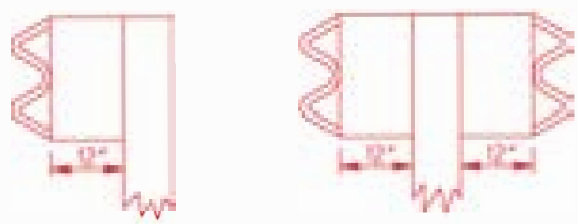
Where possible, preference is to overlap the trailing barrier end in front of / closer to the roadway than the beginning / approach end of the second barrier.



TENSIONED CABLE AT BRIDGE PIERS

Bypass Ex. Pier GR unless 12′ offset cannot be met, then treat as flared GR as shown in above

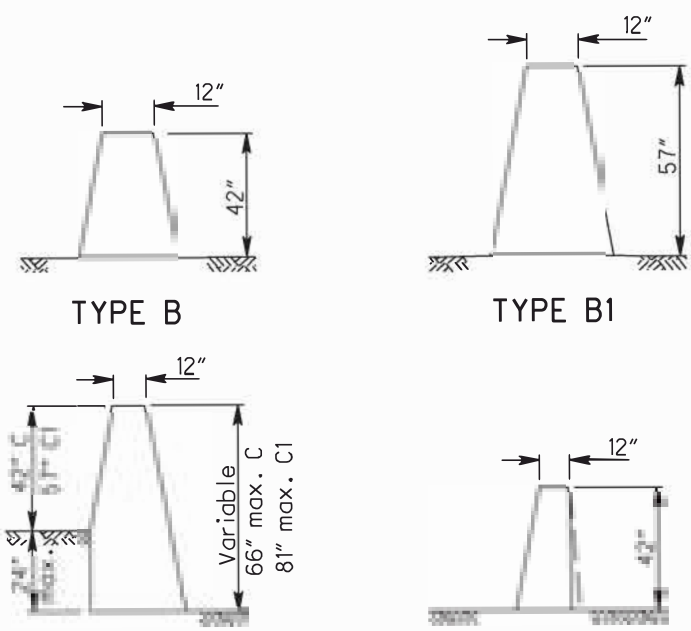
BARRIER TYPES



TYPE MGS

TYPE MGS: Barrier

STEEL BEAM GUARDRAIL



TYPE B

TYPE B1

TYPE C & C1

TYPE D

SINGLE SLOPE BARRIER

TYPICAL BARRIER USES & MINIMUM CLEARANCES

603-2

REFERENCE SECTIONS 602.1.1, 603.1

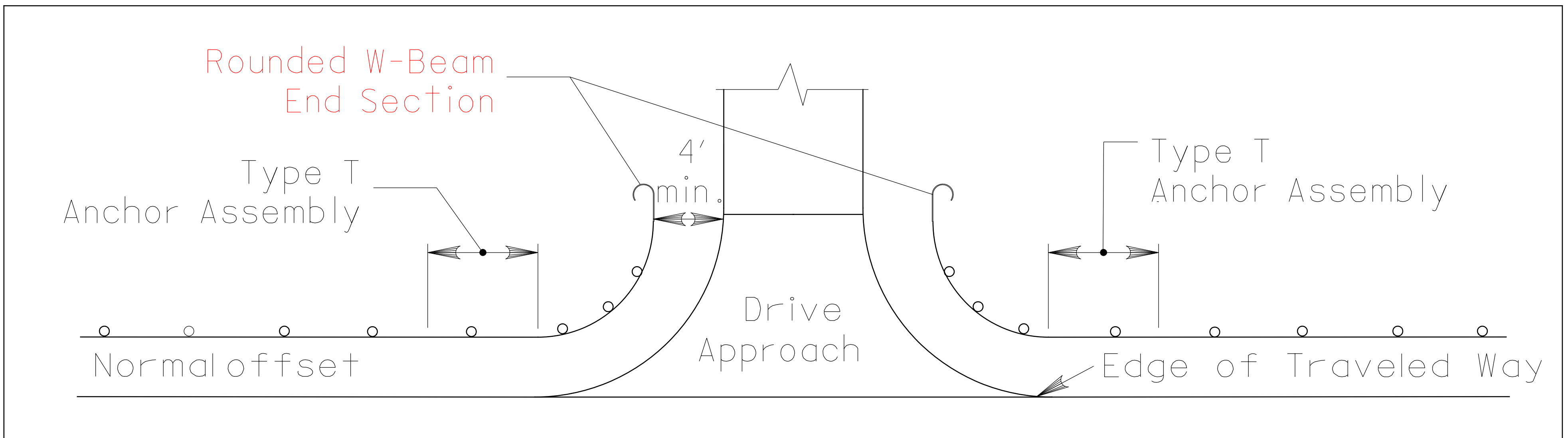
	Barrier Type	Standard Drawing	Working Width*	Typical Use
STEEL BEAM GUARDRAIL	Type MGS	MGS-2.1	5'	Roadside protection. 6'-3" Standard Post Spacing
		MGS-2.1	3'-6"	Roadside protection adjacent to fixed objects. 3'-1 1/2" Half Post Spacing
		MGS-2.1	3'	Roadside protection adjacent to fixed objects. 1'-6 3/4" Quarter Post Spacing
	MGS Barrier	MGS-2.1 MGS-6.1 MGS-6.2	5'	Narrow medians where deflections can be tolerated.
	MGS Long Span Across Culvert	MGS-2.3	8'	Used primarily to span across precast structures that have limited depths of cover
	Socketed Weak Post Mounting	MGS-2.4	5'	Used primarily on precast structures that have limited depths of cover
CONCRETE BARRIER	50” PCB	RM-4.1	6’-3”	These clearances represent unanchored PCB lateral offset to fixed objects. Can be installed with minimum 2-foot offset to MOT traffic lanes and minimum 2-foot offset to the work area.
	32” PCB	RM-4.2	5’-6”	
	Type B	RM-4.3	Width of Barrier 28”	Narrow medians where raceways or median lighting is used.
	Type B1	RM-4.3	Width of Barrier 33 3/4”	Narrow medians where additional height is required and raceways are needed.
	Type C	RM-4.3	Width of Barrier Varies up to 32 3/8”	Narrow medians where the difference in shoulder elevation is 24 inches or less.
	Type C1	RM-4.3	Width of Barrier Varies up to 38-1/4”	
	Type D	RM-4.5	Width of Barrier 20”	Roadside protection adjacent to fixed obstacles. Areas where impact angles over 15 degrees are unlikely or where maintenance may be difficult/dangerous.

*Working Width - The distance between the traffic face of the barrier before impact and the maximum lateral position of any major part of the system or vehicle after impact.

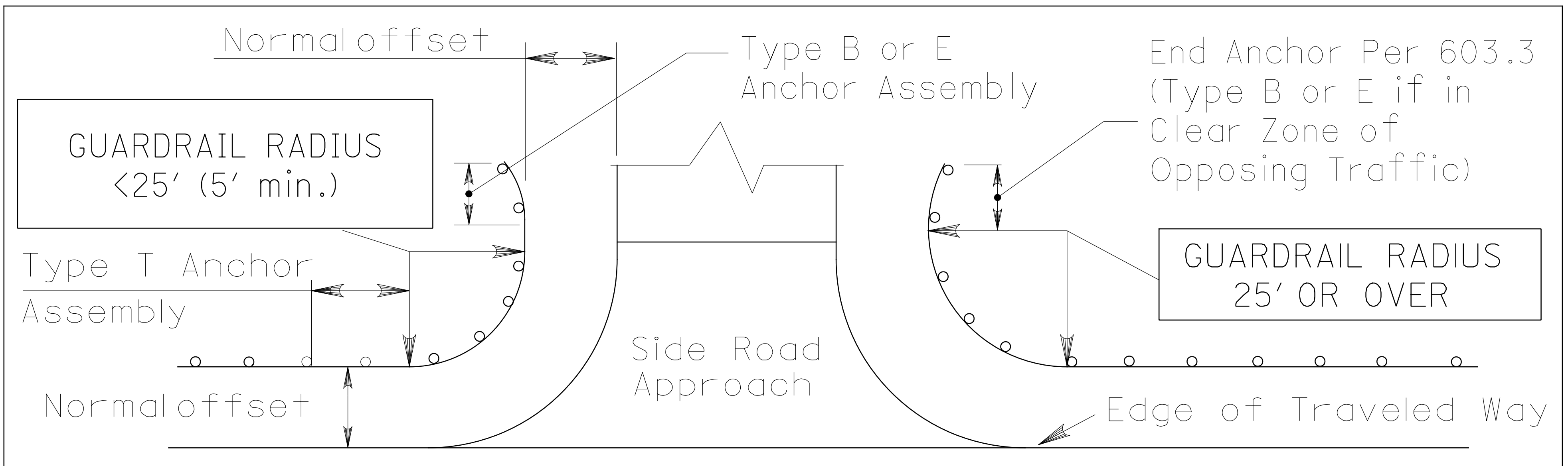
DRIVE AND SIDE ROAD GUARDRAIL OPENINGS

603-3E

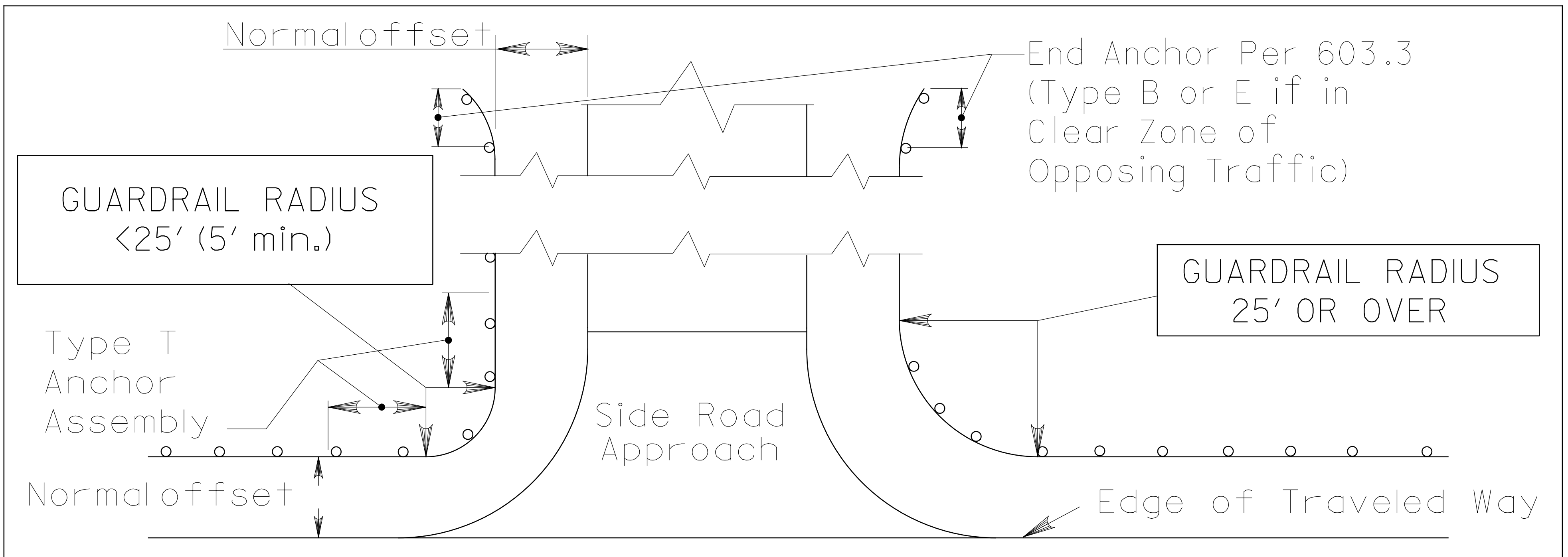
REFERENCE SECTIONS 602.4, 603.3



DRIVE GUARDRAIL OPENING



SIDE ROAD GUARDRAIL OPENING



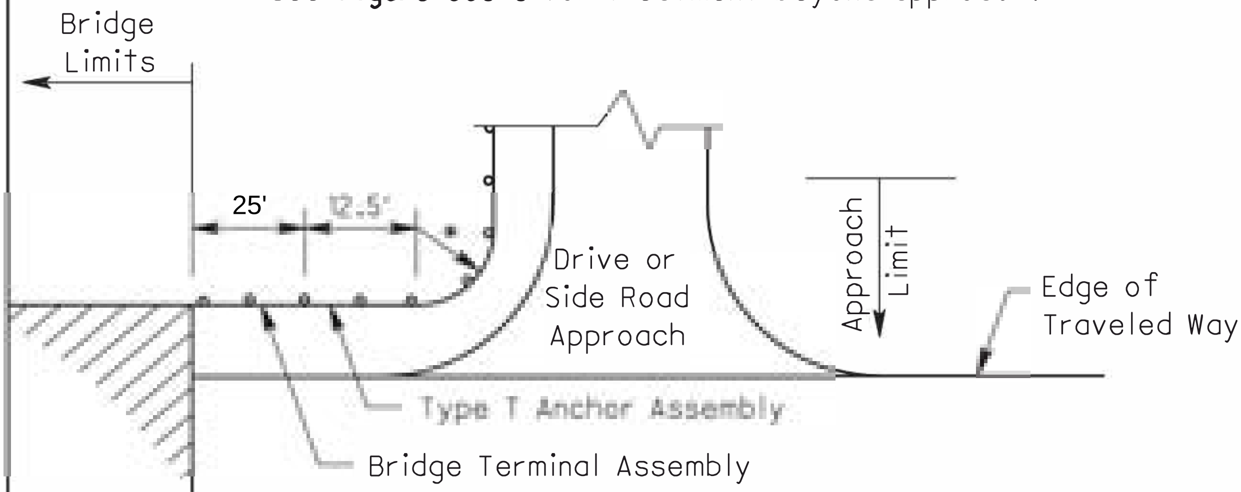
SIDE ROAD GUARDRAIL OPENING WHERE GUARDRAIL EXTENDS ALONG SIDE ROAD

January 2020

MINIMUM BRIDGE PROTECTION INVOLVING DRIVES OR SIDE ROADS

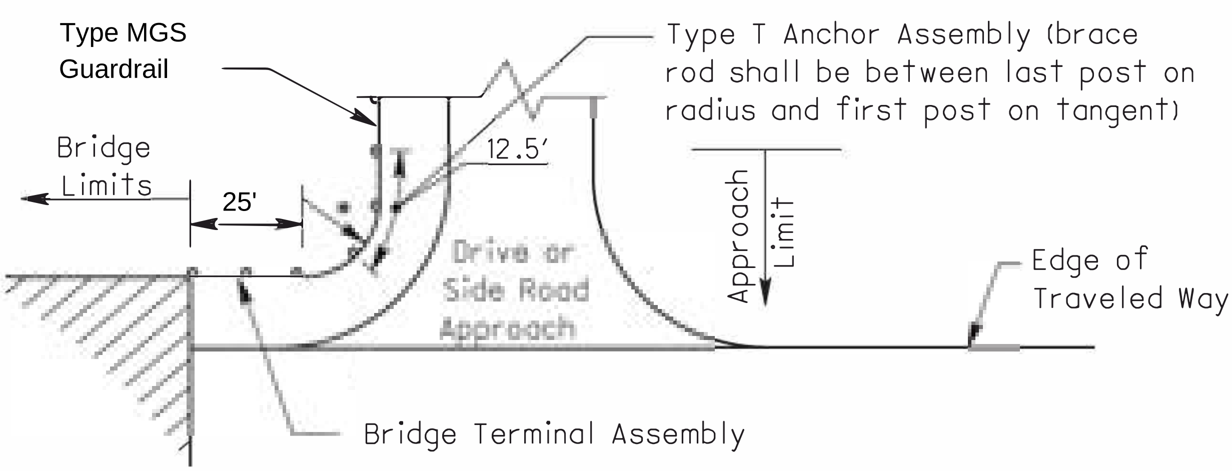
603-4
REFERENCE SECTIONS 602.5.1, 603.3.5

See **Figure 603-3** for treatment beyond approach.



PREFERRED MINIMUM APPROACH TREATMENT

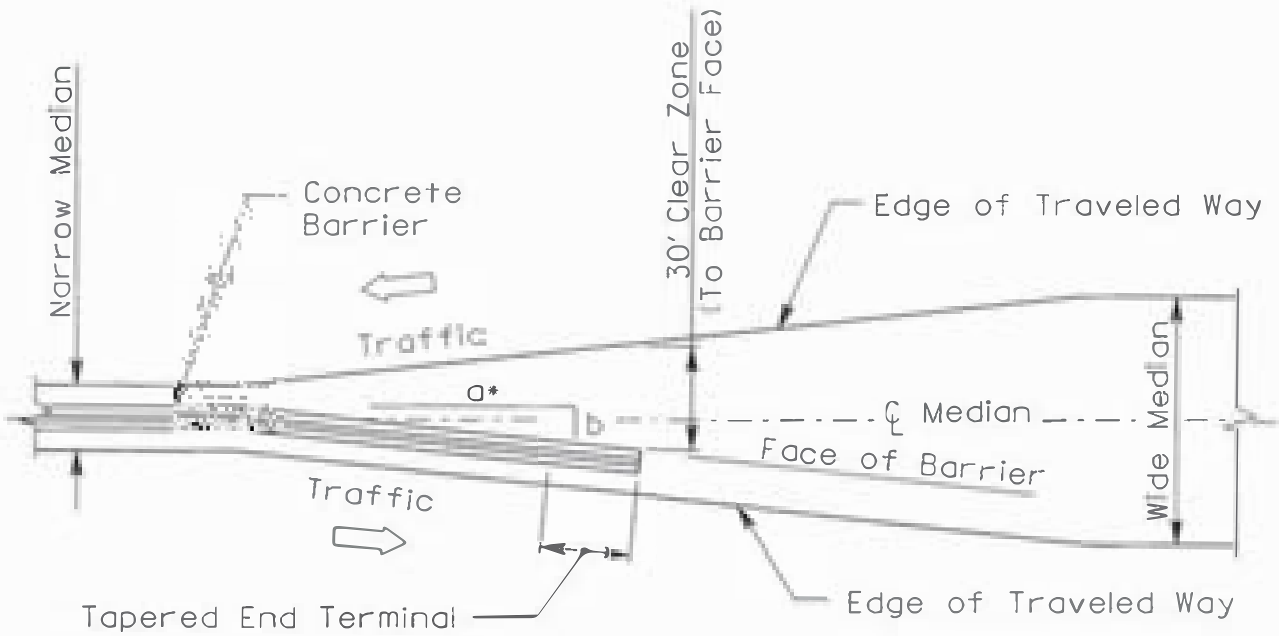
See **Figure 603-3** for treatment beyond approach.



ABSOLUTE MINIMUM APPROACH TREATMENT

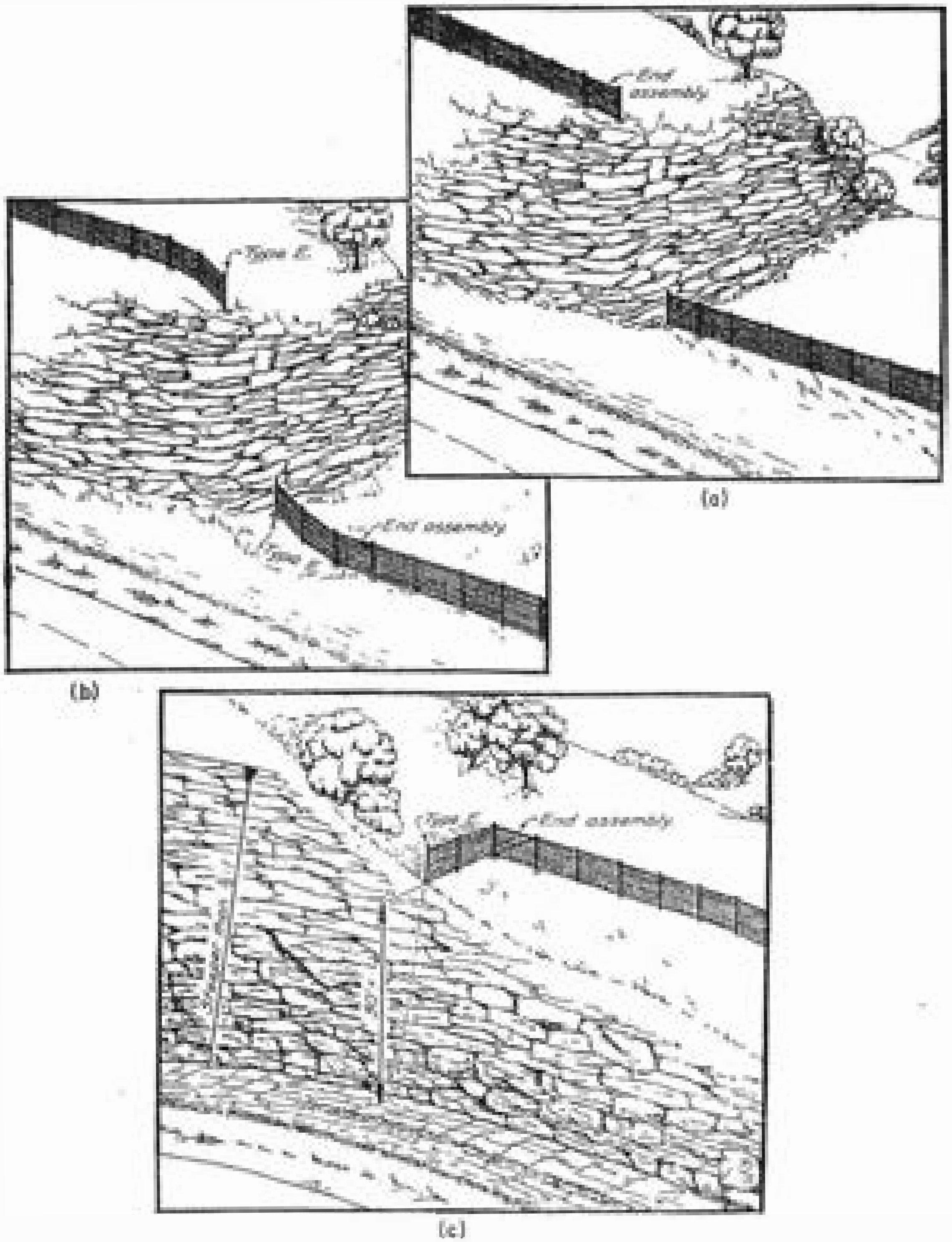
* Minimum Guardrail radius is 5'.

CONCRETE BARRIER MEDIAN TRANSITION	603-5
	REFERENCE SECTIONS 603.6



* See Figure 602-1 for barrier flare rates.

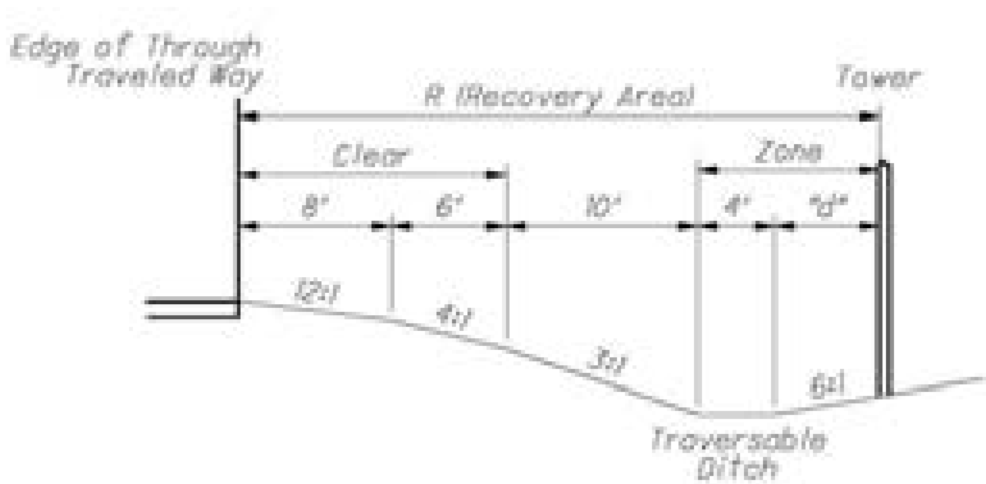
EXCEPTION TO CONTINUOUS FREEWAY FENCING	606-1
	REFERENCE SECTIONS 606.3.4



SAMPLE CALCULATIONS

Ex. 600-1	Clear Zone Measurement Using Slope Averaging (Traversable Ditch)

Problem 1: Compute the safe distance from the edge of traveled way to locate a tower for lighting. The project has a design speed of 55 mph, a design year traffic volume of 3,400 ADT and the following cross section in the area where the tower is to be located:



Solution 1: **Step 1** - Check the foreslope from the edge of traveled way to the backslope to determine if all intermediate foreslopes are either recoverable or non-recoverable. (See **Figure 600-2**.)

Since the foreslope has intermediate slopes that are recoverable (12:1 & 4:1) and non-recoverable (3:1), the clear zone may extend into the backslope if necessary.

Step 2 - Determine the weighted average of the foreslope. For sections flatter than or equal to 10:1, use a 10:1 slope. (The 12:1 shoulder slope is typically ignored; however, for this example it is included for illustrative purposes.) Decimal results of 0.5 or greater should be rounded up to the next whole numbered slope while decimal results less than 0.5 should be rounded down to the next whole numbered slope.

First, multiply the width of each slope by the rate of the slope to obtain the weighted average rise for the foreslope. Include half of the ditch bottom in the foreslope.

8' (1/10) + 6' (1/4) + (0*) + 4'/2 (1/10) = 2.5'

* Since the 3:1 foreslope is non-recoverable, it is not included.

SAMPLE CALCULATIONS

Ex. 600-1	Clear Zone Measurement Using Slope Averaging (Traversable Ditch)

(continued)

Next, add the width of each foreslope used above.

8' +6' +4'/2 = 16'

Then, divide the total recoverable width by the weighted average rise to obtain the weighted average of the foreslopes.

16'/2.5' = 6.4 (Rounded to 6:1 slope)

Now, enter **Figure 600-1** (for 6:1 or flatter foreslopes, 55 mph Design Speed and 1,501 < ADT < 6,000) to determine that the required clear zone distance is 21 feet.

Since the required clear zone is 21 feet and only 16 feet of recoverable clear zone exists, additional width must be considered from the backslope.

Step 3 - Determine if the ditch section is traversable.

Using **Figure 307-11**, a ditch with a 3:1 foreslope and 6:1 backslope is traversable.

If a non-traversable ditch section had been provided then the designer would have to consider other site conditions to determine whether or not the ditch should be used within the clear zone or if guardrail should be installed.

Step 4 - Determine the clear zone using the backslope.

Determine how much of the backslope should be included in the clear zone.

21' - 8' - 6' - 4' = 3'

Therefore, the clear zone must extend 3 feet into the backslope.

The “Recovery Area” includes the clear zone width plus any intermediate widths where the slopes are traversable, but not recoverable.

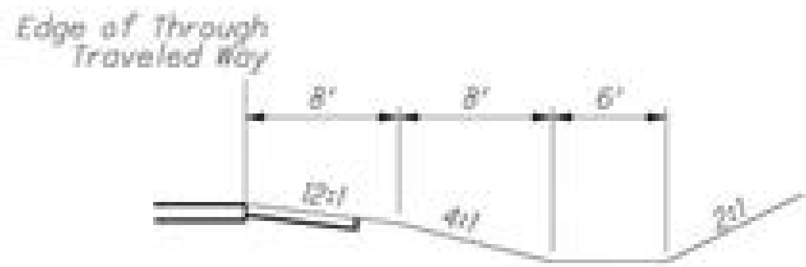
Recovery Area: 8' + 6' + 10' + 4' + 3' = 31'

Ex. 600-2	**SAMPLE CALCULATIONS**
	Clear Zone Measurement Using Slope Averaging (Traversable Ditch)

<u>Problem 2</u>: **a**) Determine the required clear zone distance for the following location on a project with a tangent alignment, a design speed of 55 mph and a design year traffic volume of 1,700 ADT.

b) Assuming this cross section occurs on the outside of a 2-Degree curve, how would this change the above results?

c) Determine the clear zone distance for a Degree of Curve of 3 degrees.



<u>Solution 2</u>: **a**) - The required clear zone distance (for foreslopes steeper than 6:1 up to 4:1, 55 mph design speed, and 1,501≤ADT≤6,000) is 27 feet. 19 feet of clear distance is available up to the center of the ditch. A trapezoidal ditch with a 4:1 foreslope, 2:1 backslope and a width equal to or greater than 4 feet is a non-traversable design (see **Figure 307-11**) and generally should not be located within the clear zone. However, if the probability of encroachment is low no additional improvement may be needed.

b) - Since this location is on the outside of a curve where the probability of encroachment is high, the designer should consider reshaping the ditch or installing guardrail.

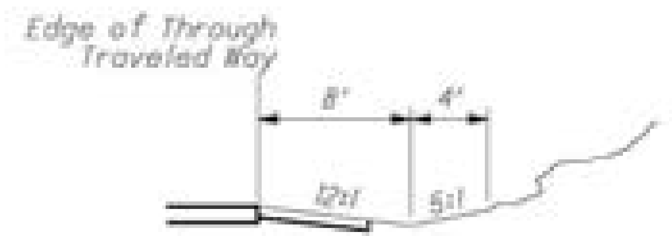
c) - The required clear zone distance determined above for a tangent alignment needs to be increased by a factor or 1.2 for locations on the outside of curves with a curvature of 3 degrees and a design speed of 55 mph. (See **Figure 600-1**.) The adjusted clear zone distance is 27 (1.23) = 33.2'. Since the adjusted value is greater than 30', use 30'.

Since 19 feet or only 63% of the required clear zone distance exists on the outside of this curve, the designer should consider reshaping the ditch or installing guardrail.

SAMPLE CALCULATIONS

Ex. 600-3	Clear Zone Measurement For a Cut Slope

Problem 3: Determine the required clear zone distance for the following location on a project with a design speed of 45 mph and a design year traffic volume of 1,300 ADT.



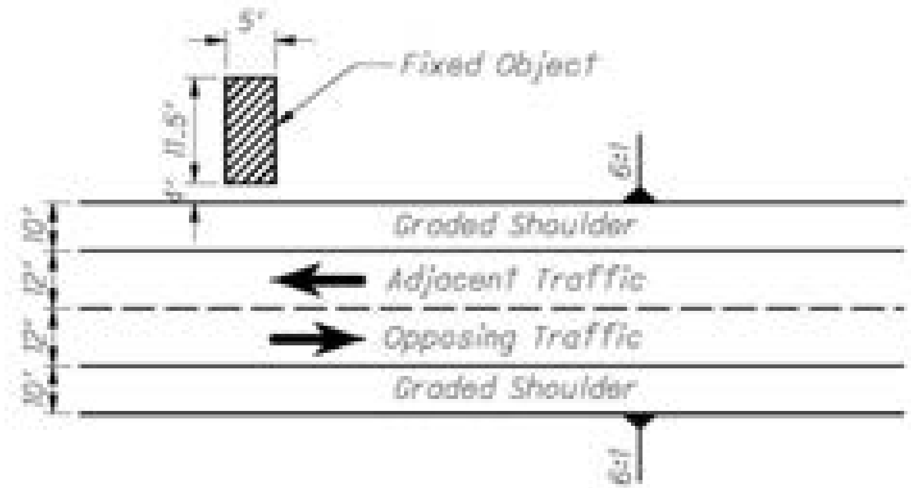
Solution 3: The required clear zone distance (for backslopes steeper than 6:1 up to 4:1, 45 mph design speed and 750≤ADT≤1,500) is 13 feet. (See **Figure 600-1**.) The required clear zone is 13 feet but only 12 feet exist. If this section of roadway has a history of accidents with the cut face then guardrail should be installed.

SAMPLE CALCULATIONS

Ex. 602-1

Tangent Barrier Design For a 2-lane Road

<u>Problem 1</u>: Design barrier if needed to shield the fixed object located on the two-lane non-NHS rural collector shown below. The project has a design speed of 60 mph, a design year traffic volume of 2,200 ADT and a 6:1 foreslope. Assume that the object cannot be removed, relocated or made traversable.



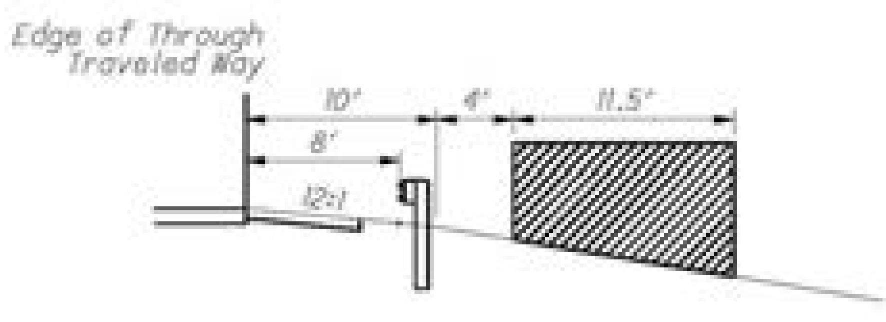
<u>Solution 1</u>: **<u>Step 1</u>** - Determine whether or not the fixed object is in the clear zone for adjacent traffic. Refer to **Figure 600-1** (for 6:1 or flatter foreslope, 60 mph design speed and 1501≤ADT≤6000) to determine that the required clear zone distance is 28 feet.

The available clear area for adjacent traffic is 10' + 4' = 14 ft.

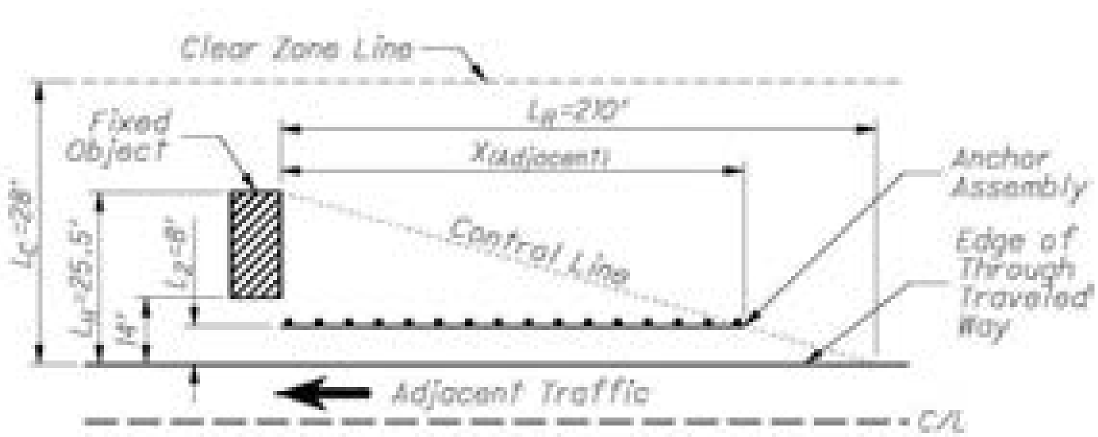
Since the object cannot be removed, relocated or made traversable and it is inside the required clear zone, a barrier should be installed to shield it.

(continued)

Step 2 - Select the type of barrier to be installed. Using **Figure 301-3**, the normal (minimum) barrier offset for a rural collector (Design Year ADT greater than 2000) is 8 feet from the edge of traveled way. The available barrier clearance at this location is (10' - 8') + 4' = 6 ft; therefore, use Type 5 Guardrail which has a minimum barrier clearance of 5.5 feet. (See **Figure 603-2**.)



Step 3 - Calculate the length of need for adjacent traffic. Assume the area along the front of the fixed object cannot be graded to provide 10:1 foreslopes; therefore, the guardrail cannot be installed with a flare.



SAMPLE CALCULATIONS

Ex. 602-1 — Tangent Barrier Design For a 2-lane Road

(continued)

From **Figure 602-1**, L_R = 210 ft. (for design speed = 60 mph and 1000≤ADT≤5000). Since the lateral offset to the back of the object (L_H) is less than the required clear zone distance (L_C), use L_H in the LON formula.

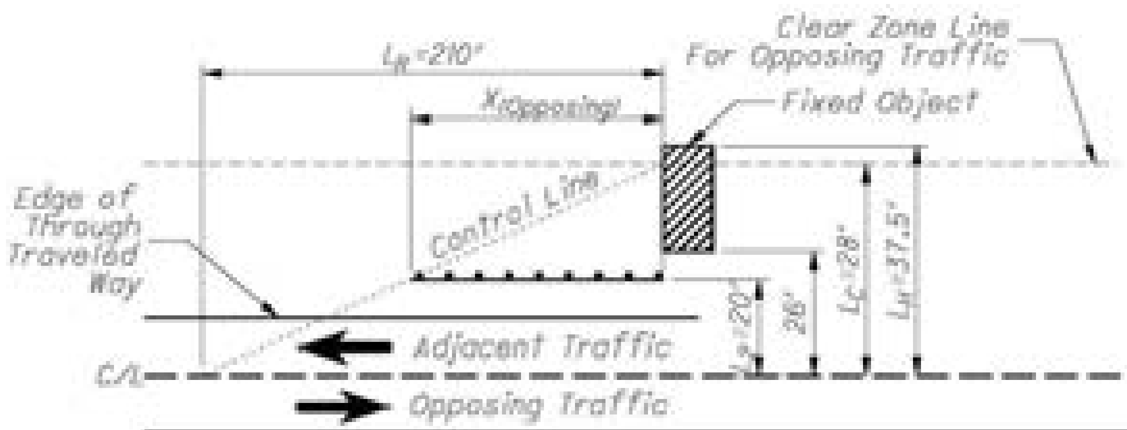
$$x = \frac{L_H + L_1 b/a - L_2}{b/a + L_H/L_R}$$

Start measuring the length of guardrail needed at the edge of the fixed object. Since the guardrail will not be flared, b/a = 0.

$$x_{\text{(adjacent)}} = \frac{25.5 + 0 - 8}{0 + 25.5/210} = 144.12 \text{ ft.}$$

<u>Step 4</u> - Determine whether or not the fixed object is in the clear zone for opposing traffic. The required clear zone is still 28 feet. The available clear area is 12' $_{\text{(lane width)}}$ + 14' = 26 ft. Since the object is in the clear zone, calculate the offset to the back of the object, L_H.

L_H = 12' + 14' + 11.5' = 37.5 ft.



Since L_H>L_C, protection only needs to be provided up to the clear zone.

$$x_{\text{(opposing)}} = \frac{L_C + L_1 b/a - L_2}{b/a + L_C/L_R} = \frac{28' + 0 - 20'}{0 + 28/210} = 60.00 \text{ ft.}$$

(continued)

The total length of guardrail required is:

$x_{(adjacent)}$ + width of object + $x_{(opposing)}$ = 144.12+ 5' + 60.00' = 209.12ft.

The length provided should be a multiple of even 12'-6" panel lengths.

x = 209.12'/12.5' = 16.73 Use 17 panels or 17(12.5') = 212.5 ft.

<u>Note</u> - If the designer had chosen to shield the entire object from opposing traffic instead of providing protection up to the clear zone, then

$$x_{(opposing)} = \frac{L_H - L_2}{L_H/L_R} = \frac{37.5 - 20}{37.5/210} = 98 \text{ ft.}$$

The total length of guardrail needed would have been:

144.12' + 5' + 98' = 247.12 ft. (or 20 panels)

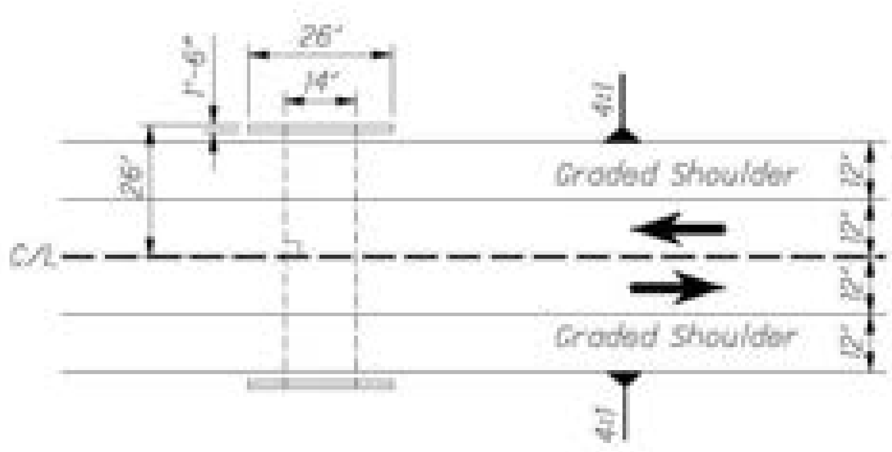
Three additional panels (37.5 feet) of guardrail would be installed. In some cases, the designer may choose to shield the entire object even though a portion of it is outside the clear zone; however, in some cases it may be uneconomical to do so.

<u>**Step 5**</u> - Select Anchor Assemblies. Since this is a non-NHS collector with a design year ADT≤4000, a Type A Anchor Assembly may be installed on the approach and trailing ends of the guardrail run.

SAMPLE CALCULATIONS

Ex. 602-2	Length of Need at a Large Culvert

Problem 2: Design barrier if needed to shield the culvert headwalls located on the two-lane non-NHS rural collector shown below. This bridge replacement project has a design speed of 55 mph, a design year traffic volume of 4,100 ADT and 4:1 foreslopes.



Solution 2 **Step 1** - Determine whether or not the headwall is in the clear zone for adjacent traffic. Refer to **Figure 600-1** (for foreslopes steeper than 6:1 up to 4:1, 55 mph design speed and 1501≤ADT≤6000) to determine that the required clear zone distance is 27 feet measured from the edge of traveled way.

The available clear area for adjacent traffic is 26' - 12' - 1'-6" = 12'-6".

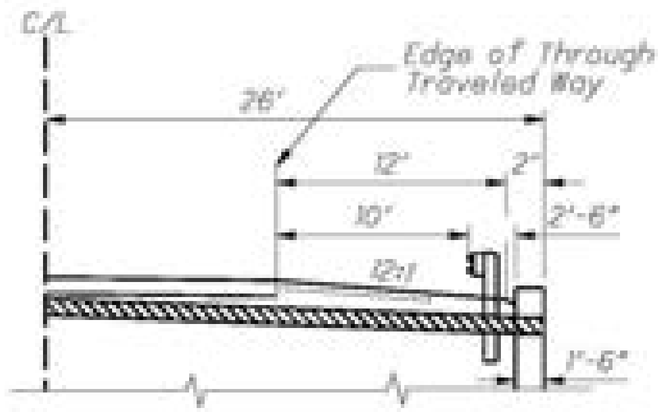
It is impractical to almost double the length of the culvert to get the headwalls outside the clear zone; therefore, barrier should be provided.

Step 2 - Select the type of barrier to be installed. Using **Figure 301-3**, the normal barrier offset for a rural collector (Design Year ADT greater than 2000) is 10' from the edge of traveled way. The available barrier clearance at this location is (12' - 10') + (2' - 1.5') = 2.5 ft.

SAMPLE CALCULATIONS

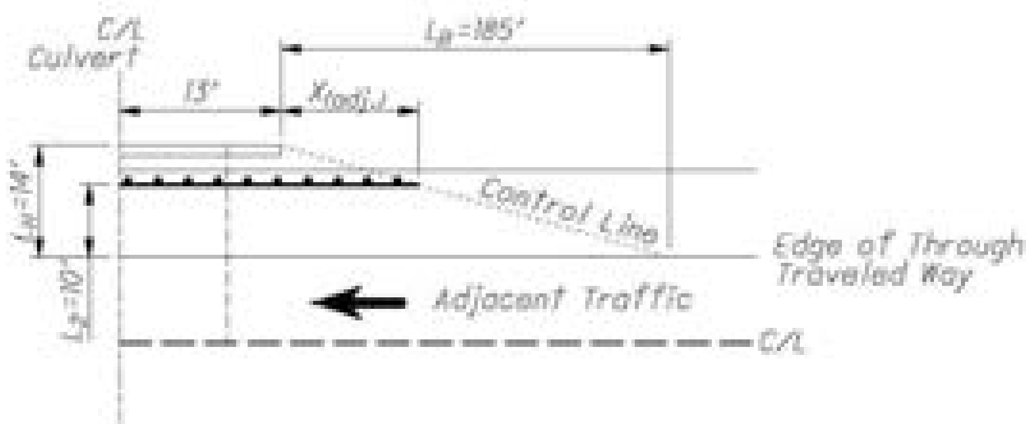
Ex. 602-2	Length of Need at a Large Culvert

(continued)



Since there is not enough clearance available for Type 5 Guardrail, which has a minimum barrier clearance of 5'-6", use Type 5 Guardrail with Tubular Backup, which has a minimum barrier clearance of 24." (See **Figure 603-2**.)

<u>Step 3</u> - Calculate the length of need for adjacent traffic. Since the foreslope along the face of the fixed object cannot be regraded to 10:1, do not flare the guardrail. (The geometrics of the roadway and the offset to the headwall are the same on both sides of the road; therefore, the lengths calculated for adjacent and opposing traffic for the eastbound lane will be the same as those calculated for adjacent and opposing traffic for the westbound lane.)



SAMPLE CALCULATIONS

Ex. 602-2	**Length of Need at a Large Culvert**

(continued)

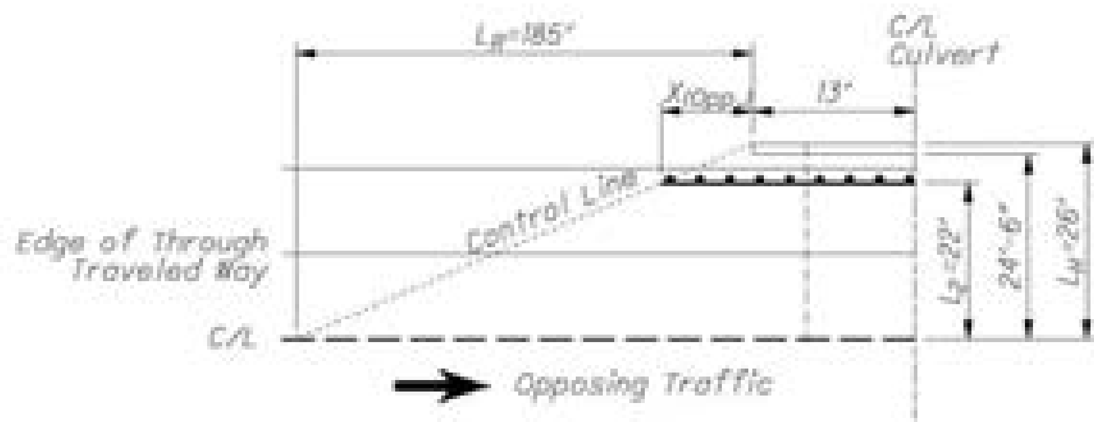
From **Figure 602-1**, L_R = 185 ft. (for design speed = 55 mph and 1000≤ADT≤5000). Since the lateral offset to the back of the headwall (L_H) is less than the required clear zone distance (L_C), use L_H in the LON formula.

$$x = \frac{L_H + L_1 b/a - L_2}{b/a + L_H/L_R}$$

Start measuring the length of guardrail needed at the edge of the headwall. Since the guardrail will not be flared, b/a = 0.

$$x_{\text{(adjacent)}} = \frac{14' + 0 - 10'}{0 + 14'/185'} = 52.85 \text{ ft.}$$

<u>Step 4</u> - Determine whether or not the headwall is in the clear zone for opposing traffic. The required clear zone distance is still 27 feet. The available clear area is 26' - 1'-6" = 24'-6".



Since L_H<L_C, $x = \dfrac{L_H + L_1 b/a - L_2}{b/a + L_H/L_R}$

Start measuring the length of guardrail needed at the edge of the headwall. Since the guardrail will not be flared, b/a = 0.

$$x_{\text{(opposing)}} = \frac{26' + 0 - 22'}{0 + 26'/185'} = 28.46 \text{ ft.}$$

(continued)

The total length of guardrail required is:

$x_{(adjacent)}$ + width of headwall + $x_{(opposing)}$ = 52.85' + 26' + 28.46' = 107.32 ft.

The length provided should be a multiple of even 12'-6" guardrail panel lengths.

x = 107.32/12.5 = 8.59   Use 9 panels or 9(12.5') = 112.5 ft.

Step 5 - Detail the final installation, including the anchor assemblies. The Type 5 Guardrail with Tubular Backup should extend to the first post off the approach and trailing ends of the structure. In this case, the headwall (not the culvert itself) is the structure that is being protected. This headwall is slightly longer than 2 panels of guardrail so use 3 panels (37'-6"). A Type 4 Bridge Terminal Assembly is required at each end of the Type 5 Guardrail with Tubular Backup. This 25' long transition is paid for as a unit and its length can be included as part of the total of Type 5 Guardrail being installed.

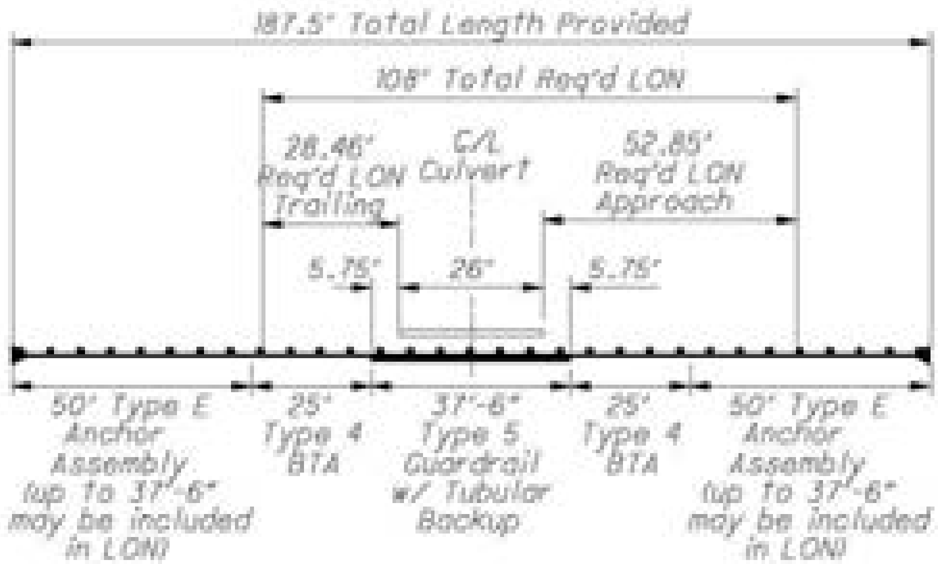
Type A Anchor Assemblies are not permitted because the design year ADT is over 4000. See **Section 603.3.4**. Refer to **Table 603-1** in **Section 603.3.3** for a Bridge Replacement Project with foreslopes steeper than 6:1 up to 4:1 to determine that a Type E Anchor Assembly should be used on the approach and trailing ends. (It is required on the trailing end because it is within the clear zone for opposing traffic.)

Since up to 37'-6" of the 50' long Type E can be deducted from the guardrail length of need, decrease the amount of rail specified for the approach end by this amount. In this case, the 25' of the BTA + the 37.5' of the Type E + 5.75' Tubular Backup = 68.25', which exceeds the 52.85' LON.

On the trailing end the amount of barrier included in the Bridge Terminal Assembly and the Type E also exceeds the 28.46' LON. (See the following final detail.)

Note: Many large culverts are located in deep channels with steep side slopes. This may necessitate that the designer use $L_H = L_C$ when calculating the required length of need.

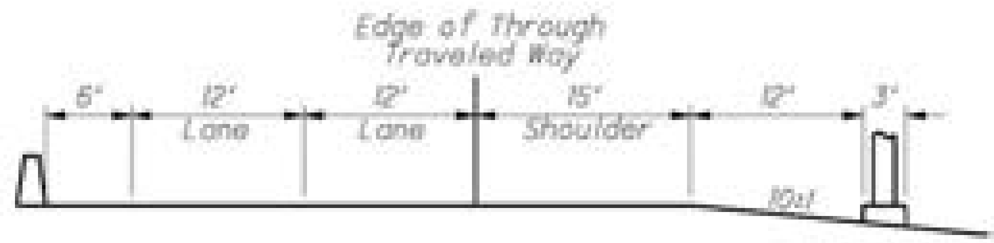
(continued)


187.5' Total Length Provided
108' Total Req'd LON
28.46' Req'd LON Trailing
C/L Culvert
52.85' Req'd LON Approach
5.75'
26'
5.75'
50' Type E Anchor Assembly (up to 37'-6" may be included in LON)
25' Type 4 BTA
37'-6" Type 5 Guardrail w/ Tubular Backup
25' Type 4 BTA
50' Type E Anchor Assembly (up to 37'-6" may be included in LON)

SAMPLE CALCULATIONS

Ex. 602-3 — Tangent and Flared Barrier Design For a Divided Highway

<u>Problem 3</u>: Design barrier if needed to shield the 3' diameter footing located on the 4-lane, divided, NHS, urban, interstate reconstruction project shown below. The project has a design speed of 70 mph, a design year traffic volume of 12,000 ADT and 10:1 foreslopes. If barrier is needed calculate how much should be provided if it is installed **a**) at the normal (minimum) barrier offset on a tangent, **b**) at the normal (minimum) barrier offset on a flare, **c**) as close to the footing as permissible on a tangent and **d**) as close to the footing as permissible on a flare.



<u>Solution 3</u>: **<u>Step 1</u>** - Determine whether or not the footing is in the clear zone for adjacent traffic. Refer to **Figure 600-1** (for foreslopes 6:1 or flatter, 70 mph design speed and ADT>6000) to determine that the required clear zone distance is 32 feet measured from the edge of traveled way. However, since this is not a high accident area a maximum clear zone distance of 30' should be used.

The available clear area for adjacent traffic is 15' + 12' = 27'

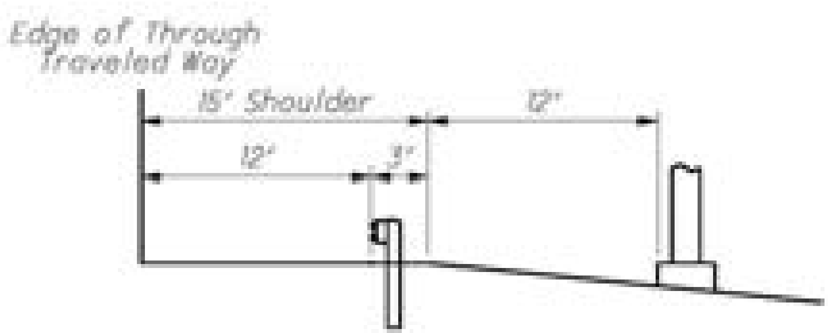
Assuming the footing cannot be relocated outside the clear zone, barrier should be provided.

<u>Step 2</u> - Select the type of barrier to be installed. Using **Figures 301-4 & 301-3**, the normal (minimum) barrier offset for an urban interstate route is 12' from the right edge of traveled way. The available barrier clearance at this location is 3' + 12' = 15'; therefore, use Type 5 Guardrail, which has a minimum barrier clearance of 5.5'. (**See Figure 603-2**.)

SAMPLE CALCULATIONS

Ex. 602-3 | **Tangent and Flared Barrier Design For a Divided Highway**

(continued)



Step 3 - Calculate the length of need for adjacent traffic. (A calculation for opposing traffic is unnecessary because the concrete median barrier prevents encroachments by opposing vehicles.)

From **Figure 602-1**, L_R = 360 ft. (for Design Speed = 70 mph and ADT over 10000).

a) For tangent guardrail at the normal (minimum) barrier offset, $L_H=L_C=30'$, $L_2=12'$, and b/a = 0.

$$x = \frac{L_H + L_1 b/a - L_2}{b/a + L_H/L_R} = \frac{30' + 0 - 12'}{0 + 30'/360'} = 216'$$ Use 18 panels.

b) For flared guardrail at the normal (minimum) barrier offset, b/a = 1/15. (See **Figure 602-1**.) Let L_1=12'-6" (one panel length). In this case, this is an arbitrary selection. Site conditions typically control the amount of tangent barrier that should be provided past the warranting feature before a flare is introduced. For instance, where a flared section of Type 5 Guardrail is attached to a tangent section of Type 5A, it is advisable to extend the Type 5A past the warranting feature such that L_1 is at least equal to one panel length. Since Type 5 and 5A have different deflection characteristics, this ensures adequate protection at the edge of the warranting feature.

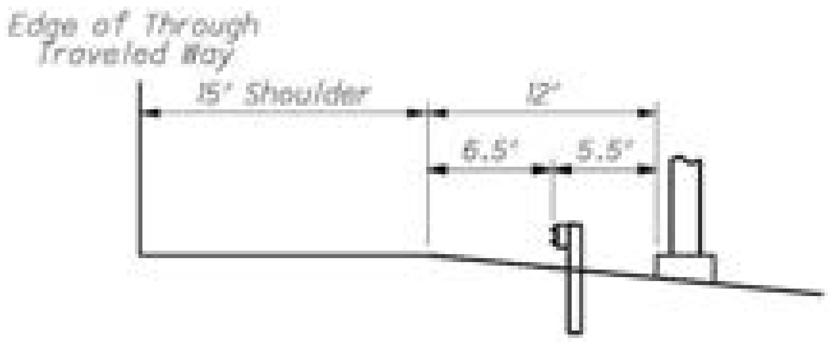
$$x = \frac{30' + 12.5'(1/15) - 12'}{1/15 + 30'/360'} = \frac{30' + 0.83' - 12'}{0.15'} = 125.55'$$ Use 10 panels.

(continued)

c) Guardrail can be installed on slopes that are 10:1 or flatter. Since Type 5 Guardrail has a minimum barrier clearance of 5.5' the guardrail can be placed at this distance in front of the footing.

L_2 = 15' + 12' - 5.5' = 21.5'. For tangent guardrail, b/a = 0. L_H is still equal to 30'.

$$x = \frac{30' + 0 - 21.5'}{0 + 30'/360'} = 102' \quad \text{Use 9 panels.}$$



d) For flared guardrail offset at 21.5':

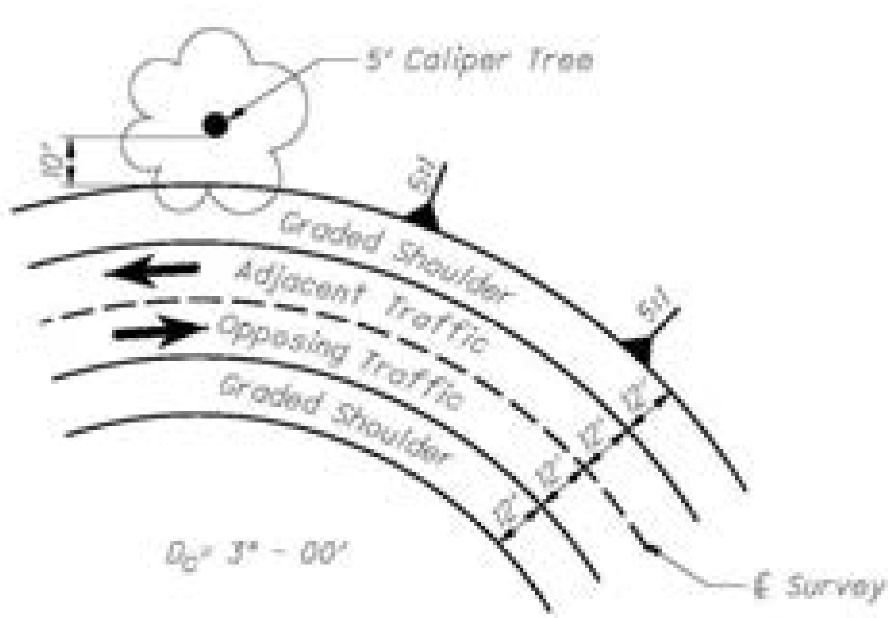
$$x = \frac{30' + 12.5'(1/15) - 21.5'}{1/15 + 30'/360'} = \frac{30' + 0.83' - 21.5'}{0.15'} = 62.22' \quad \text{Use 5 panels.}$$

All of these solutions are correct; however, d) is the best solution because it provides the most recovery area with the least amount of barrier.

Step 4 - Select Anchor Assemblies. Refer to **Table 603-1** for a major reconstruction project with 6:1 or flatter foreslopes to determine that the approach terminal should be either a Buried in Backslope or Type B Anchor Assembly. There is no backslope so select the Type B. Use a Type T Anchor Assembly on the trailing end since it cannot be impacted by opposing traffic.

SAMPLE CALCULATIONS	
Ex. 602-4	**Barrier on the Outside of a Curve**

Problem 4: Calculate the barrier length of need to shield the 200-yr old 5-ft. diameter tree located on the outside of a 3-degree curve as shown below. The HSP project is on a rural arterial and has a design speed of 55 mph, a design year traffic volume of 3800 ADT and 5:1 foreslopes. Assume that the HSP project is needed to address run-off-the-road impacts with the tree and also assume that the tree cannot be removed.



Solution 4: **Step 1** - Determine whether or not the tree is in the clear zone for adjacent traffic. From **Figure 600-1** (for foreslopes steeper than 6:1 up to 4:1, 55 mph design speed and 1501≤ADT≤6000) the required clear zone distance is 27 feet measured from the edge of traveled way. Since the tree is on the outside of a 3-degree curve, the clear zone should be widened by using the curve correction factor for 55 mph design speed (1.2) from the chart at the bottom of **Figure 600-1**.

Required Clear Zone = 1.23 (27') = 33.21 ft.

Do not reduce this value to 30 ft. since this is a high accident location.

The offset to the face of the tree is 12' + 10' = 22 ft. This is less than L_C = 33.21 ft.; therefore, install barrier.

SAMPLE CALCULATIONS

Ex. 602-4

Barrier on the Outside of a Curve

(continued)

<u>Step 2</u> - Select the type of barrier to be installed. Using **Figure 301-3**, the normal (minimum) barrier offset for a rural arterial (Design year ADT greater than 2000) is 10 feet from the right edge of traveled way. The available barrier clearance at this location is 12 feet; therefore, use Type 5 Guardrail, which has a minimum barrier clearance of 5.5 feet. (See **Figure 603-2.)**

<u>Step 3</u> - Calculate the length of need for adjacent traffic. The radius for the 3-degree curve is $R_{centerline} = 5729.58/D_C = 5729.58/3.0 = 1909.86'$.

The radius at the edge of traveled way is 1909.86' + 12' =1921.86'.

The lateral offset to the back of the tree is, $L_H = 22' + 5' = 27'$.

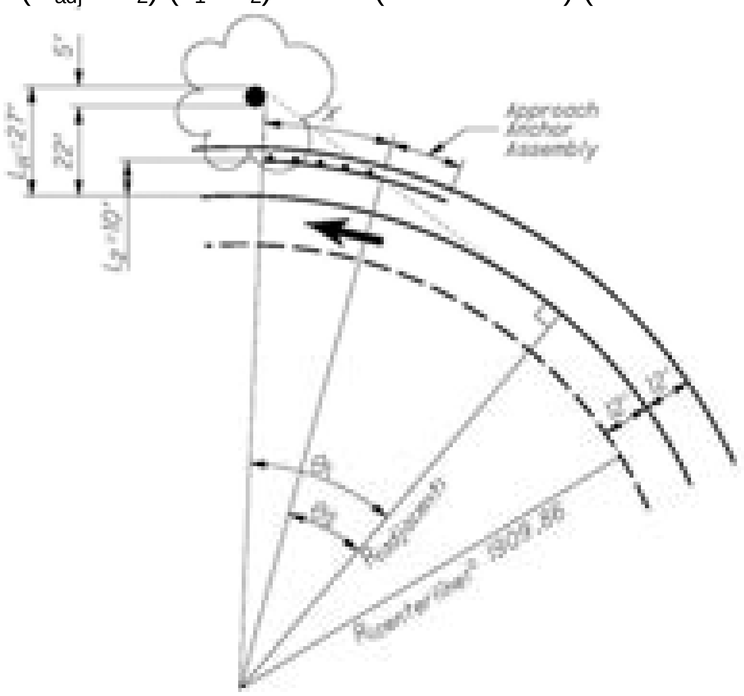
$\theta_1 = \cos^{-1} (R_{adj} / (R_{adj} + L_H)) = \cos^{-1} (1921.86 / (1921.86 + 27)) = 9.5484°$

$9.5484°(\pi/180) = 0.1666$ radians

$\theta_2 = \cos^{-1} (R_{adj} / (R_{adj} + L_2)) = \cos^{-1} (1921.86 / (1921.86 + 10)) = 5.8323°$

$5.8323°(\pi/180) = 0.1018$ radians

$X = (R_{adj} + L_2) (\theta_1 - \theta_2)$ rad. $= (1921.86 +10) (0.1666 - 0.1018) = 125.18'$



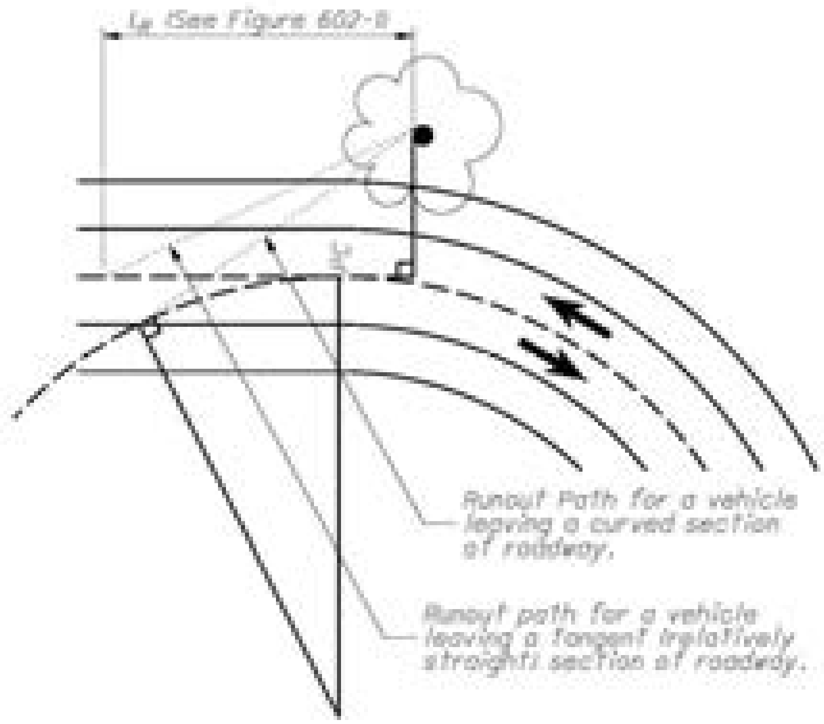
(continued)

Step 4 - Determine whether or not the tree is within the clear zone for opposing traffic. The offset to the face of the tree is 12' + 12' + 10' = 34'. Since this is outside the clear zone, guardrail is not needed past the left side of the tree to shield it from opposing traffic.

The total length of guardrail needed is 125.18' + 5' = 130.18'
Use 11 panels (137.5').

Refer to **Table 603-1** in **Section 603.3.3** to determine the recommended anchor assembly for an HSP project with foreslopes steeper than 6:1 up to 4:1. On the approach end install a Type E Anchor Assembly. Since 37'-6" of the 50' long Type E can be deducted from the guardrail length of need, decrease the amount of rail specified above at the approach end by this amount. (Use 100'.) On the trailing end install a Type T Anchor Assembly because it is outside the clear zone for opposing traffic.

Notes - If a point of curvature exists in the vicinity of the runout path, the curve may need to be extended past the PC or PT (into the tangent portion of the roadway) in order to construct the tangent control line. If this is the case, then the standard runout lengths for tangent roadways should be used to calculate length of need.



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700 Multi-Modal Considerations

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700 Multi-Modal Considerations

701 RAILROADS

701.1 Background

Ohio is interlaced with a network of railroad systems controlled by a multiplicity of local and state laws and regulations. The complexity of railroad operations and regulations requires that special consideration be given to the location of highways with respect to railroad track, whether it be the intersection of a highway with a railroad, or the location of a highway adjacent to a railroad facility.

701.2 Crossing At-Grade

701.2.1 General

Highways that cross railroad tracks on a common grade should be located to provide for a minimum of interference to highway traffic and the least amount of adjustment of railroad facilities.

Crossings at-grade will not be permitted on freeways. The creation of new grade crossings where none now exist should be avoided and will require railroad and **Court of Common Pleas approval. (Sec. 957.29 et. seq. ORC).**

701.2.2 Railroad Parallel to Highway

When locating a highway parallel to a railroad track, consideration shall be given to the need for space adjacent to railroad tracks for future industrial development. It is desirable to locate the highway a sufficient distance from the railroad to permit rail service to industrial areas without crossing the highway.

Sufficient distance from a railroad to a parallel highway should be provided along crossroads on which traffic must stop before entering the highway, to permit vehicles to stop clear of the railroad track.

701.3 Lateral Clearances

The standard gage of railroad tracks is 4 feet 8 ½ inches. Where two or more tracks are parallel, the normal centerline spacing is 14 feet.

701.3.1 New Construction

Although minimum lateral clearances vary with railroad ownership, clearance from the centerline of the outside track should normally be at least 18 feet. An additional 8 feet of lateral clearance should be provided when a railroad off-track equipment road is located parallel to the tracks.

701.3.2 Reconstruction

The above clearances should be provided when replacing an existing structure when such additional work can be accomplished at a reasonable cost. A horizontal clearance less than the existing clearance will not be permitted.

701.4 Vertical Clearance

701.4.1 New Construction

A minimum of 23 feet between the top of rail and the bottom of an overpassing structure should be provided. This vertical clearance should extend 6 feet on each side of the centerline of the outside tracks. Actual clearance requirements will be determined after the location plan has been submitted.

701.4.2 Reconstruction

Every attempt should be made to increase the minimum vertical clearance to 23 feet when such additional work can be accomplished at a reasonable cost. A vertical clearance less than the existing clearance will not be permitted.

701.4.3 Construction Clearances

Construction clearances should also be considered in the design stages since they could be a factor in the location of certain items such as catch basins, headwalls, etc. A minimum of 9 feet of lateral clearance should be maintained at all times from the centerline of the track during construction unless this is not possible because of existing conditions.

702 SHARED USE PATHS

702.1 General

Shared use paths are multi-use paths designed primarily for use by bicyclists and pedestrians, including those with disabilities, for transportation and recreation purposes. Shared use paths are physically separated from motor vehicle traffic by an open space or barrier. The following sections are based on the AASHTO Guide for the Development of Bicycle Facilities Fourth Edition, the Manual of Uniform Traffic Control (MUTCD), and the FHWA document, Shared Use Path Level of Service Calculator.

702.1.1 Accessibility Requirements for Shared Use Paths

Due to the fact that nearly all shared use paths are used by pedestrians, they fall under the accessibility requirements of the Americans with Disabilities Act (ADA). Paths in the public right of way that function as sidewalks should be designed in accordance with the proposed Public Rights of Way Accessibility Guidelines (PROWAG), or subsequent guidance that may supersede PROWAG in the future.

Shared use paths built in independent right of way should meet the draft accessibility guidelines in the Advance Notice of Proposed Rulemaking (ANPRM) on Accessibility Guidelines for Shared Use Paths or any subsequent rulemaking that supersedes the ANPRM.

702.2 Elements of Design

The first step in designing a shared use path is determining the design users. Due to the large percentage of adult bicyclists, they are the basis for most of the design recommendations.

702.2.1 Width and Clearance

The next step in designing a shared use path is determining the cross section. The width of the shared use path should be sufficient to serve the expected volume of users with a facility consistent with guidance for safe operation. The minimum paved width for a two-directional shared use path is 10 feet. Typically, widths range from 10' to 14', with wider widths applicable to areas with high use and/ or a wider variety of user groups. The FHWA document, Shared Use Path Level of Service Calculator can be used in determining the appropriate width of a pathway. Wider paths are advisable in the following situations:

- When there is a significant use by inline skaters, adult tricycles, children, or other users that need more operating width;
- Where the path is used by larger maintenance vehicles;
- On steep grades to provide additional passing area; or
- Through curves to provide more operating space.

Ideally, a graded shoulder width at least 3 to 5 feet wide with a maximum cross slope of 6:1 should be provided on each side of the pathway. At a minimum, a 2 foot graded area with a maximum slope of 6:1 should be provided for clearance from lateral obstructions such as bushes, large rocks, bridge piers, abutments, and poles. See ***Figure 701-1*** for a typical cross section of a two-way shared use path.
Where paths are adjacent to parallel bodies of water or downward slopes of 3:1 or steeper, a wider separation should be considered. A 5 ft. separation from the edge of path pavement to the top of the slope is desirable. Depending on the height of the embankment and condition at the bottom, a physical barrier, such as dense shrubbery, railing or fencing may be needed. Where a recovery area (distance between the edge of the path pavement and the top of the slope) is less than 5 feet, physical barriers or rails are recommended in the following situations (see ***Figure 701-2***):

- Slope 3:1 or greater, with a drop of 6' or greater;
- Slope 3:1 or greater, adjacent to a parallel body of water or other substantial object;
- Slope 2:1 or greater, with a drop of 4' or greater
- Slopes 1:1 or greater, with a drop of 1' or greater.

The barrier or rail should begin prior to, and extend beyond the area of need. The lateral offset of the barrier should be at least 1' from the edge of path. The ends of the barrier should be flared away from the path edge.

It is not desirable to place the pathway in a narrow corridor between two fences for long distances, as this creates personal security issues, prevents users who need help from being seen, prevents path users from leaving the path in an emergency, and impedes emergency response.

Objects shall not overhang or protrude into any portion of a shared use path at or below 8' measured from the finish surface. In some situations, a vertical clearance greater than 8' may be needed to permit passage of maintenance and emergency vehicles.

702.2.2 Shared Use Paths Adjacent to Roadways (Sidepaths)

While it is generally preferable to select path alignments in independent rights-of-way, there are situations where existing roads provide the only corridors available. Sidepaths are specific type of shared use path that run adjacent to the roadway, where right-of-way and other physical constraints dictate. Sidepaths may be considered in addition to on-road bicycle facilities. A sidepath should satisfy the same design criteria as shared use paths in independent right-of-way.

Utilizing or providing a sidewalk as a two-way shared use path is undesirable.

Paths can function along highways for short sections, or for longer sections where there are few street and/or driveway crossings, given appropriate separation between facilities and attention to reducing crashes at junctions. Two-way sidepaths can create operational concerns. These conflicts include:

1. At intersections and driveways, motorists entering or crossing the roadway often will not notice bicyclists approaching from their right, as they do not expect wheeled traffic from this direction. Motorists turning from the roadway onto the cross street may likewise fail to notice bicyclists traveling the opposite direction from the norm.

2. Bicyclists traveling on sidepaths are apt to cross intersections and driveways at unexpected speeds (speeds that are significantly faster than pedestrian speeds). This may increase the likelihood of crashes, especially where sight distance is limited.

3. Motorists waiting to enter the roadway from a driveway or side street may block the sidepath crossing, as drivers pull forward to get an unobstructed view of traffic.

4. Attempts to require bicyclists to yield or stop at each cross street or driveway are inappropriate and are typically not effective.

5. When the sidepath ends, bicyclists traveling in the direction opposed to roadway traffic may continue on the wrong side of the roadway. Similarly, bicyclists approaching a path may travel on the wrong side of the roadway to access the path. Wrong-way travel by bicyclists is a common factor in bicycle-automobile crashes.

6. Depending upon the bicyclist's specific origin and destination, a two-way sidepath on one side of the road may need additional road crossings (and therefore increase exposure); however the sidepath may also reduce the number of road crossings for some bicyclists.

7. Signs posted for roadway users are backwards for contra-flow riders, who cannot see the sign information. The same applies to traffic signal faces that are not oriented to contra-flow users.

8. Because of the proximity of roadway traffic to opposing path traffic, barriers or railings are sometimes needed to keep traffic on the roadway or path from inappropriately encountering each other. These barriers can represent an obstruction to bicyclists and motorists, impair visibility between road and path users, and can complicate path maintenance.

9. Sidepath width is sometimes constrained by fixed objects (such as utility poles, trash can, mailboxes, etc.)

10. Some bicyclists will use the roadway instead of the sidepath because of the operational issues described above. Bicyclists using the roadway may be harassed by motorists who believe bicyclists should use the sidepath.

11. Bicyclists using a sidepath can only make a pedestrian-style left turn, which generally involves yielding to cross traffic twice instead of only once, and thus induces unnecessary delay.

12. Bicyclists on the sidepath, even those going in the same direction, are not within the normal scanning area of drivers turning right or left from the adjacent roadway into a side road or driveway.

13. Even if the number of intersections and driveway crossings is reduced, bicycle-motor vehicle crashes may still occur at the remaining crossings located along the sidepath.

14. Traffic control devices such as signs and markings have not been shown effective at changing road or path user behavior at sidepath intersections or reducing crashes and conflicts.

For these reasons, sidepaths should not be used.

Guidelines for Sidepaths

Although paths in independent rights-of-way are preferred, sidepaths may be considered where one or more of the following conditions exist:

- The adjacent roadway has relatively high-volume and high-speed motor vehicle traffic that might discourage many bicyclists from riding on the roadway, potentially increasing sidewalk riding, and there are no practical alternatives for either improving the roadway or accommodating bicyclists on nearby parallel streets.

- The sidepath is used for a short distance to provide continuity between sections of path in independent rights-of-way, or to connect local streets that are used as bicycle routes.

- The sidepath can be built with few roadway and driveway crossings.

- The sidepath can be terminated at each end onto streets that accommodate bicyclists, onto another path, or in a location that is otherwise bicycle compatible.

In some situations, it may be better to place one-way sidepaths on both sides of the street or highway. Clear directional information is needed if this design is used. This design can reduce some of the concerns associated with a two-way sidepath at driveways and intersections.

A wide separation should be provided between a two-way sidepath and the adjacent roadway. The minimum recommended distance between a path and the roadway curb or edge of travelled way (where there is no curb) is 5 ft. Where a paved shoulder is present, the separation distance begins at the outside edge of shoulder. Where the separation is less than 5 feet, a physical barrier or railing should be provided between the path and the roadway. Such barriers or railings serve to prevent path users from making undesirable or unintended movements from the path to the roadway and to reinforce the concept that the path is an independent facility. The barrier or railing need not be of a size and strength to redirect an errant motorist toward the roadway, unless other conditions indicate the need for a crashworthy barrier. Barriers or railings at the outside of a structure or a steep fill embankment should be a minimum of 42 in. high. Barrier at other locations that serve only to separate the area for motor vehicles from the sidepath should generally have a minimum height equivalent to the height of a standard guardrail.

702.2.3 Design Speed

The next step in shared use path design is to determine the design speed. For most paths in relatively flat areas (grades less than 2 percent), a design speed of 18 mph is generally sufficient, except on inclines where higher speeds can occur.

702.2.4 Horizontal Alignment

After determining the design speed of the shared use path, the horizontal and vertical alignment of the shared use path should be designed. The minimum radius of horizontal curvature for bicyclists can be calculated using two different methods. One method uses “lean angle”, and the other method uses superelevation and coefficient of friction. In general, the lean angle method should be used in design. The table below shows minimum radii of curvature for a paved path using a 20-degree lean angle. See the AASHTO Guide for the Development of Bicycle Facilities 2012 Edition for information on calculating the minimum radius based superelevation and coefficient of friction.

US Customary	
Design Speed (mph)	**Minimum Radius (ft)**
12	27
14	36
16	47
18	60
20	74

25	115
30	166

Minimum Radii for Horizontal Curve on Paved Shared Use Path at a 20-degree Lean Angle

702.2.5 Cross Slope

Shared use paths should have a maximum cross slope of 2 percent, to accommodate people with disabilities.

702.2.6 Grade

The maximum grade of a shared use path contained within the roadway right of way shall not exceed the general grade established for the adjacent roadway. Where the shared use path is not contained within the roadway right of way, the maximum grade of the shared shall be 5 per cent.

702.2.7 Stopping Sight Distance

To provide path users with opportunities to see and react to unexpected conditions, shared use paths should be designed with adequate stopping sight distances.

For a crest vertical curve, the height of eye is assumed to be 4.5 ft. and the object height is assumed to be 0 in. to recognize that impediments to bicycle travel exists at pavement level. ***Figure 701-3*** can be used to select the minimum length of vertical curve needed to provide minimum stopping sight distances at various speeds on crest vertical curves.

Figure 701-4 illustrates the horizontal sight distance for a shared use path. The lateral clearance (horizontal sight line offset) is obtained using the table from ***Figure 701-5*** and the proposed horizontal radius of curvature.

Path users typically travel side-by-side on shared use paths. On narrow paths, bicyclists tend to ride near the middle of the path. Lateral clearances on horizontal curves should be calculated based on the sum of the stopping sight distances for path users travelling in opposite directions around the curve.

702.2.8 Surface Structure

The surfaces of shared use paths should be firm, stable, and slip resistant and shall comply with R302.7 of the PROWAG.

Vertical alignment shall be generally planar within shared use path (including curb ramp runs, turning spaces, and gutter areas within shared use path) and surfaces at other elements. Grade breaks shall be flush. Where shared use paths cross rails at grade, the shared use path shall be level and flush with the top of rail at the outer edges of the rails, and the surface between the rails shall be aligned with the top rail.

It is important to maintain a smooth riding surface on shared use paths. Vertical surface discontinuities shall be 0.5 in. maximum. Vertical surface discontinuities between 0.25 in. and 0.5 in. shall be beveled with a slope not steeper than 50 percent. The bevel shall be applied across the entire vertical surface discontinuity.

Utility covers and bicycle compatible grates should be flush with the surface of the pavement on all sides. Horizontal openings in gratings and joints shall not permit passage of a sphere more than 0.5 in. in diameter. Elongated openings in gratings shall be placed so that the long dimension is perpendicular to the dominant direction of travel. Railroad crossings should be smooth and be designed at an angle between 60 and 90 degrees to the direction of travel in order to minimize the possibility of falls. Flangeway gaps at pedestrian at-grade crossings shall be 2.5 in. maximum on non-freight rail track and 3 in. maximum on freight rail track.

702.2.9 Bridges and Underpasses

The receiving clear width on the end of a bridge (from inside of rail or barrier to inside of opposite rail or barrier) should allow 2 ft. of clearance on each side of the shared use path but under constrained conditions may taper to the shared use path width.

Carrying the clear areas across the structures has two advantages. First, the clear width provides a minimum horizontal shy distance from the railing or barrier, and second, it provides needed maneuvering space to avoid conflicts with pedestrians or bicyclists who have stopped on the bridge.

Protective railings, fences, or barriers on either side of a shared use path on a stand-alone structure should be a minimum of 42 in. high. There are some locations where a 48 in. high railing should be considered in order to prevent bicyclists from falling over the railing during a crash. This includes bridges or bridge approaches where high-speed, steep angle impacts between a bicyclists and a railing may occur, such as at a curve at the foot of a long descending grade where the curve radius is less than appropriate for the design speed or anticipated speed.

Openings between horizontal or vertical members on railings should be small enough that a 6 in. sphere cannot pass through them in the lower 27 in. For the portion of railing that is higher than 27 in., openings may be spaced such that an 8 in. sphere cannot pass through them. This is done to prevent children from falling through the openings. Where a bicyclist’s handlebar may come into contact with a railing or barrier, a smooth wide rubrail may be installed at a height of about 36 in. to 44 in. to reduce the likelihood that bicyclist’s handlebar will be caught by the railing.

The structural design of shared use path bridges should be designed in accordance with the AASHTO LRFD Bridge Design Specifications for Design of Pedestrian Bridges.

702.3 Shared Use Path Intersection Design

Shared use path intersection can be at a “new” mid-block location or a sidepath at an existing intersection of two roadways. Both intersection designs should consider the variable speed between the vehicles and path users, the available intersection sight distance and the traffic volumes. The objectives of both designs are:

- Alert the motorists and path users to the crossing

- Communicate who has the obligation to yield to whom
- Enable the motorists and/ or path users to fulfill their obligations

Illumination of the path/ roadway intersection should be considered, especially on unlit paths.
Curb ramps with detectable warnings should be provided at intersections. The curb ramps and detectable warnings should extend the full width of the shared use path.

702.3.1 Design of Mid-Block Crossings

It is preferable for mid-block crossings to intersect the roadway at a 90° angle to minimize the crossing distance and to maximize the intersection sight distance.

Shared use paths are unique in terms of assignment of the right of way, due to the legal responsibility to drivers to yield to pedestrians in crosswalks. Bicyclists approach the intersection at a far greater speed than pedestrians. A stop or yield sign is need to remind the bicyclists who has the legal right of way at crossings.

The least restrictive form of intersection control should be used at shared use path intersections. A common misconception is the routine installation of stop control for the pathway. Per the MUTCD, Stop signs should not be used where Yield signs would be acceptable." Sight triangles should be used in selecting the appropriate control (see ***Figures 701-6 & 701-7***).

Additional traffic control, such as a signal or active warning device, may be needed due to the traffic volumes, vehicular speed or roadway geometry.

702.3.2 Sidepath Intersection Design

The potential issues with sidepaths are discussed in ***Section 702.2.2***, but there are times when they are unavoidable. The following design measures may reduce crashes:

- Reduce the driveway density.
- Reduce the speeds of both the path user and the motorists. Tighter corner radii, median refuge islands, and no free flow right turns are several examples.
- Improve visibility. Keep approaches to intersections and major driveways clear of obstructions such as parked vehicles, landscaping elements and traffic control devices.

At signalized intersections, the following design measures should be considered:

- Prohibit right turn on red.
- Provide a leading pedestrian interval or if the volumes on the path are high, then consider an exclusive phase.
- Allow turning movements on fully protected phases only.

LIST OF FIGURES

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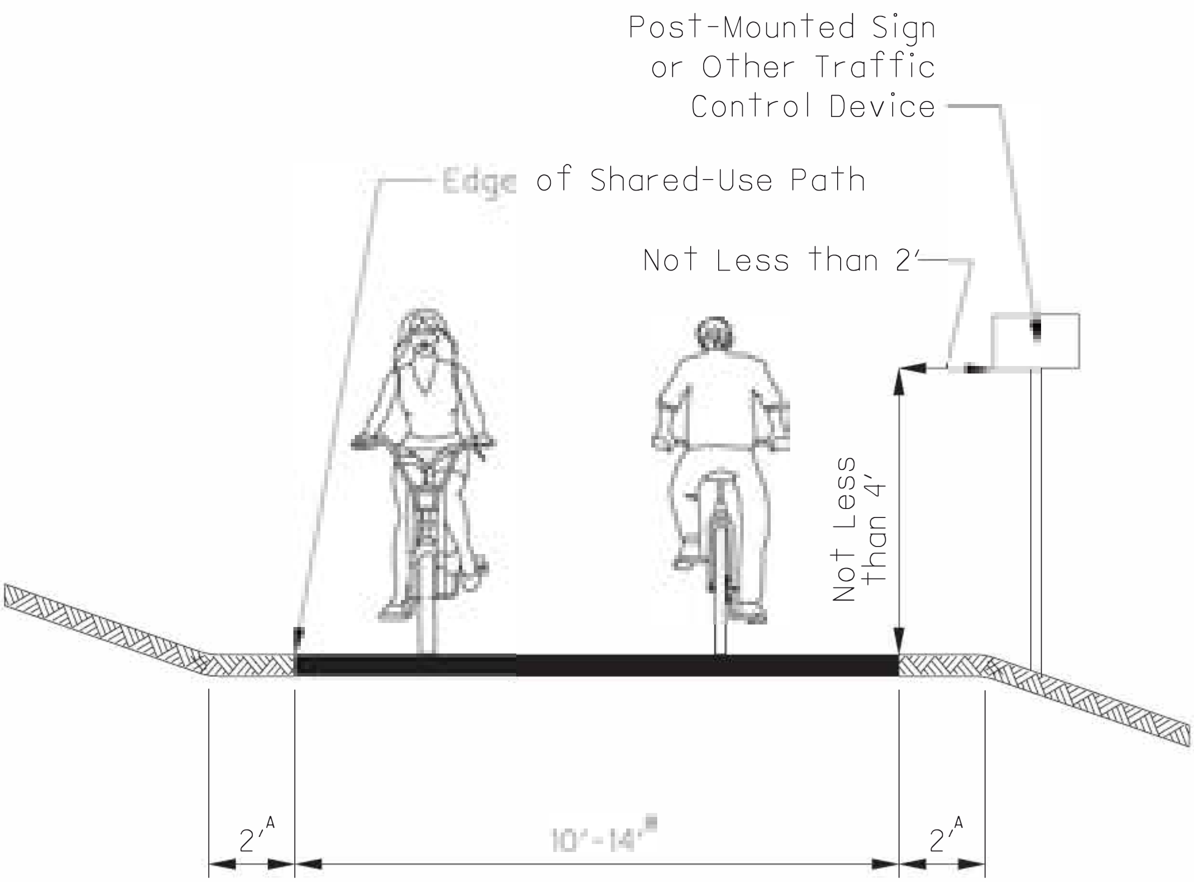
TYPICAL CROSS SECTION OF TWO-WAY SHARED USE PATH ON INDEPENDENT RIGHT-OF-WAY	701-1
	REFERENCE SECTIONS 702.2.1

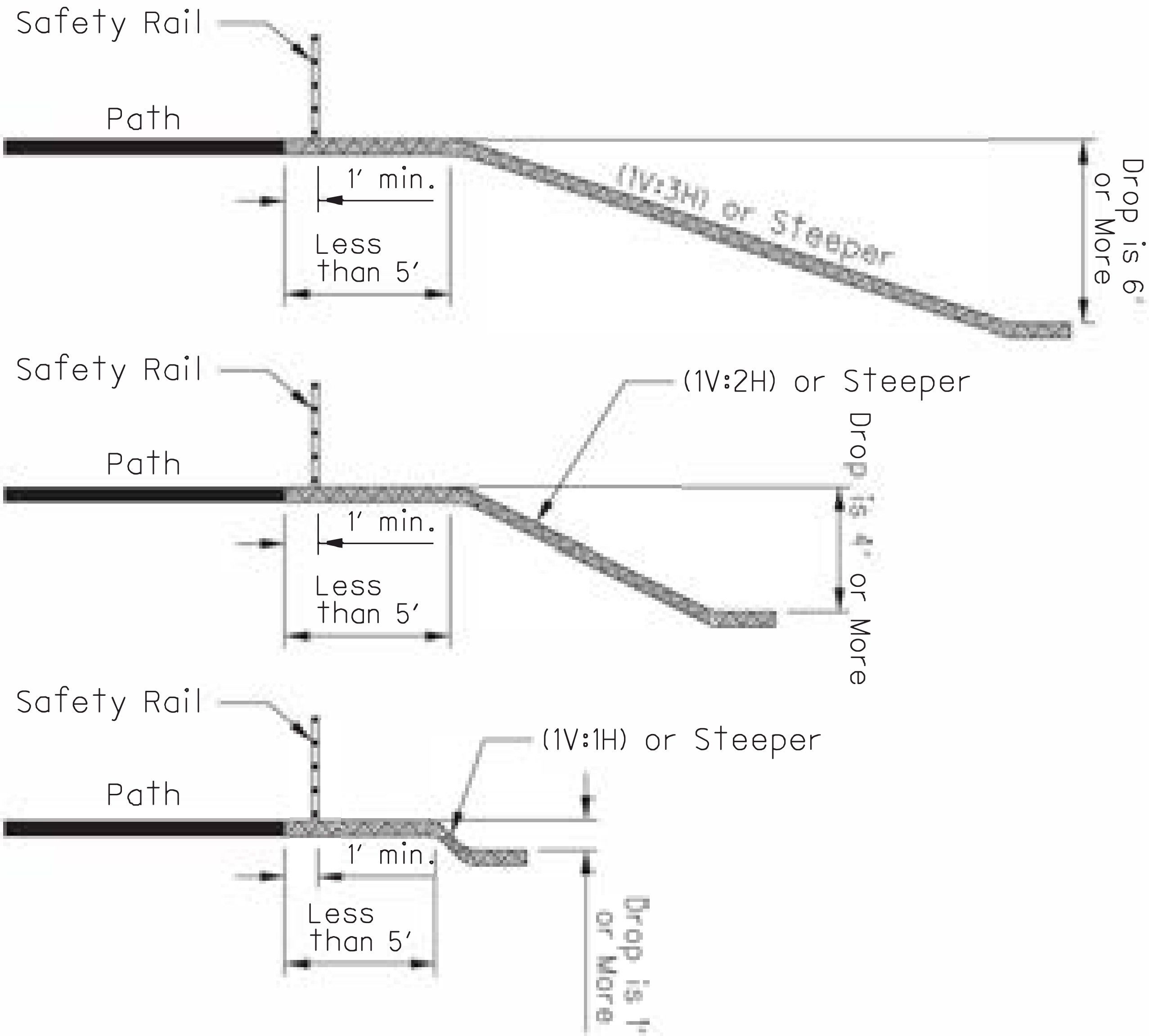


Notes:

A (1V:6H) Maximum Slope (typ.)

B More if necessary to meet anticipated volumes and mix of users, per the FHWA *Shared Use Path Level of Service Calculator*

SAFETY RAIL BETWEEN PATH AND ADJACENT SLOPE	701-2
	REFERENCE SECTIONS 702.2.1



MINIMUM LENGTH OF CREST VERTICAL CURVE BASED ON STOPPING SIGHT DISTANCE

701-3

REFERENCE SECTIONS 309.2.7

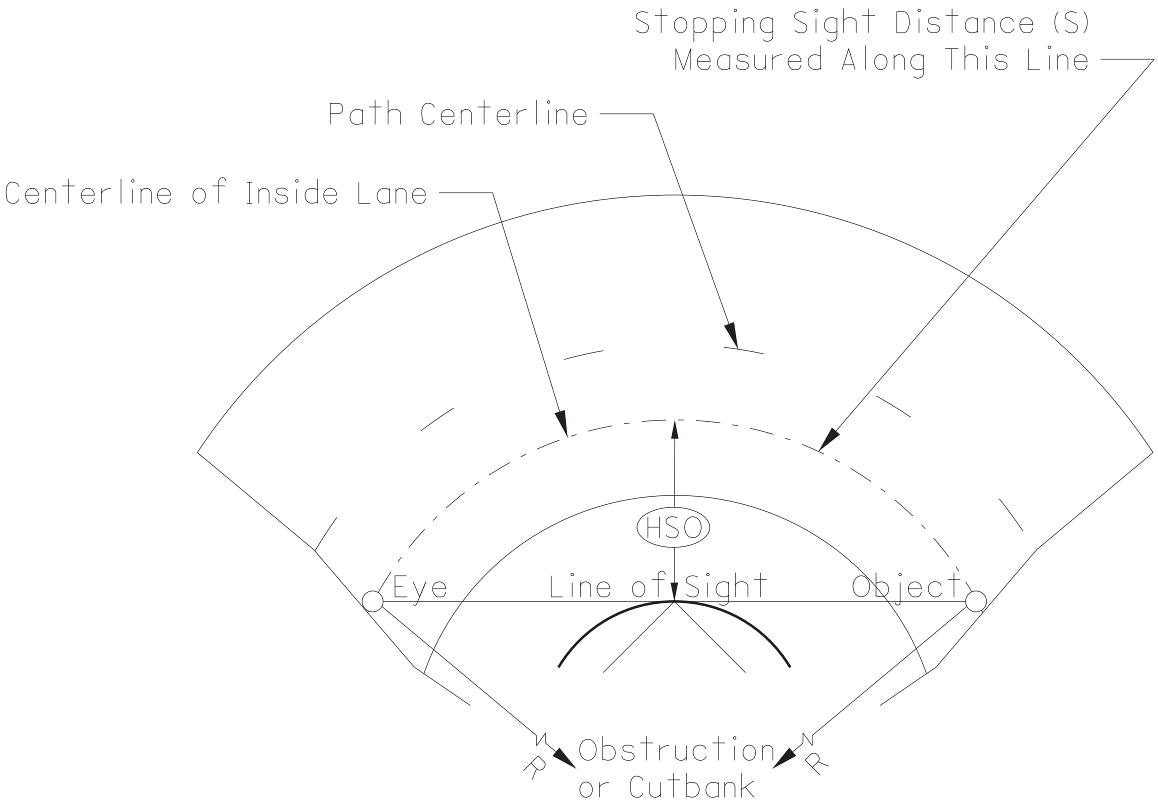
A	S = Stopping Sight Distance (ft)														
(%)	20	40	60	80	100	120	140	160	180	200	220	240	260	280	300
2												30	70	110	150
3								20	60	100	140	180	220	260	300
4						15	55	95	135	175	215	256	300	348	400
5					20	60	100	140	180	222	269	320	376	436	500
6				10	50	90	130	170	210	267	323	384	451	523	600
7				31	71	111	151	191	231	311	376	448	526	610	700
8			8	48	88	128	168	208	248	356	430	512	601	697	800
9			20	60	100	140	180	220	260	400	484	576	676	784	900
10			30	70	110	150	190	230	270	444	538	640	751	871	1000
11			38	78	118	158	198	238	278	489	592	704	826	958	1100
12		5	45	85	125	165	205	245	285	533	645	768	901	1045	1200
13		11	51	91	131	171	211	251	291	578	699	832	976	1132	1300
14		16	56	96	136	176	216	256	296	622	753	896	1052	1220	1400
15		20	60	100	140	180	220	260	300	667	807	960	1127	1307	1500
16		24	64	104	144	184	224	264	304	711	860	1024	1202	1394	1600
17		27	67	107	147	187	227	267	307	756	914	1088	1277	1481	1700
18		30	70	110	150	190	230	270	310	800	968	1152	1352	1568	1800
19		33	73	113	153	193	233	273	313	844	1022	1216	1427	1655	1900
20		35	75	115	155	195	235	275	315	889	1076	1280	1502	1742	2000
21		37	77	117	157	197	237	277	317	933	1129	1344	1577	1829	2100
22		39	79	119	159	199	239	279	319	978	1183	1408	1652	1916	2200
23		41	81	121	161	201	241	281	321	1022	1237	1472	1728	2004	2300
24	3	43	83	123	163	203	243	283	323	1067	1291	1536	1803	2091	2400
25	4	44	84	124	164	204	244	284	324	1111	1344	1600	1878	2178	2500

Shaded Area Represents S>L
Minimum Length of Vertical Curve = 3'

DIAGRAM ILLUSTRATING COMPONENTS FOR DETERMINING HORIZONTAL SIGHT DISTANCE

701-4

REFERENCE SECTIONS
309.2.7



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

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

Where:

S = Stopping Sight Distance (ft)
R = Radius of Centerline of Lane (ft)
HSO = Horizontal Sightline Offset, Distance from Centerline of Lane to Obstruction (ft)

Note:
- Angle is Expressed in Degrees
- Line of Sight is 2.3′ above Centerline of Inside Lane at Point of Obstruction

MINIMUM LATERAL CLEARANCE (HORIZONTAL SIGHTLINE OFFSET OR HSO) FOR HORIZONTAL CURVES

701-5

REFERENCE SECTIONS 702.2.7

S = Stopping Sight Distance (ft)															
R (ft)	20	40	60	80	100	120	140	160	180	200	220	240	260	280	300
25	2.0	7.6	15.9												
50	1.0	3.9	8.7	15.2	23.0	31.9	41.5								
75	0.7	2.7	5.9	10.4	16.1	22.8	30.4	38.8	47.8	57.4	67.2				
95	0.5	2.1	4.7	8.3	12.9	18.3	24.7	31.8	39.5	48.0	56.9	66.3	75.9	85.8	
125	0.4	1.6	3.6	6.3	9.9	14.1	19.1	24.7	31.0	37.9	45.4	53.3	61.7	70.6	79.7
155	0.3	1.3	2.9	5.1	8.0	11.5	15.5	20.2	25.4	31.2	37.4	44.2	51.4	59.1	67.1
175	0.3	1.1	2.6	4.6	7.1	10.2	13.8	18.0	22.6	27.8	33.5	39.6	46.1	53.1	60.5
200	0.3	1.0	2.2	4.0	6.2	8.9	12.1	15.8	19.9	24.5	29.5	34.9	40.8	47.0	53.7
225	0.2	0.9	2.0	3.5	5.5	8.0	10.8	14.1	17.8	21.9	26.4	31.3	36.5	42.2	48.2
250	0.2	0.8	1.8	3.2	5.0	7.2	9.7	12.7	16.0	19.7	23.8	28.3	33.1	38.2	43.7
275	0.2	0.7	1.6	2.9	4.5	6.5	8.9	11.6	14.6	18.0	21.7	25.8	30.2	34.9	39.9
300	0.2	0.7	1.5	2.7	4.2	6.0	8.1	10.6	13.4	16.5	19.9	23.7	27.7	32.1	36.7
350	0.1	0.6	1.3	2.3	3.6	5.1	7.0	9.1	11.5	14.2	17.1	20.4	23.9	27.6	31.7
390	0.1	0.5	1.2	2.1	3.2	4.6	6.3	8.2	10.3	12.8	15.4	18.3	21.5	24.9	28.5
500	0.1	0.4	0.9	1.6	2.5	3.6	4.9	6.4	8.1	10.0	12.1	14.3	16.8	19.5	22.3
565		0.4	0.8	1.4	2.2	3.2	4.3	5.7	7.2	8.8	10.7	12.7	14.9	17.3	19.8
600		0.3	0.8	1.3	2.1	3.0	4.1	5.3	6.7	8.3	10.1	12.0	14.0	16.3	18.7
700		0.3	0.6	1.1	1.8	2.6	3.5	4.6	5.8	7.1	8.6	10.3	12.0	14.0	16.0
800		0.3	0.6	1.0	1.6	2.2	3.1	4.0	5.1	6.2	7.6	9.0	10.5	12.2	14.0
900		0.2	0.5	0.9	1.4	2.0	2.7	3.6	4.5	5.6	6.7	8.0	9.4	10.9	12.5
1000		0.2	0.5	0.8	1.3	1.8	2.4	3.2	4.0	5.0	6.0	7.2	8.4	9.8	11.2

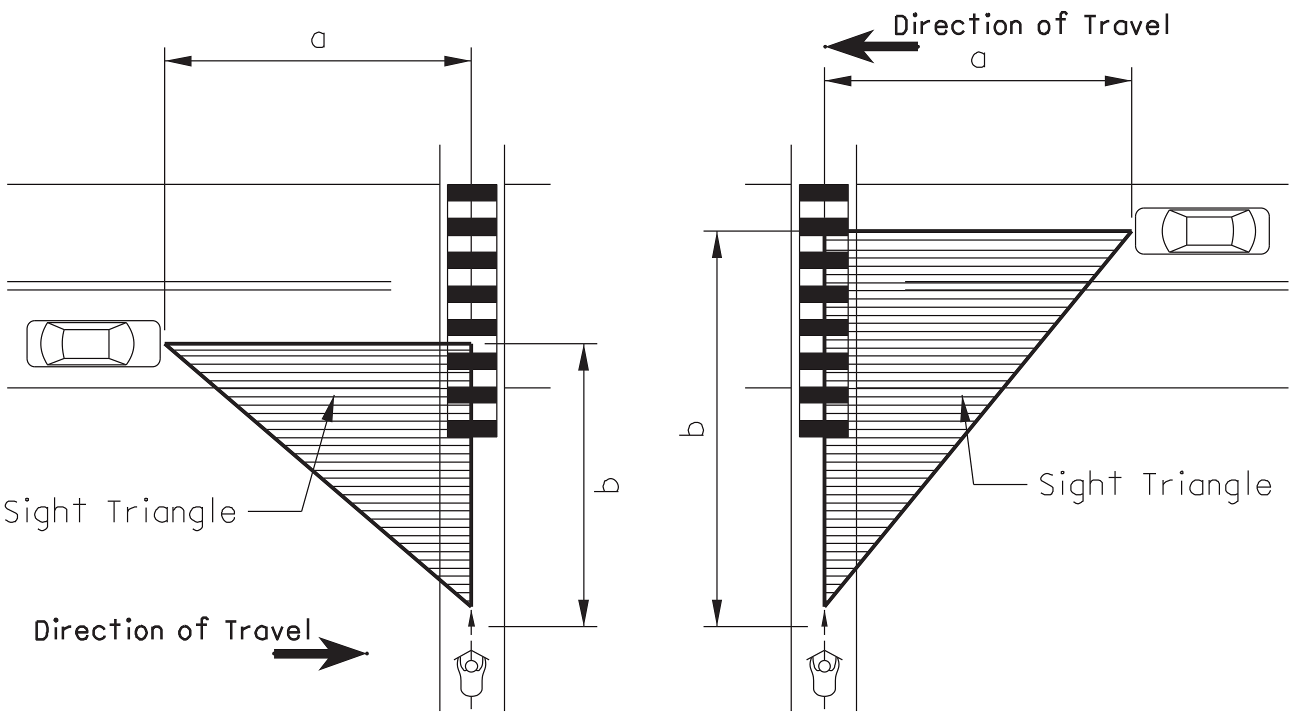
STOPPING SIGHT DISTANCE

701-6

REFERENCE SECTIONS
702.2.7, 702.3

Bicycle Design Speed (mph)	Stopping Sight Distance (Design Values)						
	No Grade Adjustment	% Down Grade			% Up Grade		
		3	6	9	3	6	9
8	43	46	51	60	41	40	38
10	58	63	71	85	55	52	51
12	75	81	93	113	70	66	64
14	93	102	117	145	86	82	78
16	113	125	145	181	104	98	93
18	134	150	175	221	123	116	110
20	157	176	207	264	144	135	127
25	222	253	301	390	202	187	176
30	298	341	411	539	268	247	231

YIELD SIGHT TRIANGLES	701-7
	REFERENCE SECTIONS 702.3



Length of Roadway Leg of Sight Triangle		
$t_a = \frac{S}{1.47V_{path}}$ $t_g = t_a + \frac{w+L_a}{0.278V_{path}}$ $a = 1.47V_{road}t_g$		
where:		
t_g	=	travel time to reach and clear the road (s)
a	=	length of leg sight triangle along the roadway approach (ft)
t_a	=	travel time to reach the road from the decision point for a path user that doesn't stop (s)
w	=	width of the intersection to be crossed (ft)
L_a	=	typical bicycle length = 6 ft
V_{path}	=	design speed of the path (mph)
V_{road}	=	design speed of the road (mph)
S	=	stopping sight distance for the path user traveling at design speed (ft)

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800 Access Control, R/W Use Permits and Drive Design

Table of Contents

800 Access Control, R/W Use Permits and Drive Design

801 ACCESS CONTROL

801.1 Access Control Directives

Access policies for highway right-of-way are as set forth in the following directives:

The State Highway Access Management Manual establishes procedures for processing permit applications, defines permissible uses of right of way for various highway classifications and establishes procedures and standards for access provisions adjacent to major commercial and industrial developments.

It is intended that ***Section 800*** of this manual supplement the State Highway Access Management Manual with respect to access policies and R/W use permits as well as provide the designer with the criteria necessary to design most types of drives.

801.2 Access Control Policies

The policy of permitting access on highways is summarized below:

801.2.1 Interstate Limited Access

Direct access to an Interstate Highway will not be permitted except as outlined in ***Sections 801.2.6*** and ***801.2.7***. All crossroads and railroad grades shall be separated.

801.2.2 Limited Access

If a highway is now, or is designated to be an ultimate fully limited access freeway and access rights have been acquired:

1. If the highway has no existing private access points, direct private access to such highway will not be permitted.
2. If the highway has existing private access points and the ultimate freeway design has been determined, temporary access improvements may be permitted. However, at the time the improvement is permitted, the method for deleting the temporary access points must be determined and necessary agreements made with the property owner to facilitate their deletion in the future.
3. If the highway has existing private access points and the ultimate freeway design has not been established, modifications of existing access will not be permitted until the ultimate freeway design has been determined.
4. Provision generally shall be made for future separation of crossroads and railroad grades by purchase of right-of-way as a part of the initial project.

801.2.3 Controlled Access Highways

Modifications of existing points of access or changes from one location to another within the limits of the applicant's property may be permitted, if such modification or change would be beneficial to both the highway operation and property development. However, new additional points of access will not be permitted. Crossroads and railroads need not be separated unless very high volumes dictate its consideration.

801.2.4 Non-Limited Access Highways

Access to a non-limited access highway is permissible in accordance with the State Highway Access Management Manual. However, such access is subject to the conditions prescribed by the Director of Transportation under authority granted by ***Section 5515.01*** **of the Ohio Revised Code**.

801.2.5 Interchange Controls

No access shall normally be allowed on intersecting highways adjacent to highway interchanges for a minimum of 600 feet at diamond-type interchanges and 1,000 feet at other types of interchanges. This distance applies to each direction along the intersecting highway, measured from the outer-most ramp terminal intersections with the highway. See ***Figures 801-1*** and ***801-2*** for additional details.

801.2.6 Locked-Gate Access to Freeways and other Limited Access Highways

Locked-gate access to freeways and other limited access highways is considered an access point and requires documentation of the proposal for approval. Locked-gate access to interstates for purposes other than roadside maintenance requires submission to and approval by FHWA. The Office of Roadway Engineering will evaluate submissions for freeways and other limited access highways, and will coordinate submissions with FHWA for interstates.

The typical purpose for locked gate access is for emergency access or roadside maintenance from areas outside of the highway right-of-way. Since the gate is intended for a few select users, they should be inconspicuous to the general travelling public with limited improvements. Key consideration in the location and design of locked-gate access are sight distance where vehicles will be entering the freeway and acceleration of the entering vehicles. The proposal document should clearly describe to whom the access is granted, how the access will be secured, and maintenance responsibilities. Locked-gate access for purposes other than roadside maintenance must be sponsored by a public agency.
Locked-gate access to interstate highways for the purpose of roadside maintenance do not require FHWA submission and approval.

801.2.7 Temporary Construction Access to Freeways and other Limited Access Highways

Temporary access to the construction site by way of an L/A break that **DOES NOT** provide access for construction vehicles to the travel lanes shall conform to CMS 107.10 and can be approved by the District Construction Engineer, the District Real Estate Administrator and the District Environmental Coordinator.

Temporary access to the construction site by way of an L/A break that **DOES** provide access for construction vehicles to the travel lanes shall conform to CMS 107.10 and shall require the approval of the District Construction Engineer, the District Real Estate Administrator and District Environmental Coordinator prior to submission to the Office of Roadway Engineering for approval. The Office of Roadway Engineering will also coordinate with the FHWA for approval of the temporary access L/A breaks that provide access to and from the travel lanes of the Interstate.

Approvals for all Temporary Construction Accesses through Limited Access Right of Way will expire at the conclusion of the project construction and the right of way will be fully restored, including replacement of all fencing, as applicable.

802 HIGHWAY USE PERMITS

802.1 General R/W Use Criteria

802.1.1 Approvals and Agreements

Permission to use highway R/W is required for fencing, storm sewers, sanitary sewers, public utilities, points of access, or other similar types of work. ODOT does not allow non-ODOT agencies to use ODOT right of way for the purpose of locating stormwater Best Management Practices (BMPs). When a request is made to alter, modify or otherwise use highway R/W, Federal and/or State approvals must be obtained and the necessary agreements or permits between the State and applicant must be completed before any work can be initiated.

802.1.2 Authority

Permits for the use or occupancy of State Highway right-of-way may be granted, upon formal application, by the Director of Transportation. Such permits, when granted, shall be subject to the policies and regulations set forth herein under authority granted by ***Section 5515.01*** **of the Revised Code of Ohio**.

802.1.3 Application Procedures

The procedure for applying for permits is included in the State Highway Access Management Manual.

802.1.4 Right-of-Way Use Prohibitions

No parking, servicing of vehicles, erection of lights, signs or other advertising devices will be permitted on highway right-of- way. Similarly, no device or structure will be permitted to overhang highway right-of-way. Provisions should be made in the design of driveways or approaches on rural highways so that a vehicle will not be required to back onto the right-of-way or highway pavement to gain access to the highway.

802.1.5 Future Highway Improvement Controls

When granting permits, consideration should be given to the extent of future highway improvements. The location and design of driveways or public road approaches should then be governed by the general access criteria (***Section 802.2***) of the future highway facility.

802.1.6 Drainage Considerations

When any owner or developer of land adjacent to highway R/W proposes to route site drainage into the highway drainage system, see the Location and Design Manual, Volume 2, Section 1001.4 for guidance.

802.2 General Access Criteria

802.2.1 Highway Access Considerations

The basic considerations that govern the location and design of highway access shall be to facilitate:

1. The safe and expeditious movement of vehicles on the street or highway.
2. The provision of the best service possible to the private or public facility being served by the drive access.
3. The safe movement of pedestrian traffic.

802.2.2 Median Openings

Median openings are normally not permitted on divided highways. Exceptions may be for public roads or streets or traffic generators such as large shopping centers or industrial plants, if satisfactorily justified and in the public interest.

If a median opening exists prior to the construction of a drive, the opening may be further modified, including relocation, to accommodate the turning movements of the expected traffic. The design modifications shall, however, be consistent with the overall design of the highway.

802.2.3 Added Highway Lanes

The construction of an additional lane adjacent to the existing highway lanes to serve as right or left turn lanes may be permitted if benefit to the operation of the through highway will result. The design of any added lane must be consistent with the overall design of the highway and ***Section 401.6***.

802.2.4 Number of Drives Permitted and Their Location

Refer to Section 4.0 of the State Highway Access Management Manual for the acceptable access locations and the number permitted.

802.2.5 Joint Drives

A jointly owned drive may be permitted upon joint application by both property owners.

802.2.6 Location of Drive in Relation to Side Property Line

Figure 802-1 shows the controls for locating drives in relation to side property lines.

1. Controls
    a. 90º Control Line - a line at right angles to the centerline of the highway which extends through the intersection of the side property line with the highway right-of-way line.
    b. 4-foot Control - maximum width of driveway approach flare as measured along the 90º control line from the highway edge of traveled way.

2. Curbed Highways - the approach radius may begin at the intersection of the 90º control line with the highway edge of traveled way but may not cross the 90º control.

3. Uncurbed Highways - the approach radius, but not the approach edge extension, may cross the 90º control line within the limits of the 4-foot control.

A permit may be issued for the construction of a driveway which encroaches on the abutting property frontage in excess of the controls set forth above only when written permission from the affected property owner is presented and made a part of the State's record of the permit, and only when such encroachment does not interfere with an existing driveway. It shall be the responsibility of the permit applicant to make all necessary arrangements and agreements with the affected property owners when the relocation of existing driveways is necessary. The expense involved shall be borne by the applicant.

802.2.7 Location of Drive in Relation to an Intersection

For acceptable locations of driveways near intersections, see Section 4.0 of the State Highway Access Management Manual.

802.2.8 Drive Sight Distance

Wherever possible, drives should be located in accordance with the intersection sight distance criteria in ***Section 201.3***.

802.2.9 Drives with Restricted Movements

In some situations, it is desirable to control or prohibit certain movements through the use of median islands or channelizing islands. Median islands divide the ingress and egress movements and are used to prevent cross movements of internal traffic (See ***Section 304.3.2***, ***Section 401.7.3*** and ***Section 803.6***). Channelizing islands are used to control and direct turning movements on a driveway approach (See ***Section 401.7.2***).

803 DRIVE GEOMETRIC DESIGN

803.1 Mailbox Facilities

803.1.1 Mailbox Supports

Mailbox installations located within the clear zone shall be installed as shown in ***Figure 803-1*** using "breakaway" type supports. Satisfactory supports are as follows:

1. Maximum 4 inches by 4 inches square or 4½ inch diameter round timber.
2. Maximum 2 inch diameter (2-3/8" O.D.) Schedule 40 standard strength steel pipe.
3. Any material with breakaway cross section characteristics equivalent to 1 or 2 above.

Group mailbox supports should be placed on three foot centers and the turnout lengthened to accommodate the grouping. No more than two mailboxes shall be placed on each post.

Where guardrail exists, mailboxes and their supports should be located behind the guardrail. Supports must still meet the breakaway requirements listed above.

803.1.2 Mailbox Turnouts

Where the existing or proposed highway shoulder paving is less than 6 feet wide, mailbox turnouts should be provided as shown in ***Figure 803-1*** and ***SCD BP-4.1***. Mailbox turnouts should be constructed of the same material used in the drive approach and combined with the drive approach where possible.

803.2 Rural Residential and Field Drives

Rural residential drives and field drives should normally conform to the Type 1 design shown in **SCD BP-4.1**.

803.2.1 Drive Intersection Angle

New drives should intersect the highway at an angle between 70º and 90º. However, in some cases, it may be necessary to retain existing drive angles that vary from these desirable angles.

803.2.2 Drive Widths

If the project involves existing drives, the existing width is normally retained unless it is less than 12 feet. In which case, it should be widened to provide a 12 foot throat width. In the case of new drives, the width should normally be 12 feet. If the new driveway is a combined drive between two properties, the width should normally not exceed 24 feet. Also, a wider field drive may be used if it will keep the farm equipment operator from encroaching on the opposing traffic lane when entering or exiting the highway.

803.2.3 Drive Radii

The radii of the Type 1 driveway should normally be 25 feet. The radii may be increased on field drives if it is deemed that the larger values will improve driveway operation and reduce the hazard to the motorists and farm equipment operator.

803.2.4 Curbed Drives

Driveways abutting uncurbed highways may be curbed. However, the curb shall not extend closer to the mainline edge of traveled way than 8 feet or the treated shoulder width, whichever is greater, to avoid curb obstruction for vehicles, snowplows, etc., using the shoulder and be transitioned in height per ***Section 305.4.1***.

803.3 Urban Residential Drives

Either Type 1 or 2 drives, shown in **SCD BP-4.1**, may be used in urban areas. If used in urban areas, the radii and flare dimensions may be reduced so that the apron does not extend past the back of the sidewalk, or past the right-of- way line if there are no sidewalks. The desirable minimum radii for Type 1 drives, when the through highway is curbed, is 15 feet.

Shown on ***Figure 803-2*** are three methods for designing driveways between the curb line and sidewalk to provide for turning vehicles. Other designs, may be used if they are approved for use by the local governmental agencies responsible for maintenance of the project. Additional details are shown in ***Figure 803-3*** when the tree lawn is less than 6 feet. Residential drives on curbed streets should use a dropped curb as shown in ***Section B-B on Figure 803-2***.

803.4 Service Station Drives

--Section Deleted--

803.5 Commercial Drives

The access requirements of most commercial developments can be served by driveways having standard design characteristics. The exceptions are driveways having high traffic volumes, those being used by large vehicles, or those serving businesses which have traffic patterns unique to the business being conducted.

803.5.1 Standard Commercial Drives (See Figure 803-8)

1. Radii:
    a. 15 foot minimum, when the through highway is curbed.

    b. 25 foot minimum, when the through highway is uncurbed.

    c. Maximum should be based upon the design vehicle turning template.

2. Width - 35 foot maximum

3. A dropped curb should be used on curbed streets as shown in ***Section B-B*** **on** ***Figure 803-2***.

803.5.2 Exceptions to Standard Commercial Drives

Where access requirements are such that a non-standard driveway is necessary, the design may approximate the design of shopping center driveways as discussed in ***Section 803.6*** or public road intersections, ***Section 401***.

Specially designed radii and a width greater than 35 feet may be permitted, as necessary, to accommodate the type vehicle using the driveway. (Example: A truck stop may require two one-way driveways or a single drive with width greater than 35 feet and radii as great as 75 feet to facilitate turning movements).

803.6 Shopping Center and Industrial Drives (See Figure 803-9)

This section is intended as a guide for the design of driveways to high volume traffic generators such as shopping centers, industrial plants, industrial parks, and other types of developments having similar traffic characteristics. Many of the design features discussed in ***Section 401***, Intersections At-Grade, will be applicable. Geometric considerations are listed below:

1. Driveways should intersect the highway at an angle between 70ºand 90º.

2. Each driveway traffic lane should have a minimum width of 10 feet, with 12 feet preferred.

3. Major driveways in shopping centers should be constructed to prevent cross movement of internal traffic within 100 feet of the entrance approach. This may be accomplished by use of a raised divider, 6 inches high, 6 feet wide (min.) and 100 feet long, and/or by use of curbing, sidewalk or other barrier along the drive edges for a length of 100 feet (See ***Figure 803-9***).

4. Driveways designed for traffic signal operation should have curbed radii and should provide a minimum of two lanes for vehicles entering the highway.

804 DRIVE PROFILE DESIGN

804.1 Drive Profiles (Uncurbed Roadways)

Drive profiles on uncurbed roadways shall slope down and away from the edge of traveled way at the same slope as the graded shoulder. Any vertical curve should be developed outside the normal graded shoulder width. Vertical curve lengths should be 10 feet to 20 feet, depending on the grade differential. Under normal circumstances, rural drive grades should not exceed 10 percent with 8 percent considered to be the preferred maximum.

804.2 Residential Drive Profiles (Curbed Roadways)

The design vehicle used to develop the profile criteria of this section is shown on ***Figure 803-2***. The profile criteria shown provides clearance for this vehicle when its springs are completely compressed. If conditions of a particular driveway do not meet the cross-section criteria listed below, a template of the design vehicle can be used to design the driveway profile.

For tree lawns 6 feet or wider, the ramp grade from the gutter to the edge (the ramp cross-slope rate from the gutter to) of the sidewalk will be 1 inch per foot or less for normal cross-section design. ***Figure 803-2*** shows this condition for the following cross-section conditions:

1. Sidewalk and tree lawn slope of 1/4 inch per foot, and

2. 6 inch height of curb with pavement slope of 3/16 inch per foot or 1/4 inch per foot , or

3. Type 2 curb and gutter with pavement slope of 3/16 inch per foot.

If the cross-section design does not meet the above conditions (has sharper grade breaks), the profile should be designed using a template of the design vehicle.

For tree lawns less than 6 feet wide, ***Figure 803-3*** shows the profile treatment. Clearance for the design vehicle is achieved by depressing the sidewalk 1 inch at the driveway. The sidewalk cross-slope of 1/4 inch per foot is retained. The design may be used directly with curbed highways having cross-section criteria as listed above and the profile conditions of ***Figure 803-2***. For other cross-sections, a template of the design vehicle may be used to design the profile.

Figure 803-3 shows an isometric view and profile for a driveway where only a 3-foot tree lawn is available. This design is shown, not because it is desirable, but because right-of-way width and property development may require this type of design. Whenever feasible, the tree lawn should be 8 feet or wider, as discussed in ***Sections 306.2.4 and 306.2.5***.

Where the total width of tree lawn and sidewalk is less than 7 feet, the minimum 3-foot apron designs are inappropriate, and cannot be used, as they extend curb or sharp flares into the sidewalk area. For this condition, the sidewalk and curb are transitioned to meet the drive profile as shown on the lower portion of ***Figure 803-3***. The profile of the drive meets the 1 inch depressed grade of the sidewalk as shown in the drive profile of ***Figure 803-3***.

The tree lawn and walk design shown in ***Figures 803-2*** and ***803-3*** will keep storm water, flowing at the curb design height or less, from flowing over the sidewalk. If it is necessary to lower the curb and sidewalk more than 1 inch, the drainage condition should be checked thoroughly.

804.3 Commercial Drive Profiles (Curbed Roadways)

Commercial drive profiles usually use a dropped curb across the approach. However, some commercial drives serving large traffic generators may be designed as at-grade intersections, without dropped curbs, because of their high traffic volumes.

Shown on ***Figure 804-1*** are the grade controls for commercial driveways. The grade should be as flat as possible and still meet drainage requirements. The 20-foot length between grade breaks is required by the low clearance and the long axle spacing of the commercial design vehicle (***Figure 804-2***). Tree lawn profile design should be in accordance with ***Figures 803-2*** and ***803-3***. The grade break at the face of the curb is critical for some commercial vehicles and the cross-section requirements for residential drives on curbed streets should be used.

805 DRIVE PAVEMENT DESIGN

805.1 Field Drives

Field driveways should be paved with 6 inches of 411 or 304 aggregate. They shall be paved from the edge of traveled way or treated shoulders, to a point where the grade of the new driveway intersects the grade of the existing driveway, or on relocated driveways to where the grade of the new driveway intersects the existing ground.

805.2 Residential Drives

Residential driveways shall be paved from the edge of new pavement to the point where the grade of the new driveway intersects the grade of the existing driveway, or on relocated driveways to the point where the geometric limits of the new driveway meet the existing driveway.

Residence driveways having an existing hard surface or an existing aggregate surface shall be replaced with a pavement of a similar type, insofar as practicable, using one of the following designs for the portion beyond the flared apron:

1.	6 inches	452	Non-Reinforced Concrete Pavement
2.	2 inches	441	AC Surface Course, Type 1, (448), PG64-22
	6 inches	304	Aggregate Base (or 411 Stabilized Crushed Aggregate)
3.	1.25 inches	441	AC Surface Course, Type 1, (448), PG64-22
		407	Tack Coat
	3.5 inches	301	Asphalt Concrete Base, PG64-22
4.	8 inches	304	Aggregate Base
			(or 411 Stabilized Crushed Aggregate)

For the Item 407 Tack Coat application rate, see CMS 407.06

The Item 411 Asphalt Concrete may be changed to match the asphalt concrete material specified on the adjacent pavement.

In uncurbed areas, the apron pavement design depends on the treated shoulder material as follows:

1. The flared portion of residence driveways adjacent to paved shoulders shall be constructed of the same material and composition as used in the treated shoulder paving.

2. The flared portion of residence driveways adjacent to surface treated aggregate shoulders shall be constructed of the same material as used in the treated shoulder, except it shall be surfaced with 2 inches of 441 Asphalt Concrete, Type 1, (448), PG64-22.

3. The flared portion of residence driveways on projects for which earth shoulders are specified shall be paved with either 6 inches 452 Non-Reinforced Concrete Pavement, or with 2 inches of 441 Asphalt Concrete, Type 1, (448), PG64-22 on 6 inches of 411 or 304 aggregate.

805.3 Commercial Drives

Commercial driveways shall be paved from the edge of the new pavement to the point where the grade of the new driveway intersects the grade of the existing driveway, or on relocated driveways to the point where the geometric limits of the new driveway meet the existing driveway.

Commercial driveways having an existing hard surface or aggregate surface shall be replaced with a pavement of a similar type insofar as practical, using one of the following designs for the portion beyond the return or apron:

1.	8 inches	452	Non-Reinforced Concrete Pavement
2.	1.25 inches	441	AC Surface Course, Type 1, (448), PG64-22
		407	Tack Coat
	1.75 inches	441	AC Intermediate Course, Type 2, (448)
	8 inches	304	Aggregate Base
3	1.25 inches	441	AC Surface Course, Type 1, (448), PG64-22
		407	Tack Coat
	5 inches	301	Asphalt Concrete Base, PG64-22

For the Item 407 Tack Coat application rate, see CMS 407.06.

The Item 441 Asphalt Concrete may be changed to match the asphalt concrete material specified on the adjacent pavement.

Additional thicknesses may be provided for the above courses where unusual weights or types of vehicles are expected to use the commercial driveway.

Commercial driveway aprons shall be constructed as previously outlined for residential driveway aprons, except that additional thicknesses should be provided to meet nominal pavement design for commercial driveways.

805.4 Pavement Treatment of Undisturbed Drives

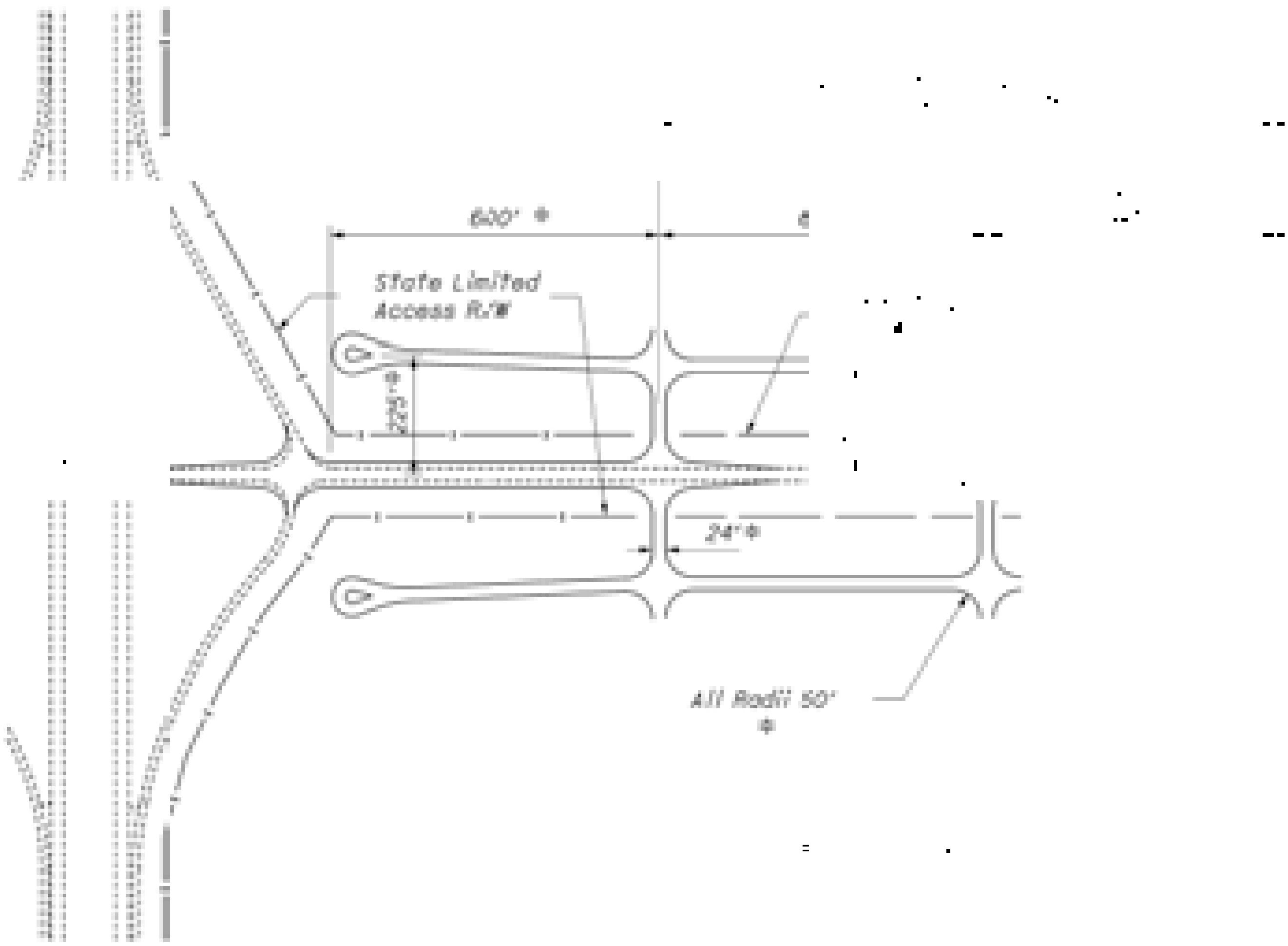
The preceding treatment of driveways does not apply to resurfacing or widening and resurfacing projects when the existing driveway is not disturbed beyond the edge of proposed pavement. Item 411 or 304 aggregate shall be used to adjust aggregate driveways to meet the new pavement surface for widening and/or resurfacing projects. Asphalt concrete shall be used for adjusting bituminous or concrete driveways to meet the new pavement surface, which adjustment shall be accomplished within a reasonable distance from the edge of the pavement. As a general rule, this can be done within the limits of the roadway shoulders.

LIST OF FIGURES

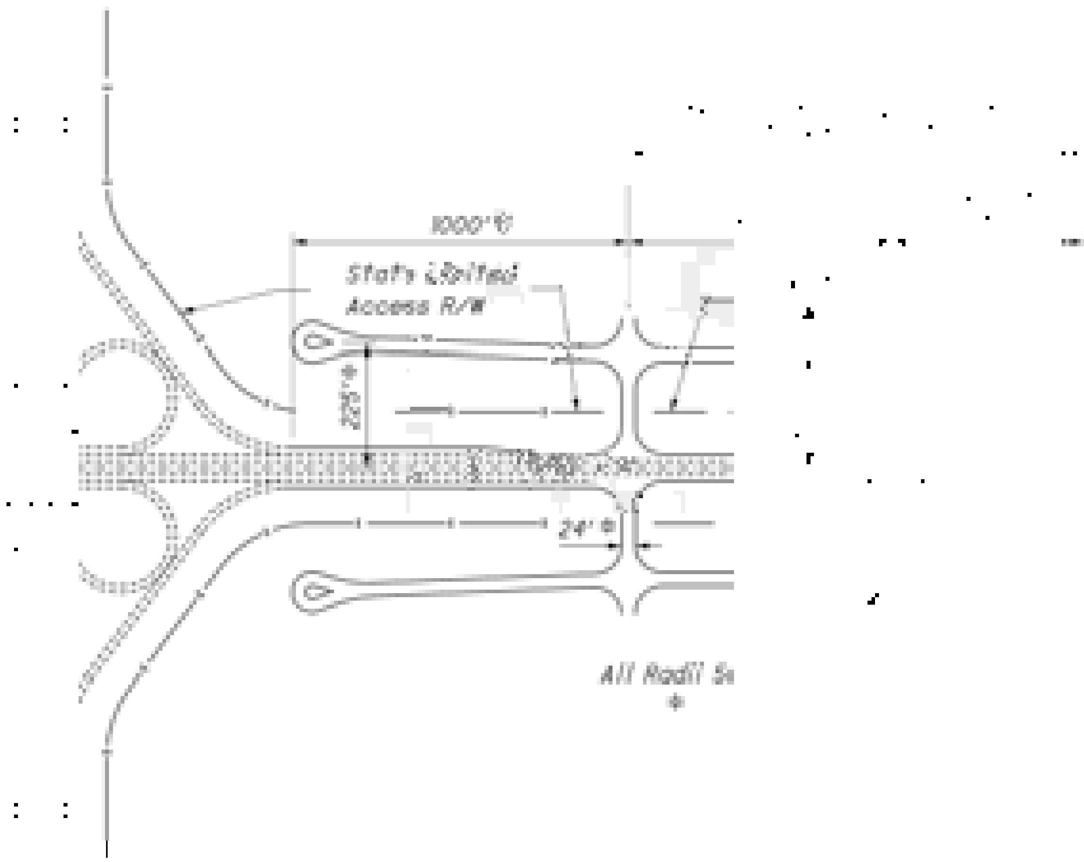
GUIDELINES FOR LIMITATION OF ACCESS AT DIAMOND TYPE INTERCHANGES

801-1

REFERENCE SECTION **801.25**

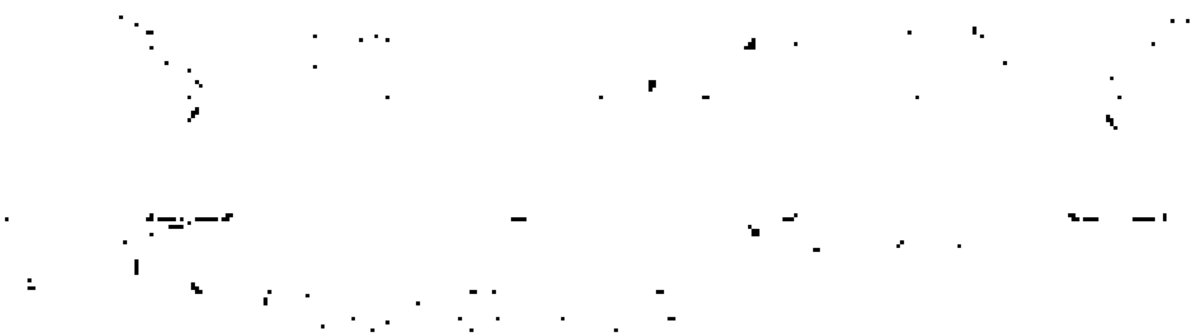


GUIDELINES FOR LIMITATION OF ACCESS AT CLOVERLEAF-TYPE INTERCHANGES	801-2
	REFERENCE SECTION 801.25

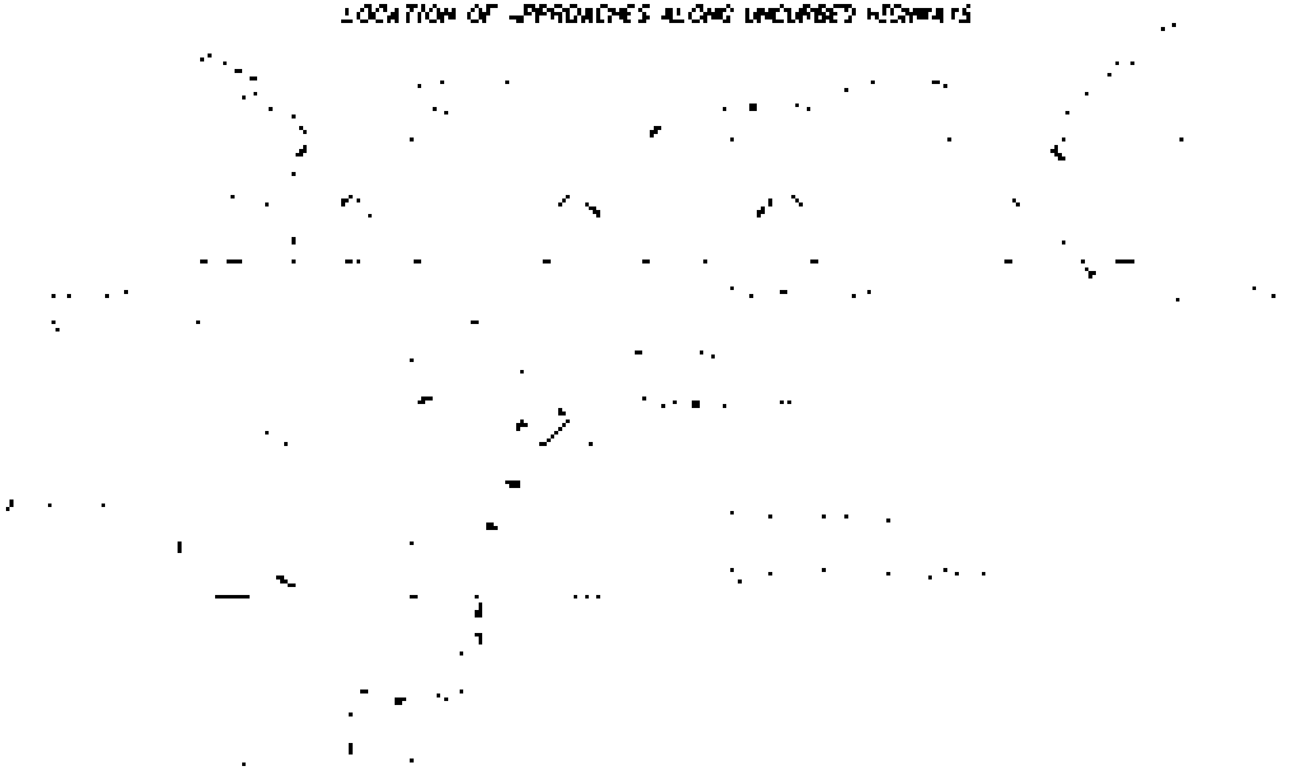


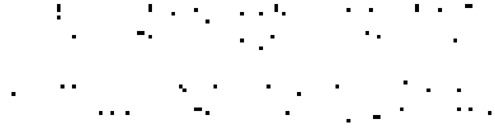
LOCATION OF DRIVES IN RELATION TO PROPERTY LINES	802-1
	REFERENCE SECTION 802.26

LOCATION OF APPROACHES ALONG CURBED HIGHWAYS



LOCATION OF APPROACHES ALONG UNCURBED HIGHWAYS



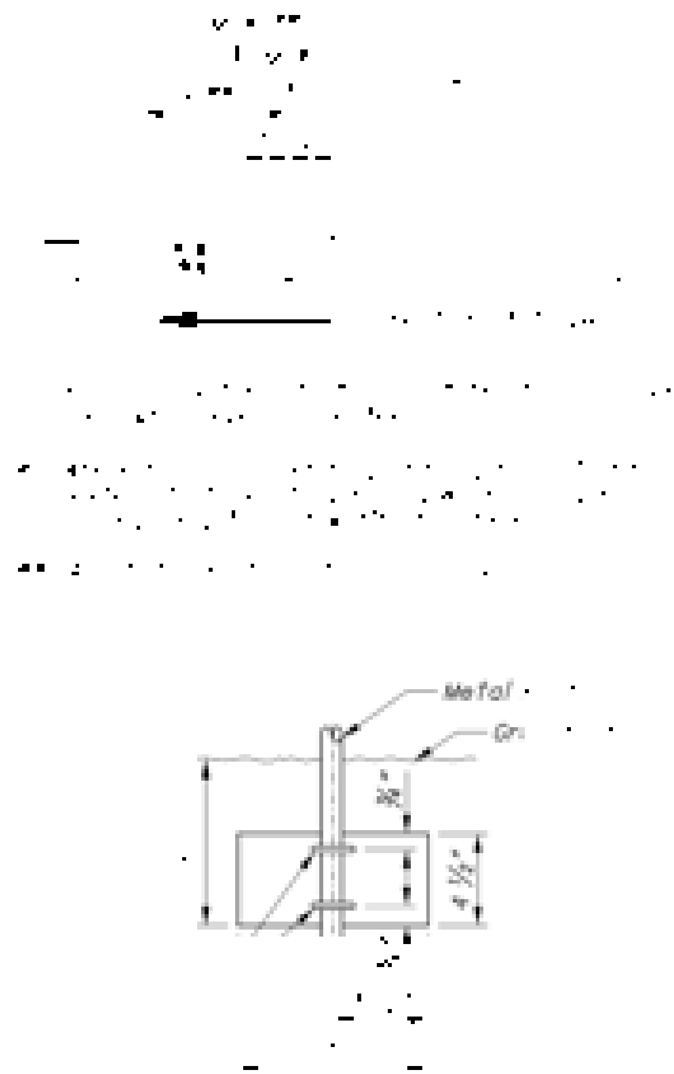


DETAIL FOR DETERMINING MAXIMUM UTILIZATION OF HIGHWAY FRONTAGE

MAILBOX FACILITIES

803-1

REFERENCE SECTION **803.1**

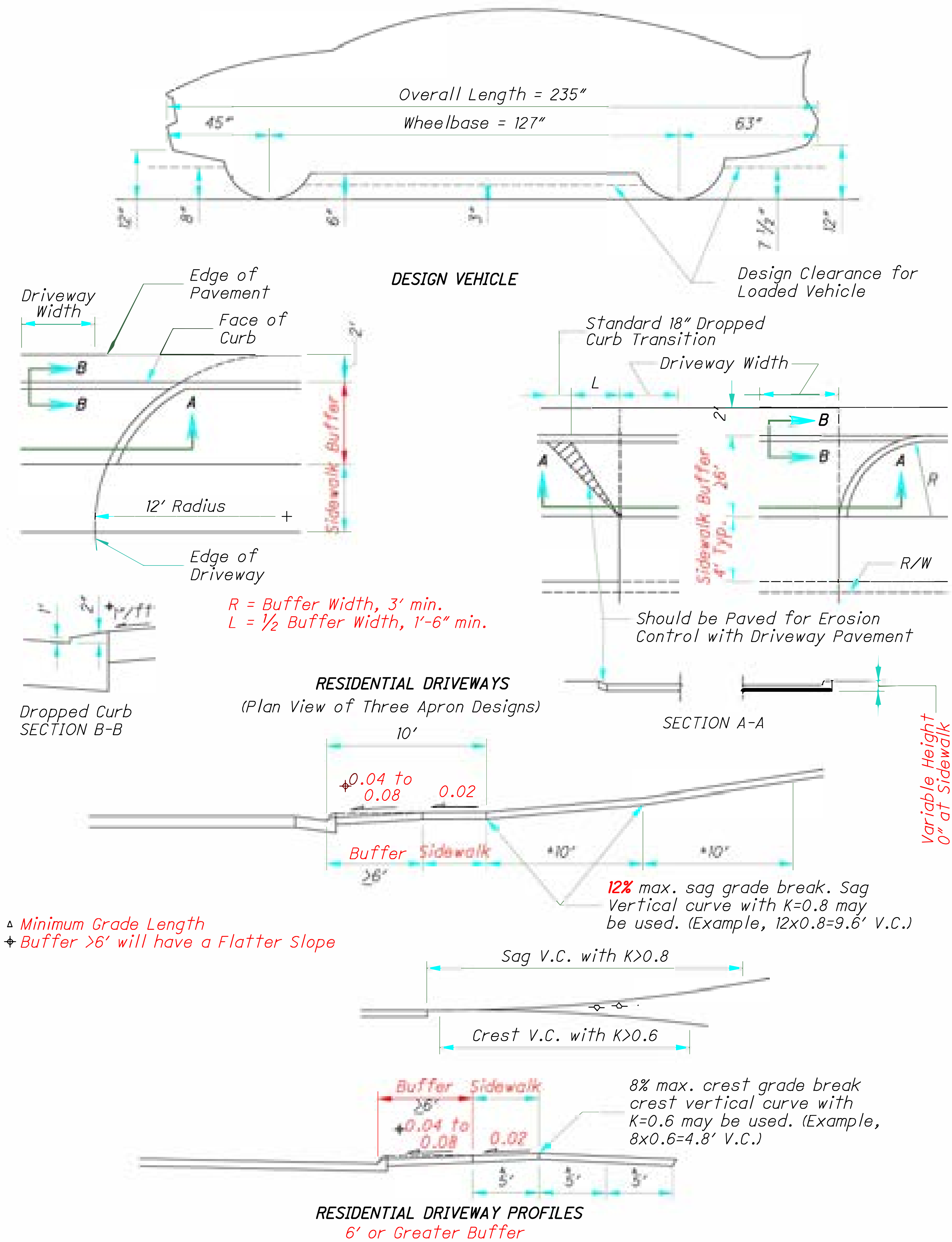


ANTI-TWIST PLATE

GROUP MAILBOX INSTALLATION

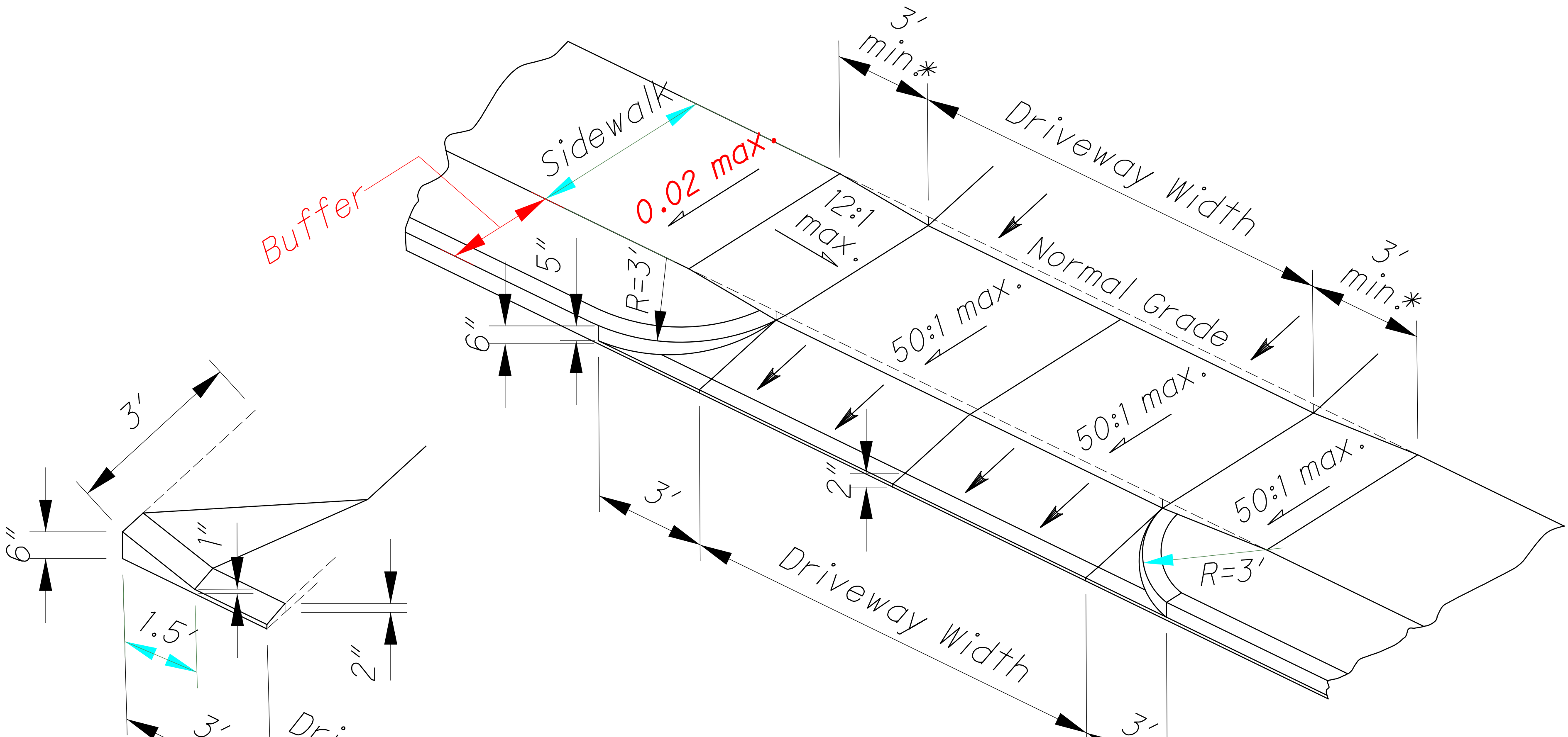
January 2020

URBAN RESIDENTIAL DRIVE DETAILS	803-2
	REFERENCE SECTION 803.3 & 804.2



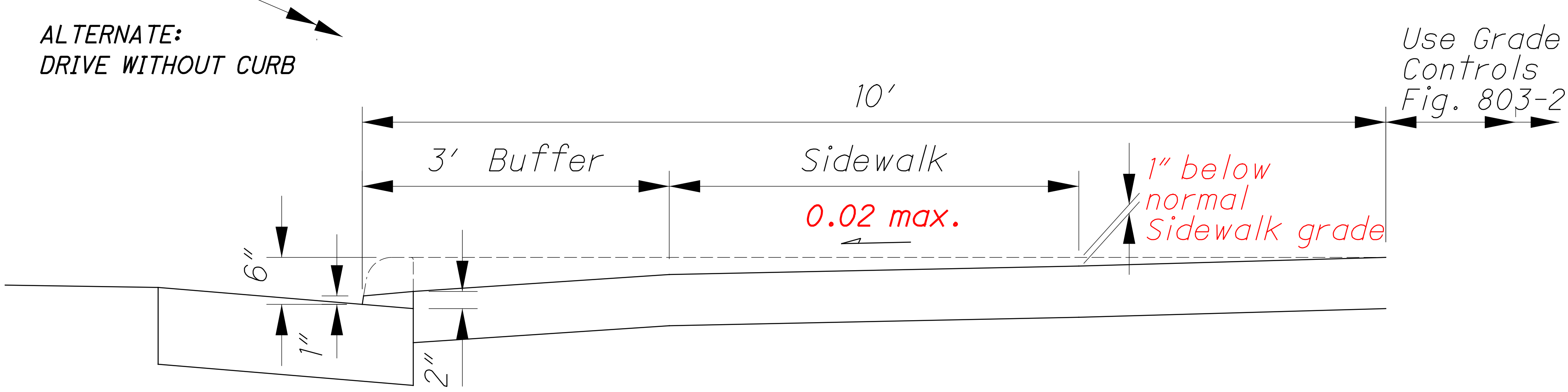
January 2020

URBAN RESIDENTIAL DRIVE DETAILS	803-3
	REFERENCE SECTION 806.2, 803.3, 804.2



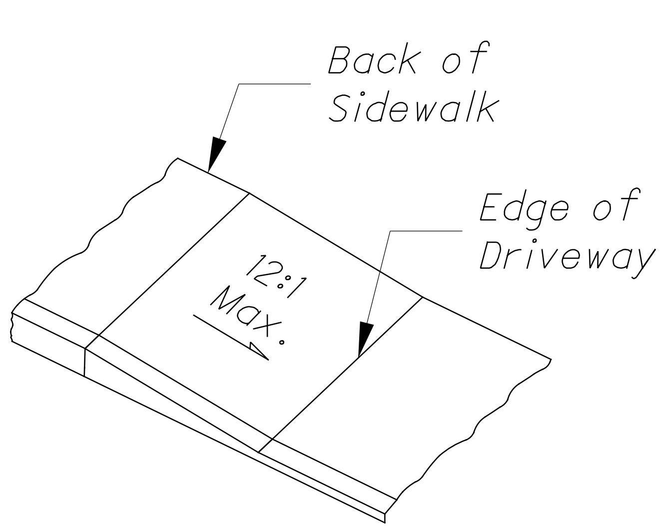
ISOMETRIC VIEW

ALTERNATE:
DRIVE WITHOUT CURB

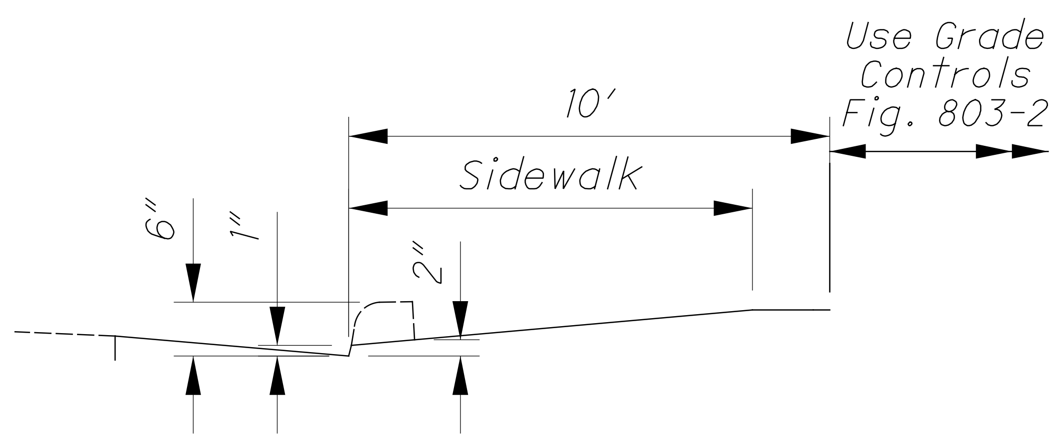


DRIVEWAY PROFILE

Recommended Minimum Apron Design
(3' Buffer with 1" depressed sidewalk)



ISOMETRIC VIEW

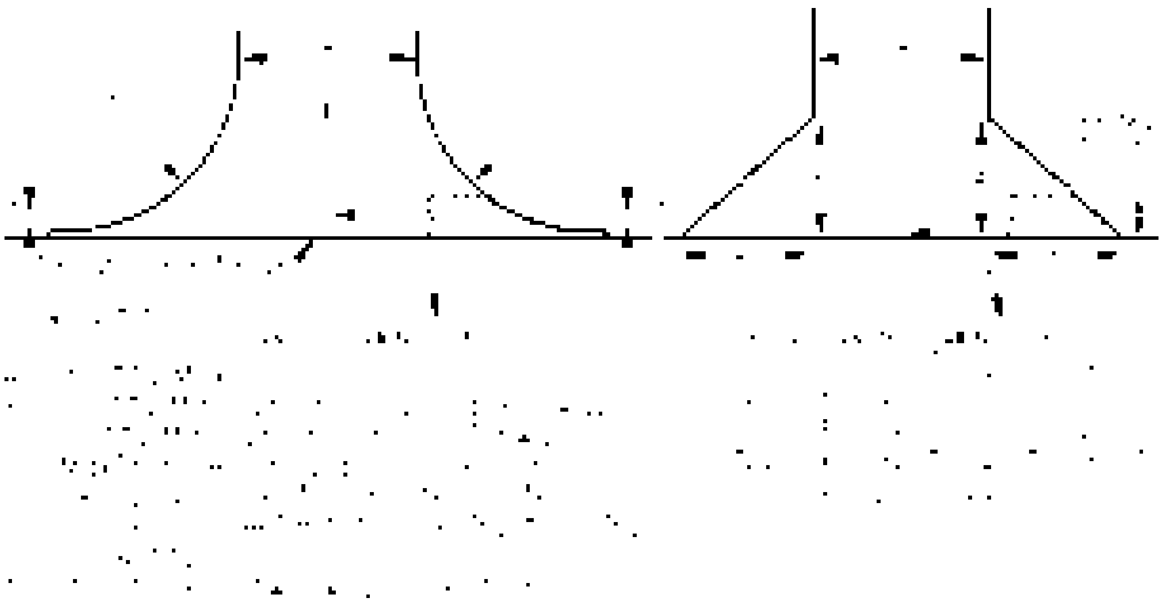


DRIVEWAY PROFILE

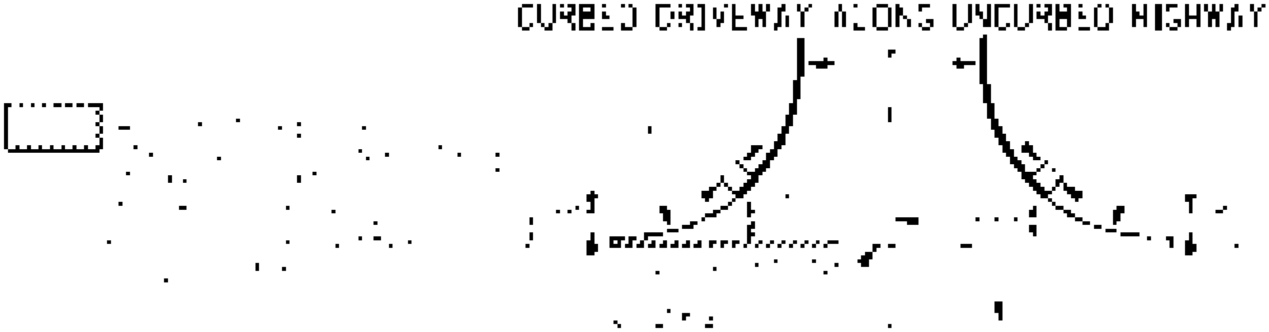
(Sidewalk without any Buffer)

STANDARD COMMERCIAL DRIVE DESIGNS	803-8
	REFERENCE SECTION 803.5

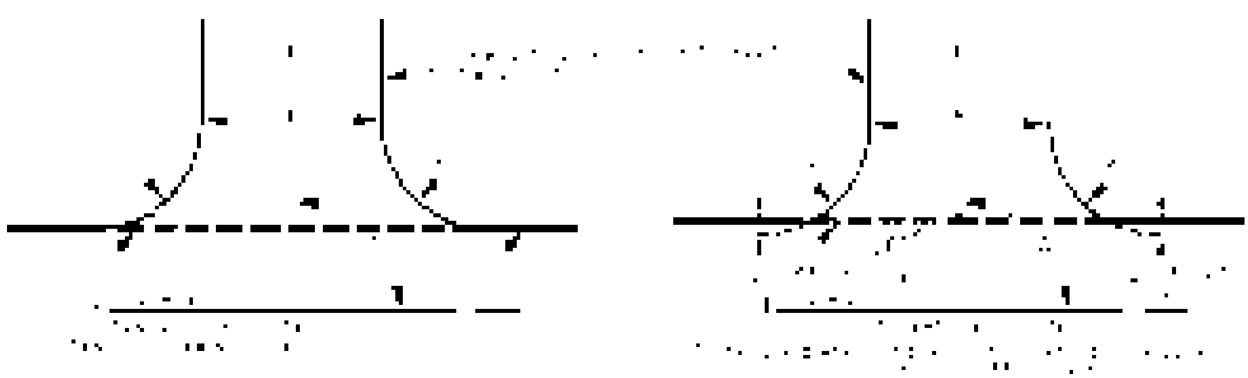
UNCURBED DRIVEWAY ALONG UNCURBED HIGHWAY



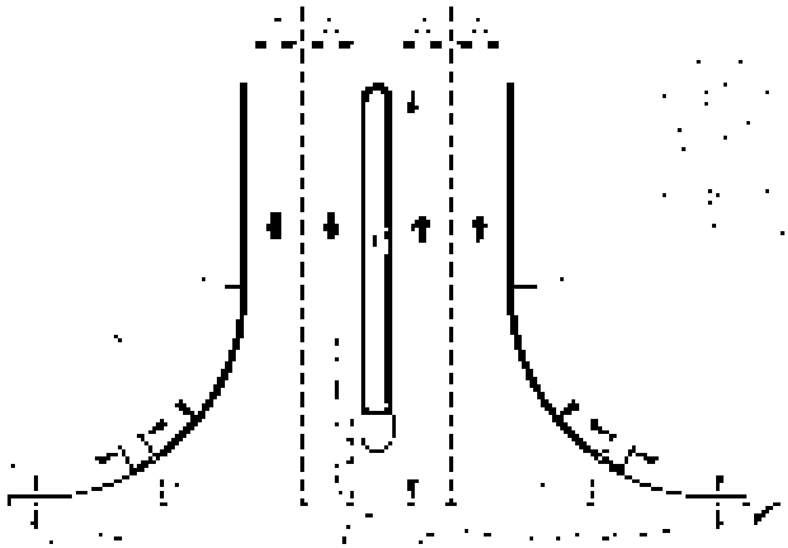
CURBED DRIVEWAY ALONG UNCURBED HIGHWAY



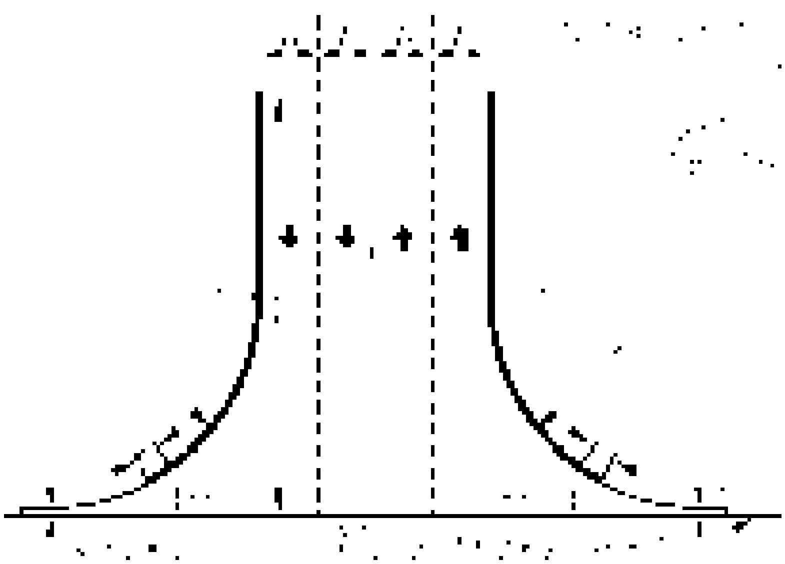
CURBED OR UNCURBED DRIVEWAYS ALONG CURBED HIGHWAY



SHOPPING CENTER & INDUSTRIAL DRIVE DESIGNS	803-9
	REFERENCE SECTION 803.6



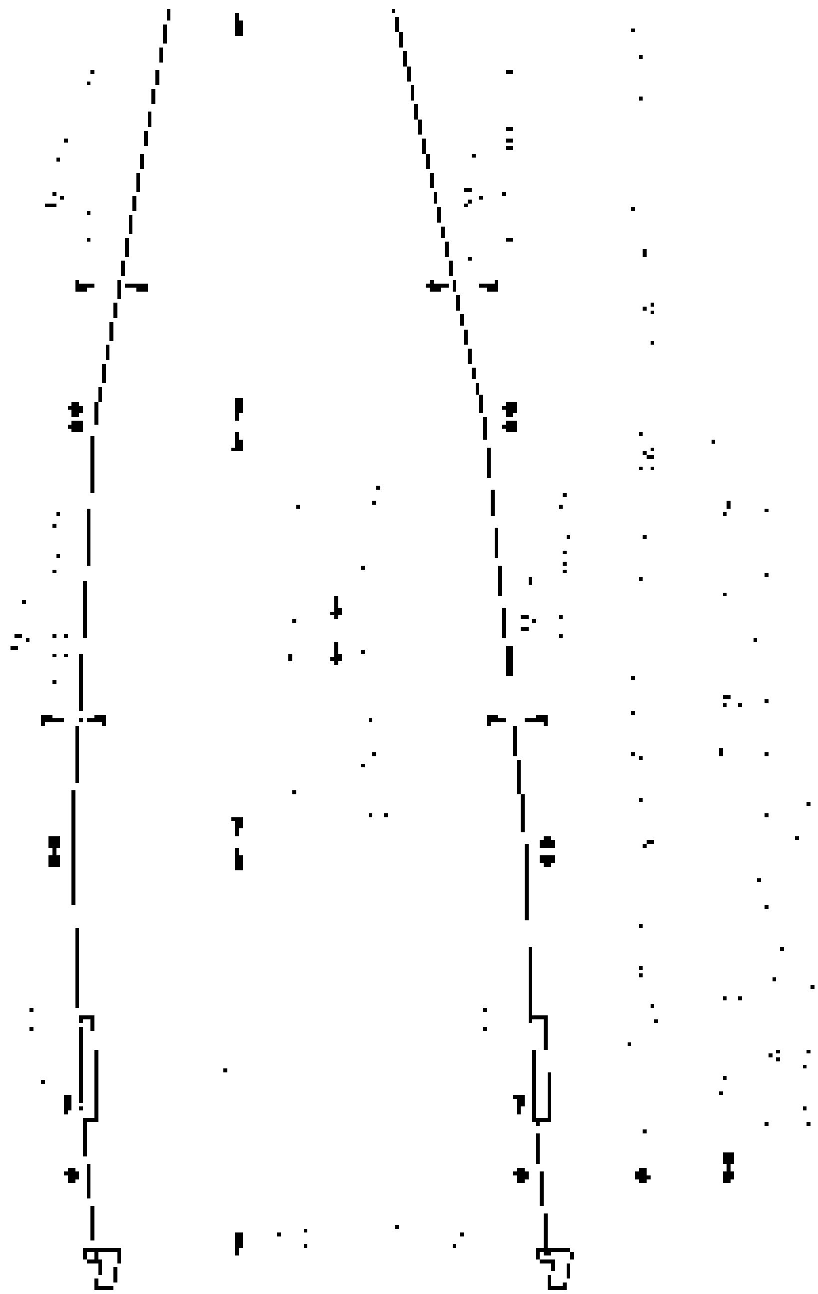
DIVIDED DRIVE



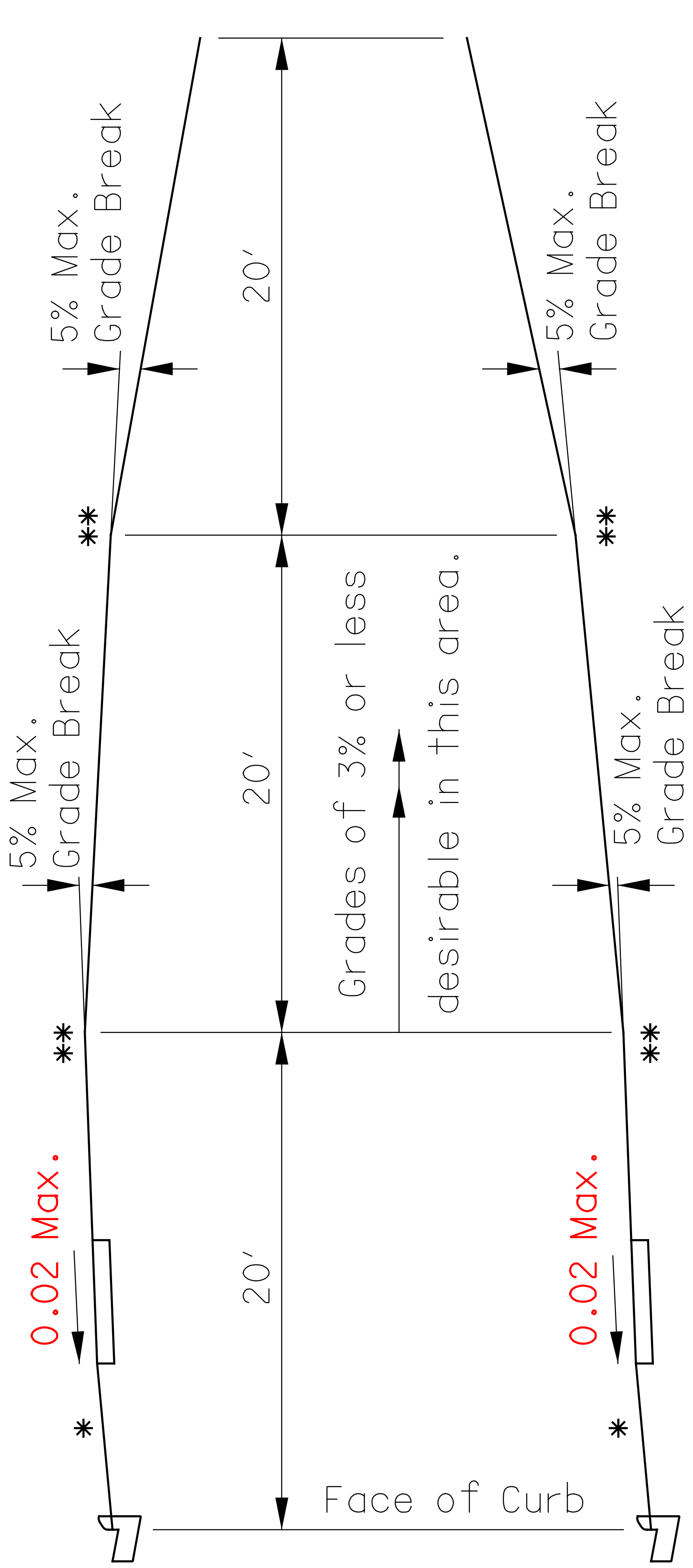
UNDIVIDED DRIVE

COMMERCIAL DRIVE PROFILE CRITERIA

804-1
REFERENCE SECTION 804.3



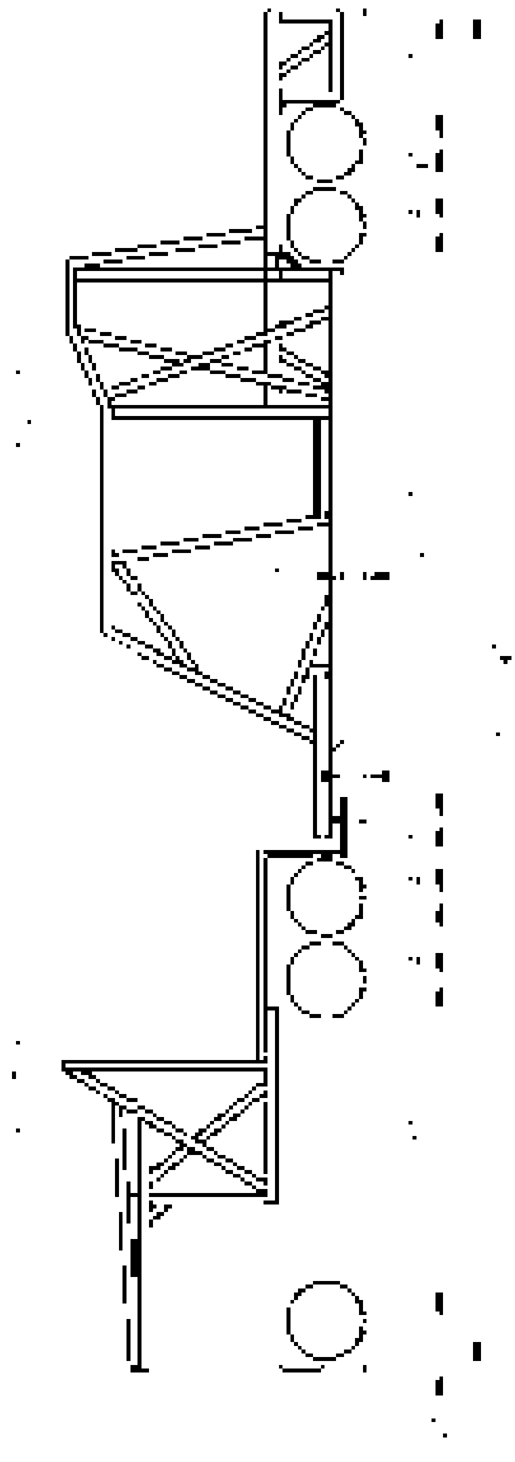
COMMERCIAL DRIVE PROFILE CRITERIA	804-1
	REFERENCE SECTION 804.3



* See Fig. 803-2 & 803-3 for Buffer and Sidewalk treatment.

** Although the use of grade breaks is allowable, a 10′ rounding is desirable at these locations.

COMMERCIAL DESIGN VEHICLE	804-2
	REFERENCE SECTION 804.3



900 Access Control, R/W Use Permits and Drive Design

Table of Contents

900 Access Control, R/W Use Permits and Drive Design

901 PURPOSE

901.1 General

These guidelines provide direction for landscaping within highway rights-of-way. The information provided in this guide is primarily safety-related and is intended for use by designers who already possess a good working knowledge of roadside safety design and landscape design. ODOT's Vision is to provide a safe and mobile transportation system. Landscape projects, therefore, shall be designed with the safety of the traveling public and maintenance crews as the top priority. The following guidelines follow the principles offered by AASHTO’s Roadside Design Guide.

901.2 Background

The basis for this section stems from the fact that trees are a major cause of injuries and fatalities on the nation's highways. While it is desired to increase the amount of aesthetics on the State highway system, and these guidelines try to encourage that end, it cannot be understated: trees are proven killers when placed by the roadside. Single vehicle crashes with trees account for 3,000 fatalities each year nationwide. Trees are not generally a highway element that engineers have control over, except in landscaping projects where the designer can make decisions to reduce the consequences of vehicles leaving the road.

901.3 Additional Information

This section is written for primarily the roadside safety aspect of landscaping. However, by necessity, this section contains other information for the landscape designer to consider in developing themes, schemes and layouts. But in no way is this information considered to be all inclusive.

902 GENERAL SAFETY

Trees are potential obstructions by virtue of their size and their location in relation to vehicular traffic. Generally, existing trees with an expected mature size of greater than 4 inches are considered fixed objects. Landscaping elements shall be selected and located to maintain adequate sight distances and clear zone setbacks. These elements shall not interfere with the function of the pavement, shoulders, longitudinal barriers, end treatments, drainage systems, traffic signs, signals, utilities and other highway structures and appurtenances.

903 PLAN REQUIREMENTS

903.1 Preliminary Field Review

All landscape projects should involve a preliminary field review prior to the scoping meeting with the consultant/designer and a district/county designee(s) knowledgeable in landscape design and roadside design/safety. At the preliminary field review, conceptual locations available for planting wildflowers, seedlings, trees, shrubs and other landscaping elements should be identified.

903.2 Scoping

Experience has shown that proper project scoping is invaluable in heading off later misunderstandings between landscaping proponents and highway engineers. Agreeing in advance of the project to require detailed plans, permissible landscape elements, final field reviews, and maintenance agreements are important to providing a beautiful, yet safe roadside landscaping.

903.3 Landscape Plan Details

Landscape plan should be concise and easily understood. Plans should be drawn to scale and developed on standard plan and profile sheets. Plans should indicate the following:

- design and legal speeds for the landscaped roadways
- type of adjacent land use (e.g. farmland, commercial, residential, etc.)
- topographic features such as slope limits and slope rates
- contour grading at interchanges is preferred
- locations of all utilities
- location and descriptions of existing landscaped areas
- location of all existing longitudinal barriers, end treatments, impact attenuators,
- curbs and sidewalks
- location and configuration of ditches and other drainage features
- plant lists (including botanical and common names)
- size and spacing of plants as well as area of occupancy at maturity

Although many landscape designers desire to use "conceptual" layouts, it is imperative for the highway engineer to have as much of the above information as possible in a standard format to make informed decisions on the safety merits of the plan. Omission of such information will only lead to delays, and possibly to denial of otherwise acceptable planting arrangements.

903.4 Permit Applications

Landscaping permit requests shall include landscape plans as described in ***Section 903.3*** and be directed to the District Deputy Director. A Maintenance & Repair permit application (M&R 505) can be obtained from the District Permit Office. The District should consult the ODOT County Manager before issuing the permit to ensure coordination of different projects scheduled in the same area.

903.5 Final Field Review

After the plans have been accepted and all permits have been approved, the consultant/designer and a district/county designee(s) knowledgeable in landscape design and roadside design/safety should conduct a final field review.

904 LANDSCAPE DESIGN CONSIDERATIONS

904.1 General

Landscape design can serve several important functions within the highway environment. In addition to making the roadway more aesthetically pleasing, landscaping can also be used to do the following:

- control erosion
- create a living snow fence
- minimize maintenance requirements and costs
- screen undesirable views
- preserve desirable views
- shield headlight glare
- preserve/enhance the natural environment
- reduce unwanted noise, and possibly to serve as a substitute for noise barrier at the request of a local community (see ***Figure 904-1*** Vegetative Screening in lieu of Noise Barrier)

Landscaping projects must be done as a part of a community sponsored comprehensive plan. The plan must be sponsored by the public agency that will also be responsible for maintenance of the landscape features. Landscaping at an interchange should incorporate the entire interchange rather than just individual ramps. Landscaping may be permitted along highway segments if it is sponsored and maintained by a public agency. The goal is to provide a community endorsed, consistent theme along the highway rather than isolated, independent projects. Landscaping that contains advertising or company logos will not be permitted. It is permissible for individual property owners abutting the highway to request a permit to clear, mow, or plant replacement trees along their frontage to improve the visibility from the highway.

It is recommended that the designer choose plants carefully. Highway plantings used in the roadside environment should be hardy for the Planting Zone, salt sprays, and air pollutants (see ***Section 906***).

Trees are not to encroach on the sight distances, have trunks greater than 4" mature diameter when planted in certain locations (see ***Section 905***), or have canopies that will encroach over the road.

Highway landscaping should result in designs that do not require extensive maintenance. In fact, at the end of the five year maintenance period described in ***Section 908.1***, landscaped areas should not require any more maintenance than the natural roadside. Therefore, plant materials noninvasive to the area should be used whenever practical.

904.2 Landscaping Elements & Fixed Objects

Landscaping elements may consist of natural as well as manmade features, e.g., groundcovers, flowers, trees, and pavers. Many of these features such as most groundcovers and pavers allow a vehicle to safely pass over them and, therefore, do not pose a significant risk to an errant motorist. However, other features may be considered fixed objects and are, therefore, potential safety hazards. In general, a fixed object is any object that cannot be driven over safely by an errant vehicle. This includes but is not limited to the following:

- individual trees with a trunk caliper (diameter) greater than 4 inches at maturity, trunk caliper is measured at 54" up from the ground,

- clusters of smaller caliper trees or shrubs with multiple trunks or groups of small trees planted close together (within 6 feet), where the sum of their calipers at maturity exceeds 4 inches,
- decorative walls,
- rock formations and other free standing objects or fixed objects with a diameter or height greater than 4 inches. Fixed objects shall not be installed within medians or along the roadside within the setback areas specified in ***Section 905***.

Aesthetic fixed objects shall never be placed within the clear zone. In addition to basic clear zone requirements the following shall apply to aesthetic fixed objects:

- Within interchanges shall only be constructed in areas designated for "Tall Plantings and Trees" as shown on **Figures 905-2** and **905-3** of L&D volume 1.
- Shall not affect interchange lighting.
- Shall not be constructed at the bottom of a non-traversable slope.
- Shall not be constructed within 50 feet of the edge of the traveled way on safety graded sections.
- The offset of a fixed object to protective barrier shall not be less than the working width of the barrier, see **Figure 603-2.**
- Guardrail and concrete barriers, while tested to be crashworthy are still fixed objects. Installing protective barriers with the sole purpose of protecting an aesthetic treatment is not preferred. Other options such as relocating the aesthetic feature or utilizing existing barriers should be exhausted before contemplating the installation of new barrier.

904.3 Bodies of Water

Bodies of water present unique safety concerns. The department recommends the use of longitudinal barriers to protect naturally occurring ponds located within the setback areas. Ponds/pools and other landscape water features shall not be built within highway rights-of-way. This does not preclude the construction of treatment ponds or water retention basins within the right-of-way when mandated in the environmental process.

904.4 Accessories

In community gateways and downtown business districts many municipalities seek to install street furniture, pavers, bollards, ornamental lighting, planters and other landscaping features to the design. Features within the lateral offset distances described in ***Figures 904-2 & 904-3*** are to be crashworthy, as specified in NCHRP Report 350 or MASH. Amenities located beyond the appropriate offset distances shown in this guideline may be allowed. Any feature placed within ODOT's Right-of -Way is allowed solely at ODOT's discretion. Landscaping plans that include decorative signs must conform to ***Section 210-3* of the Traffic Engineering Manual**.

904.5 Irrigation Systems

Many lavish plantings will not survive unless maintenance is provided. Some communities protect their investment by installing irrigation systems. Irrigation systems cannot be a hazard to the motorist. Systems cannot have hazardous stub heights (4” diameter max.), exposed pipes or meters in the specified offset distance. Nor should the spray be directed to the roadway, nor is ponding or sheet flow permitted on the traveled way. In all cases, maintenance and repair of irrigation systems will be the responsibility of the project sponsor.

905 PLACEMENT FOR SAFE ROADSIDE DESIGN

905.1 Roadside Grading

Since operational safety can be affected by the landscape, a continuous length of the highway must be visible to the driver (sight distance) and a lateral run out area (clear zone) must be traversable and free of physical obstructions.

Clear zones provide areas for drivers of errant vehicles to regain control after running off the road. Although minimum setbacks for large trees and other fixed objects are prescribed in the following sections, consideration should be given to providing additional clearance where practical. Setback distances are measured to the face of the fixed object from the traveled edge line of the adjacent roadway. For facilities with curb and gutter, setback distances are measured from the face of curb to the face of the object. Bike lane and parking lane widths may be included in the setback distance. For trees, this measurement shall be taken to the face of the trunk 2 feet above the ground line.

Large trees and shrubs may be planted within the setback limits specified in this section where the likelihood of an impact by an errant vehicle is negligible; for example, on cut slopes above a retaining wall or behind existing longitudinal barrier. See ***Section 307*** for details on the following types of grading, ***Section 600*** for clear zone criteria, and ***Section 201*** for details on required sight distances.

905.1.1 Safety Graded Sections

Trees and large shrubs shall not be planted within 50 feet of the edge of the traveled way on safety graded sections. Low maintenance flowers, ground covers and other plants 18 inches or less in height at maturity may be located within this setback area as long as adequate sight distance is provided. See ***Figure 307-1*** for Safety Grading.

Trees and other plants taller than 18 inches may be located beyond this setback distance with the following restrictions:

- These plants shall not be located within a ditch or on a backslope within 20 feet of the ditch flowline.

905.1.2 Clear Zone Graded Sections

Trees and large shrubs shall not be planted within 30 feet of the edge of the traveled way on clear zone graded sections. Low maintenance flowers, ground covers and other plants 18 inches or less in height at maturity may be located within this setback area as long as adequate sight distance is provided. See ***Figure 307-3*** for Clear Zone Grading.

Trees and other plants taller than 18 inches may be located beyond this setback distance with the following restrictions:

- These plants shall not be located on foreslopes
- These plants shall not be located within a ditch or on a backslope within 10 feet of the ditch flowline

905.1.3 Common Graded Sections

Plantings shall be located at least 4 feet behind the ditch line in cut sections and 2 feet outside the shoulder break in fill sections. See ***Figure 307-4*** for Common Grading.

905.1.4 Barrier Graded Sections

An ideal location for large trees and shrubs is behind existing longitudinal barriers, provided the landscape designer allows for a maintenance access. The lateral offset to these plants shall be 15 feet measured from the face of a w-beam guardrail to allow the barrier to deflect to its design deflection in an accident, but to also allow maintenance vehicles to navigate on the back side of the barrier. Other types of barriers have different deflection limits. Barriers shall not be installed solely to permit the use of large trees or other potentially hazardous landscaping elements along the roadside. See **Figure *307-4*** for Barrier Grading.

905.1.5 Gating End Terminals

Advances in the performance of guardrail end terminals and impact attenuators (crash cushions) have dramatically increased the safety of the traveling motorist. Many of these systems are designed to be "gating" (or "non-redirective") in certain types of impacts. Gating terminals function successfully by allowing approaching vehicles to pass through (or "gate") the very end of the end terminal. Impacting vehicles are only slightly impeded by the interaction with the terminal, and possibly still are traveling at a high speed. Thus, no fixed objects are allowed in a runout area that is defined by FHWA to be a minimum of 20 feet wide behind and perpendicular to the rail and 75 feet long beyond the terminal parallel to the rail. ***Figure 905-1*** shows the permitted landscaping offset needed to protect this runout area behind gating terminals.

If the landscaping designer does not know which treatment is used at the end of a guardrail run, for the purpose of the landscaping plan it will be considered to be gating. All associated runout areas will remain free of fixed objects.

905.2 Urban Design

The Roadside Grading section generally deals with high speed rural roadways. Municipalities may desire to landscape gateways into their communities, which is often a state highway or an interchange that leads to an arterial. The highway facilities in these gateways are often roadways with lower speeds than found on the rural state system. These roads may be lower speed, divided or not, or curbed or not. Refer to ***Section 600.2.2*** for discussion on Urban Lateral Offsets where Clear Zone cannot be achieved and ***Figures 600-3, 600-4, 904-2 & 904-3***. The following discussion gives highway engineers and landscape designer's additional guidelines for placement of large trees, small trees and foliage in urban areas. Other landscaping features, such as lighting, stones, boulders, bollards, or water ponds, etc. are to meet guidelines listed elsewhere.

Refer to ***Figure 904-2*** for treatment in curb sections. Curbing is considered mountable, a vertical 4-inch curb (or even 6 inches or more) is not going to stop a vehicle. Large trees are considered to be nonfrangible and have a final (mature) trunk diameter of 4 inches or greater. The sum of the individual trunk dimensions of multi-stemmed tree are considered as one object over a 6-foot vehicle width. Setbacks in curbed sections are from the front face of the curb unless bike lanes or full-time parking lanes are present. Since urban tree locations have considerably less offset than high speed facilities, vertical clearance becomes an issue. All trees, especially those planted close to a curb will have their canopies clipped by trucks in the lane adjacent to the trees. Plant trees to ensure their mature canopy will not infringe on this area.

905.3 Highway Design Elements

Certain highway features provide a special opportunity for communities to express themselves through landscaping. Interchanges and intersections are ideal locations, although they do require special attention by designers.

905.3.1 Interchanges

Interchanges provide an opportunity for establishing and/or preserving attractive landscapes along our highways. Because an interchange often serves as a major focal point, both from the highway and from the cross road, the major components should be coordinated to achieve an overall design that is aesthetically pleasing. Major components of an interchange include: structural design, texture and detailing, railings, lighting, contour grading and plant material.

Generally, a minimum 50-foot setback (from the edge of traveled way) within a loop ramp is considered an appropriate sight distance setback for trees and shrubs with mature heights above 18 inches. ***Figures 905-2 & 905-3*** provide details for landscape plantings at cloverleaf and diamond interchanges. For interchanges, all plantings shall provide ramp and collector-distributor road sight distances equal to or greater than those required by the design speed criteria in ***Section 201***.

905.3.2 Intersections

A driver attempting to enter a through road must be able to see traffic at a distance along the intersecting road in order to safely enter the intersection. The required intersection sight distance varies with the speed of the traffic on the main highway. ***Section 201.3*** provides standards for various intersection sight distance conditions. The triangular setback areas shown in ***Figure 905-4*** are based on these principles. No plantings above 18 inches shall be permitted within these setback areas. This figure shows a tangent condition; a graphical solution is required when the through road is curved.

In general, an offset of 50 feet on the inside of a curve with a degree of curvature of 2 degrees or greater should be provided to ensure adequate horizontal sight distances.

905.3.3 Roundabouts

Landscape elements are vital to the proper operation of a roundabout and needs to be in place when the roundabout is opened to traffic. The purposes of landscape elements in the roundabout are to:

- Make the central island conspicuous to drivers as they approach the roundabout
- Clearly indicate to drivers that they cannot pass straight through the intersection. Restrict the ability to view traffic from across the roundabout through mounding of the earth and plantings. This will lead to slower entering speeds, which increases safety.
- Require motorist’s to focus toward on-coming traffic from the left

- Help break headlight glare
- Discourage pedestrian traffic through the central island
- Help blind and visually impaired pedestrians locate sidewalks and crosswalks
- Improve and complement the aesthetics of the area

When designing landscaping for a roundabout it is important to:

- Consider maintenance requirements early in the program stages of development
- Develop a formal municipal agreement describing the landscaping and maintenance requirements for roundabouts elements early in the scoping process and prior to design of the facility.
- Maintain adequate sight distances
- Avoid obscuring the view to signs
- Minimize fixed objects such as trees, poles, or guardrail
- Apply the guidance below relative to approach speeds and the permissible use of fixed objects such as trees, poles, non-hazard walls, non-hazard rocks/boulders, or guardrail

Clear zone and lateral clearance requirements are provided in ***Section 601***.

Typically a portion of the splitter island is situated within the critical sight triangles, the landscaping in these areas may be constructed with low-growth plants or grass. Grass or low shrubs are also desirable due to their ability to blend well with nearby streetscapes and the fact that they require only limited maintenance. Splitter islands should generally not contain trees, planter boxes, or light poles. Hardscape treatments like a simple patterned concrete or paver surface may be used on splitter islands in lieu of landscaping.

Landscape the central island by mounding the earth and providing plantings. Refer to ***Figure 905-5*** for the general layout of the central island. The truck apron is not included in the clear zone distance. The clear zone for the central island is considered to begin at the inside curb adjacent to the central island landscaping. The combination of the earth mound and plantings in the central island shall provide a visual blocking such that drivers will not be able to see through the roundabout central island. The central island area is considered a low speed environment; however errant vehicles occasionally end up in the central island or crossing the central island.

The approach highway speed is an indicator of the probability of an errant vehicle entering the central island. Therefore, when the posted speed on any approaching leg to the roundabout is greater than 35 mph, hazards and fixed objects such as concrete, stone, or wood walls and trees having a mature diameter greater than 4 inches are prohibited within the central island.

Where the approaching leg to a roundabout has a posted speed of 35 mph or less there may be objects that appear to be hazardous such as walls or rocks, but they are to be constructed with materials and in a manner that is not hazardous to errant vehicles. It is important to minimize the consequences of an errant vehicle that may impact a wall or rocks/boulders. The inner portion of the central island is typically most vulnerable to drivers/vehicles that for some reason leave the roadway and enter the central island at a high impact angle. If in the event that a driver is driving too fast to negotiate a curved approach to a roundabout , or otherwise distracted and/or is not aware of the upcoming roundabout the impact angle entering the central island typically will be much greater than 25 degrees and outside the realm of roadside design. The consequence of hitting a fixed object at an angle greater than 25 degrees is severe.

Minimize the consequence of hitting a wall or boulders by following these guidelines:

1. *Do not allow* any walls in the central island with cast in-place or reinforced concrete or natural
2. boulders.

3. Construct any walls with light-weight, Styrofoam type, artificial bricks/blocks typically used in landscaping and boulders with chicken wire and stucco. No mortar or reinforcing between the
4. bricks/blocks. Minimize the wall thickness while maintaining stability.
5. If light-weight walls are desired for aesthetic reasons then construct at a height 20-inch or lower. This will tend to keep flying debris at a lower level as not to penetrate a windshield, or impact other vehicles.
6. Do not allow fill material in back of the light-weight brick/block wall for approximately 2 feet. Then at ground level begin to slope the earth up and away from the non-hazardous wall at a 6:1 slope or flatter.

Design the slope of the central island with a minimum grade of 25:1 and a maximum of 6:1 sloping upward toward the center of the circle. The earth surface in the central island area forms an earth mound that is a minimum of 3.5-feet to a maximum of 6-feet in height, measured from the circulating roadway surface at the curb face. As an absolute minimum, keep the outside 6 feet of the central island free from landscape features to provide a minimum level of roadside safety, snow storage, and unobstructed sight distance. In some situations this central island area may need to maintain a low profile beyond 6-feet to allow over sized vehicle loads to pass over the central island without the axles passing over the central island.

Avoid items in the central island that may be considered an attractive nuisance that may encourage passersby to go to the central island for pictures, or other objects that might distract drivers from the driving task. When reasonable, consider a frost proof water supply (small hand hydrant, not fire hydrant) and electrical supply to the central island. The water supply should be considered for long term use not just to establish plant material.

905.4 Additional Planting Constraints

Accident Locations - Offset distances greater than the minimum setbacks should be considered at locations with a history of run-off-the-road crashes.

Agriculture - Plants shall not obstruct, shade, or cause harm to crops planted in adjacent farm fields. When wind breaks and living snow fences are proposed adjacent to agricultural use properties, permission to plant should be obtained from the property owner.

Billboards - Plants shall not obstruct the view of billboards. However, naturalized trees blocking billboards should be cut only with permission of the district. This work shall be done by permit using a certified arborist.

Businesses - Trees, shrubs and wildflowers should be planted to blend in with the natural environment.

Canopy Obstruction - Trees and shrubs shall be offset far enough from the edge of the traveled way to prevent damage to vehicle windshields or interference with overhead utilities and signals.

Ditches - No planting other than seeding shall occur within ditches.

Irrigation Systems - Irrigation systems should be designed to minimize overspray onto the traveled way. The systems should be located so that the potential for damage to and from vehicles is prevented.

Scenic Views - Materials should be selected and placed to preserve desirable scenic views along the roadside.

Sight Distance - Proposed plants shall not restrict the horizontal and vertical sight distance of the roadway. Although the minimum setbacks provided in these guidelines were selected to ensure adequate sight distances, this should be field-verified and the setbacks shall be increased where necessary. In cases where an existing facility does not already provide adequate sight distance because of geometric restrictions, no further reduction of the sight distance shall be allowed.

Slopes - Evergreen and deciduous seedlings are the preferred vegetation; mature trees may be used when required for mitigation. Wildflower and native grasses (Construction and Material Specification (CMS) 870, Seed Mixtures Table) may be used with District Deputy Director approval.

Snow Fence - Only evergreens may be planted as living snow fence. Multiple rows shall be staggered. A general rule of thumb is that snow will be deposited on the leeward side of a snow fence over a distance approximately equal to the height of the snow fence. Care should be taken to ensure that the snow fence is planted far enough from the edge of the pavement to prevent snow from being deposited onto the roadway. (Also see Windbreak.)

Windbreak - Use the Ohio Windbreak Guide, published by the Ohio Department of Natural Resources as a source for design and species selection. An excellent source of information is found on-line at http://www.dnr.state.oh.us/portals/18/landowner/pdf/windbreaks_guide.pdf.

906 PLANT MATERIAL

Several lists of acceptable plants are available through ODOT Central Office, or certain ODOT District Offices.

906.1 Native or Non-Invasive Plants

All plant material shall be disease and pest free. A copy of the nursery inspection should be made available upon request.

906.1.1 Wildflowers

Wildflower sites should be composed of Ohio native perennial forbs and grasses. Other mixtures should be approved by the District Deputy Director, or designated employee. Wildflower areas should be designated as No Mow. See CMS Item 659.09 for available species acceptable for planting on the Right-of-Way.

906.1.2 Seedlings

Both Deciduous and Evergreen Seedlings should be salt tolerant and planted area should be signed as "No Mow."

Evergreen Seedlings may be used to create living snow fences and screenings. Locations include but are not limited to:

- slopes
- erosion prone areas
- interchanges (see ***Figures 905-2 & 905-3***)

906.1.3 Trees and Shrubs

Site design should use plant materials in a way that is low maintenance, has multi-seasonal interest and looks natural. Approval of locations should be based on safety, aesthetics and maintenance concerns. Typically trees and shrubs may be planted in the spring and fall. However, for optimum growth, trees shall be planted during the months recommended for the individual species.

906.1.4 Species

An acceptable list of tree and shrub species is available in the Ohio section of The Roadside Use of Native Plants, FHWA ep-99-014 or the Ohio State University Extension Office's The Native Plants of Ohio_ (Bulletin 865, 1998), and from the Office of Material's Management. It is preferable that noninvasive species be used. Hybrids and cultivars may be substituted only with permission from the District Deputy Director, or designated employee, when native species are not available.

906.2 Zones

All trees shall be suitable for growth in Ohio Zone 5a or lower (USDA Hardiness Zones). Trees should be from Ohio growers whenever practical.

906.3 Emerald Ash Borer Insect

Landscape designers should be aware of the infestation of ash trees throughout Ohio and the efforts of Ohio Department of Agriculture (ODA) to combat this insect, which kill ash trees within three to five years from infestation.

It is recommended to refrain from planting ash trees for the next several years. If a landscaping project is utilizing exiting ash trees in the design, then trees should be monitored for Emerald Ash Borer signs, which can be found at the ODA website at www.ohioagriculture.gov/eab. (Some of the signs are "D" shaped exit holes, "S" shaped tunnels beneath the bark, dieback at the tops of the trees, sprouting around the trunk, woodpecker damage, or bark splits.) For more information about the pest, its current status, or ways to assist in early detection, calls the Emerald Ash Borer hotline at 1-888-OHIO-EAB.

907 PLANTING

Planting and bracing details are shown on Roadway Standard Construction Drawing LA-1.2.

Planting trees and shrubs too deeply is a persistent problem. To address this problem, the Ohio Nursery and Landscape Association and the ODNR Division of Forestry developed a set of tree planting specifications. This effort, called "Sample Tree Planting Specifications" is included as at the end of this Guideline.

908 MAINTENANCE

908.1 General

Unless otherwise specified, all maintenance of all plants shall begin upon installation and be arranged by the project sponsor. Plants shall be maintained by the permit holder for at least five years. The Department should inspect the landscape during this time and require maintenance as needed.

Refer to the CMS 651 thru 673 for detailed roadside installation and maintenance requirements. See M&R 632 for mowing specifications.

Maintenance shall include but not be limited to:

- watering, pruning, mowing, and replacement
- weeding, fertilization, mulching
- removal
- litter pick up
- insect control (by a licensed applicator, when required)
- herbicides (by a licensed applicator)

908.2 Watering, Pruning, Mowing, and Replacement

Watering - watering of the new plant material is essential for their survivability, and is the responsibility of the project sponsor.

Pruning - All trees and shrubs shall be maintained and only pruned as necessary to retain their natural shape or remove deadwood. For example, water sprouts (suckers) shall be removed from the base of each species as needed.

Mowing - Trees should be spaced sufficiently far apart and shrubs should be grouped and mulched in beds shaped to avoid excessive mower maneuvering and the need for hand trimming.

Replacement - All dead, dying or diseased plants shall be removed and disposed of Construction & Material Specification 105.13. Replacement shall be left up to the project sponsor.

908.3 Planting Stakes

Trees planted with support stakes and guy wires shall have all such appurtenances removed no less than 12 months and no more than 18 months after installation.

908.4 Winter Hazards

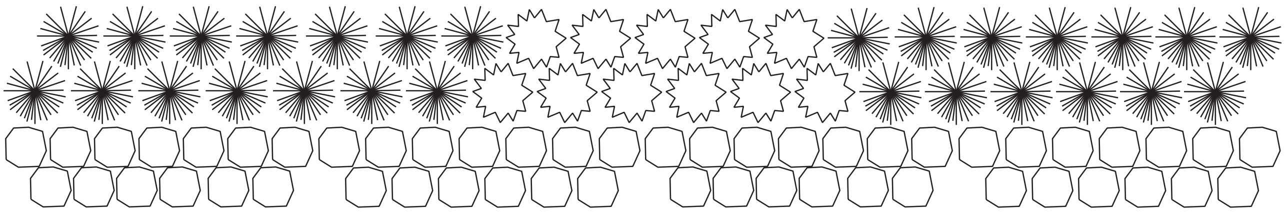
Landscaping shall not reduce safety for the traveling public or maintenance crews. Trees and shrubs should be placed in locations and trimmed to a size that does not hinder snow and ice removal. Removal or thinning of trees that shade the pavement creating icy spots should be considered. Some sections of the roadside should be kept open to allow sunlight to aid new tree growth.

908.5 Maintenance of "NO MOW" Areas

Naturalized (No Mow) areas can have a "neat" appearance without the removal of trees or shrubs. These areas within ODOT Right-of-Ways are frequently maintained by municipalities. If a community desires to maintain ODOT's Right-of-Way, an M&R 505 permit is required. Districts offices should also receive a maintenance plan from the community. If maintenance of Right-of-Way areas is done without obtaining the permit, communities can be held liable and be made to perform restitution.

LIST OF FIGURES

VEGETATIVE SCREENING	904-1
	REFERENCE SECTIONS 904.1



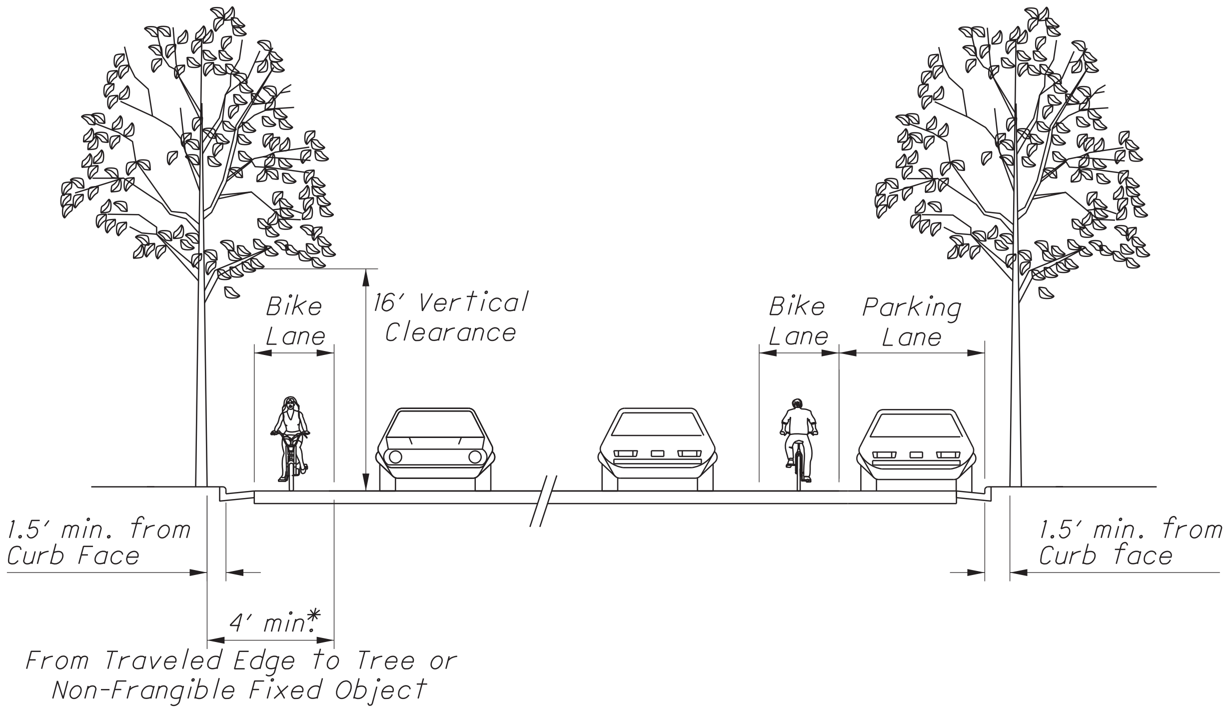
VEGETATIVE SCREENING IN LIEU OF NOISE BARRIER

Vegetation in lieu of a noise of a noise barrier is intended to provide psychological relief and is not inteded as a noise abatement measure. The provided drawing is an example. Alternative planting designs may be submitted for approval from the ODOT noise coordinator. All planting must provide 100" opacity year round to height of 6' within 3 years of installation.

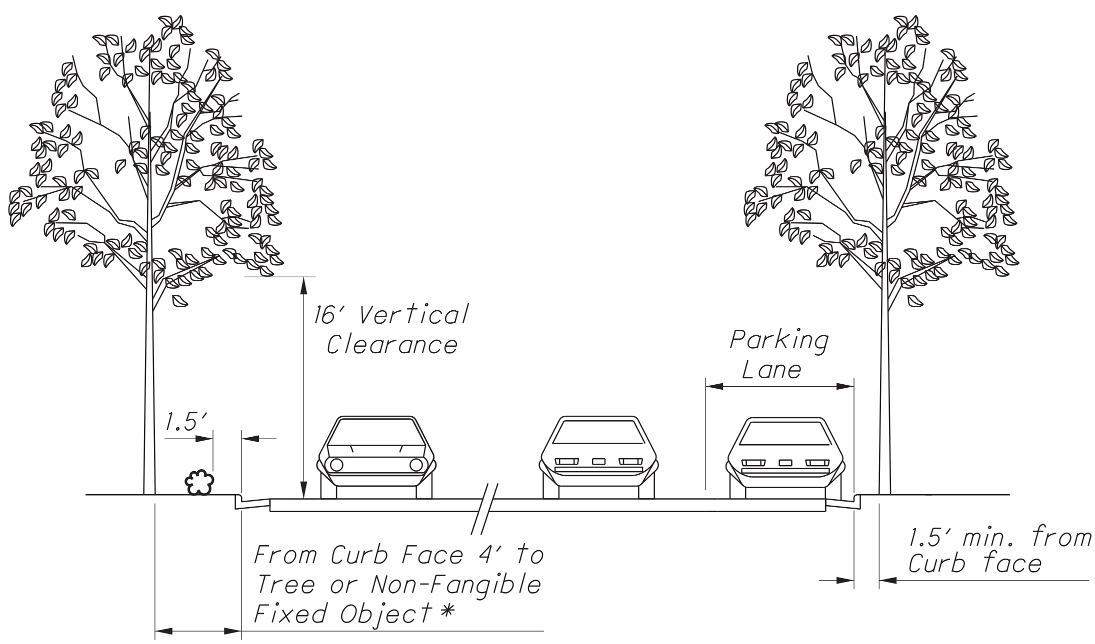
Place evergreen trees in an offset pattern with rows 8' apart and 8' on center. Plant trees in single species masses of a at least 15 trees. Plant minimum 5' tall evergreen trees from the following list: Chamaecyparis Thyoides - Atlantic White Cedar, Juniperus Virginiana - Eastern Red Cedar, Picea Abies - Norway Spruce, Picea Pungens - Colorado Spruce, Pinus Nigra - Austrian Pine.

Place shrubs in staggered alternating rows with plants 3' on center. Plant shrubs in single species masses of a minimum 25 plants. Alternate evergreen and deciduous shrub masses. Plant minimum 3' tall shrubs from the following list: Viburnum Prunifolium - Blackhaw Viburnum, Aronia Melanocarpa - Black Chokeberry, Ceanothus Americanus - New Jersey Tea, Juniperus Communis - Common Juniper (Cultivars - "Compressa", "Depressa", "Hills Vaseyi" and others with a similar habit).

URBAN LANDSCAPING TYPICAL CURBED SECTION 45 MPH OR LESS	904-2
	REFERENCE SECTIONS 905.2



CURBED SECTION
WITH BIKE LANES OR ON STREET PARKING

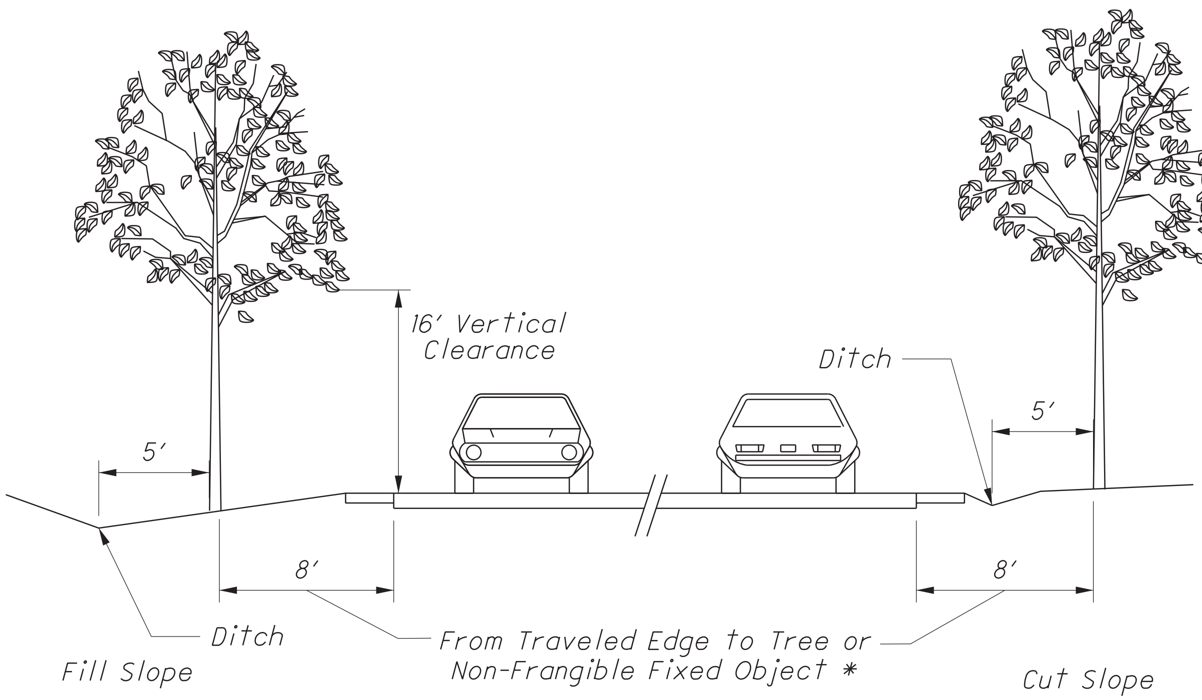


CURBED SECTION

* 6' min. in High Risk Areas Such as Outside Curves

July 2014

URBAN LANDSCAPE TYPICAL UNCURBED SECTION 45 MPH OR LESS	904-3
	REFERENCE SECTIONS 905.2

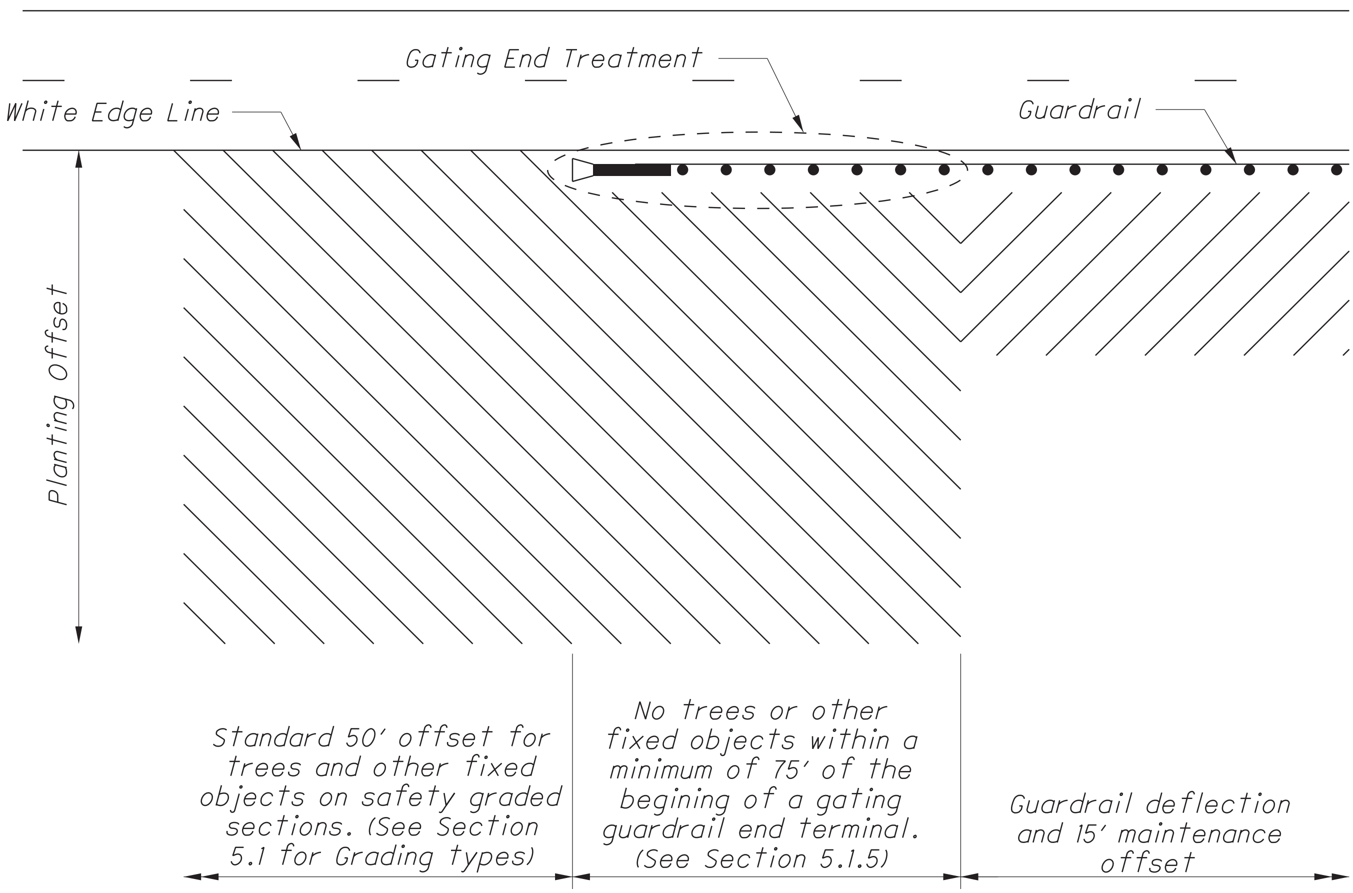


* 12′ min. in high risk areas such as outside of curves

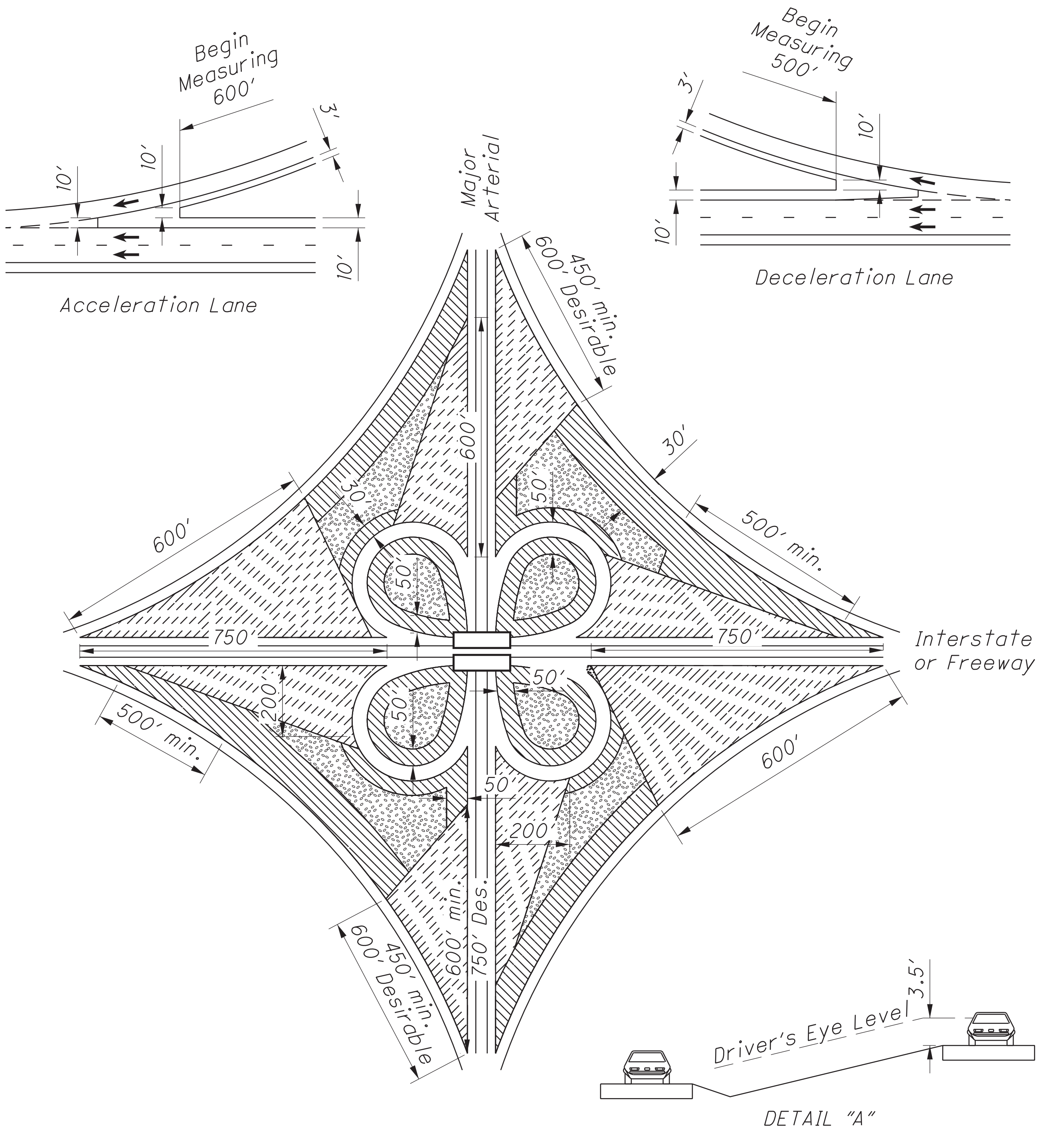
NOTE:
When the widths of the shoulders and ditches do not conform with these typical sections, the 2′ min. distance behind the ditch and 2′ min. distance outside the shoulder break shall govern.

GATING GUARDRAIL END TERMINALS OFFSETS

905-1
REFERENCE SECTIONS
905.1.5



GUIDE FOR LANDSCAPE PLANTING AT CLOVERLEAF INTERCHANGES FIGURE 3	905-2
	REFERENCE SECTIONS 905.3.1, 906.1.2

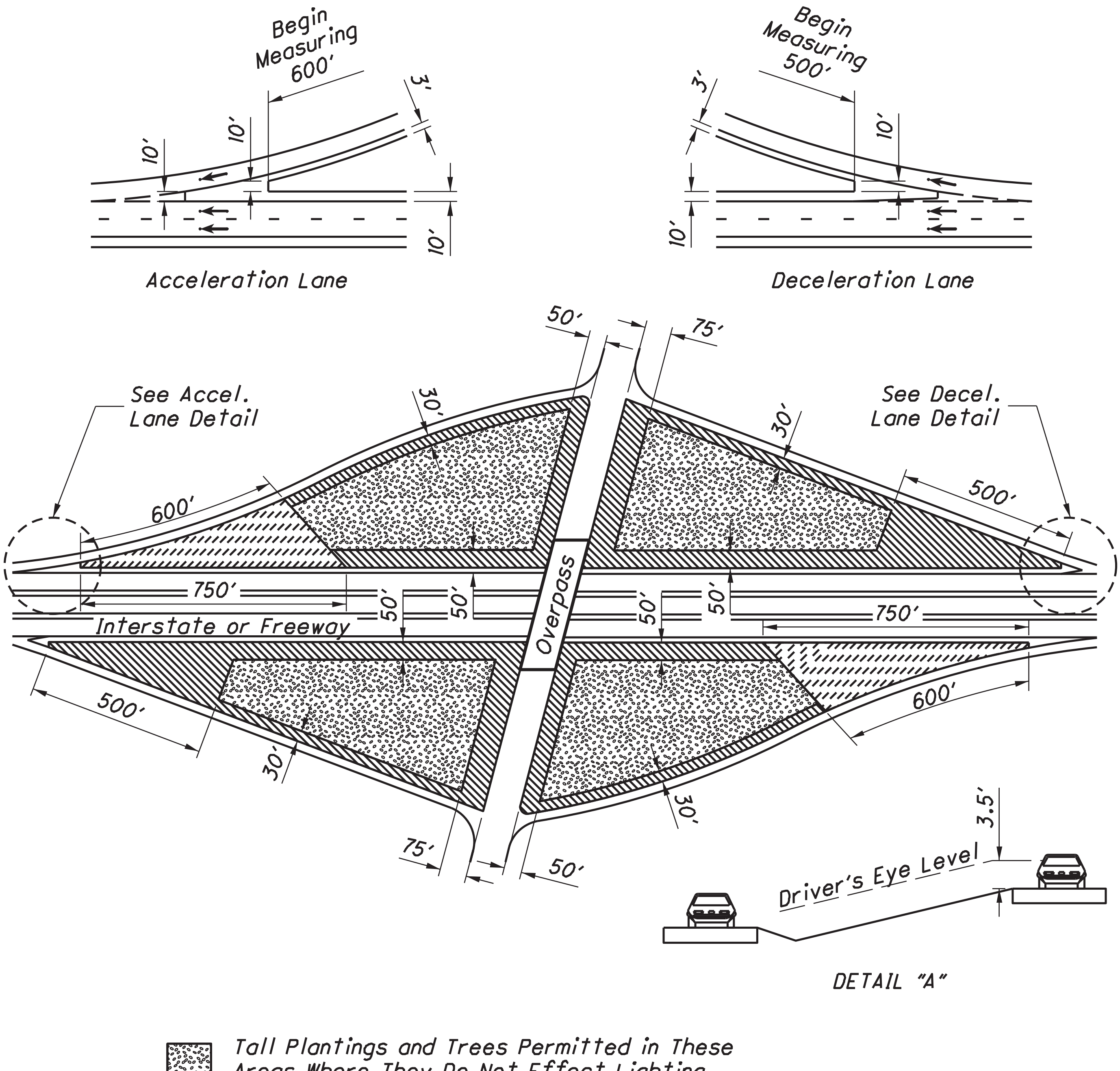


Tall Plantings and Trees Permitted in These Areas Where They Do Not Effect Lighting

Low Plantings Not to Obstruct Driver's View Permitted in These Areas, Except on Shoulders and Ditches

Low Plantings Not to Obstruct Driver's View of the Pavement on Shoulders and Ditches (See Detail A)

GUIDE FOR LANDSCAPE PLANTING AT DIAMOND INTERCHANGES FIGURE 4	905-3
	REFERENCE SECTIONS 905.3.1, 906.1.2



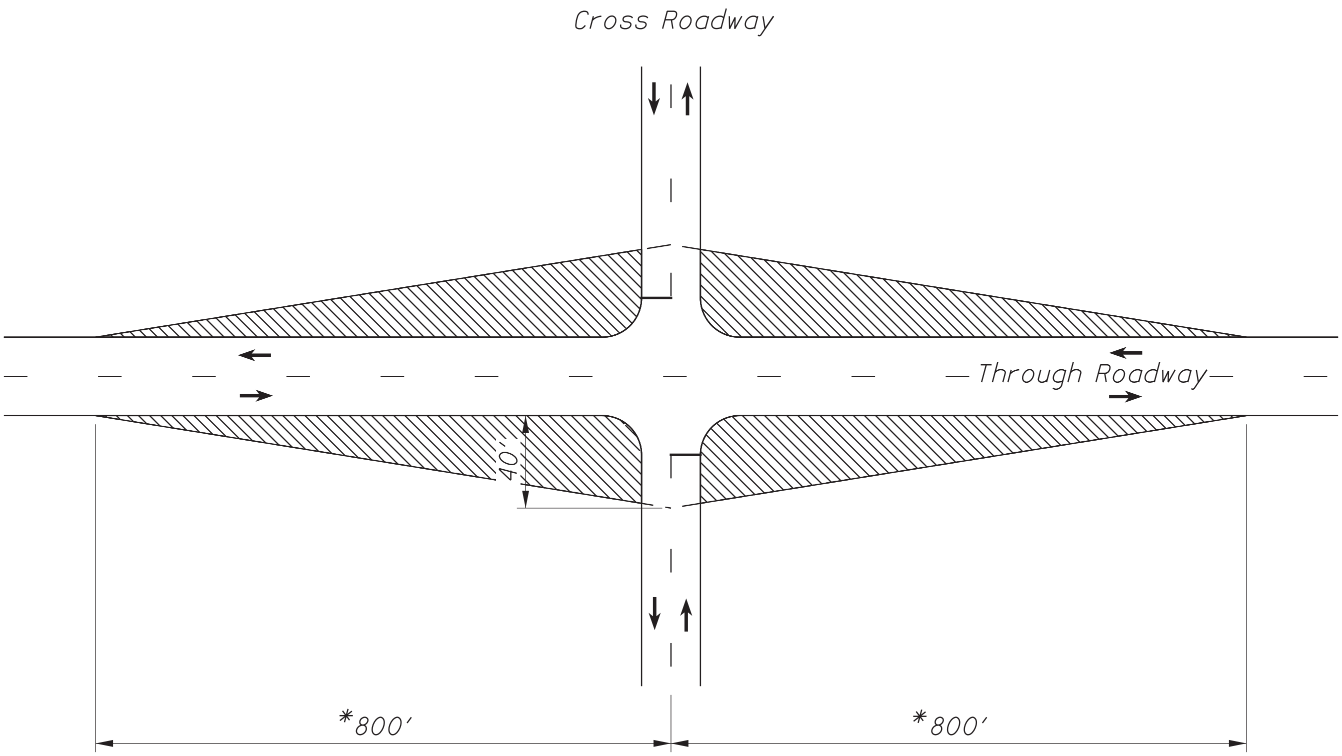
Tall Plantings and Trees Permitted in These Areas Where They Do Not Effect Lighting

Low Plantings Not to Obstruct Driver's View Permitted in These Areas, Except on Shoulders and Ditches

Low Plantings Not to Obstruct Driver's View of the Pavement on Shoulders and Ditches (See Detail A)

LANDSCAPING SETBACKS AT INTERSECTIONS

905-4
REFERENCE SECTIONS
905.3.2



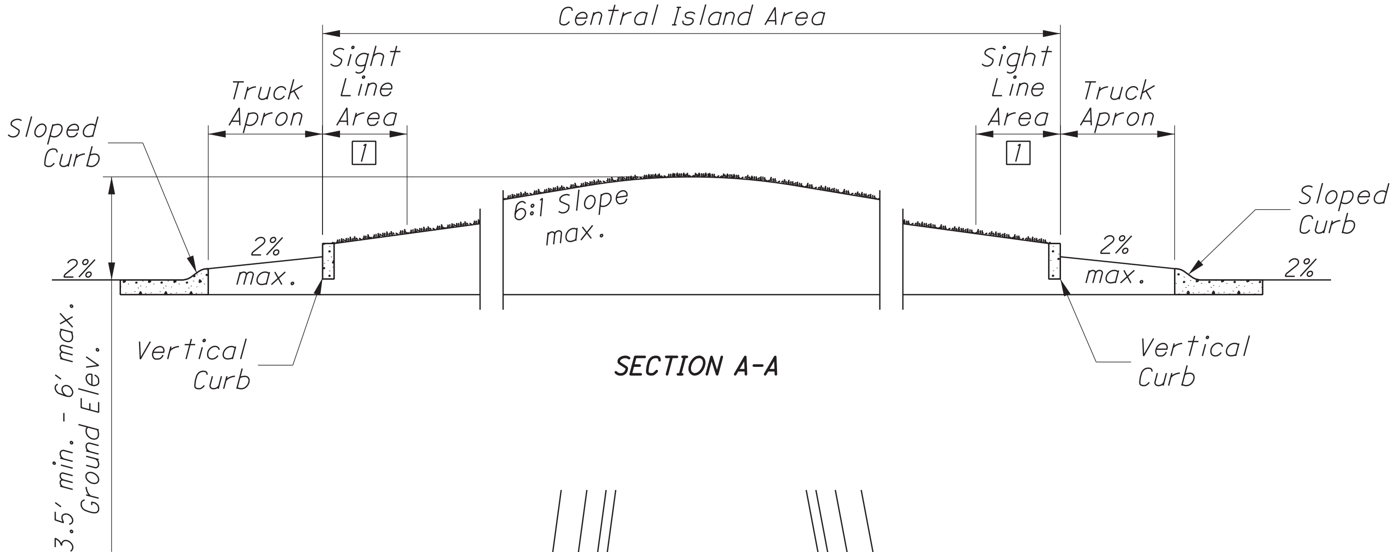
* These distances apply where speeds do not exceed 55 MPH

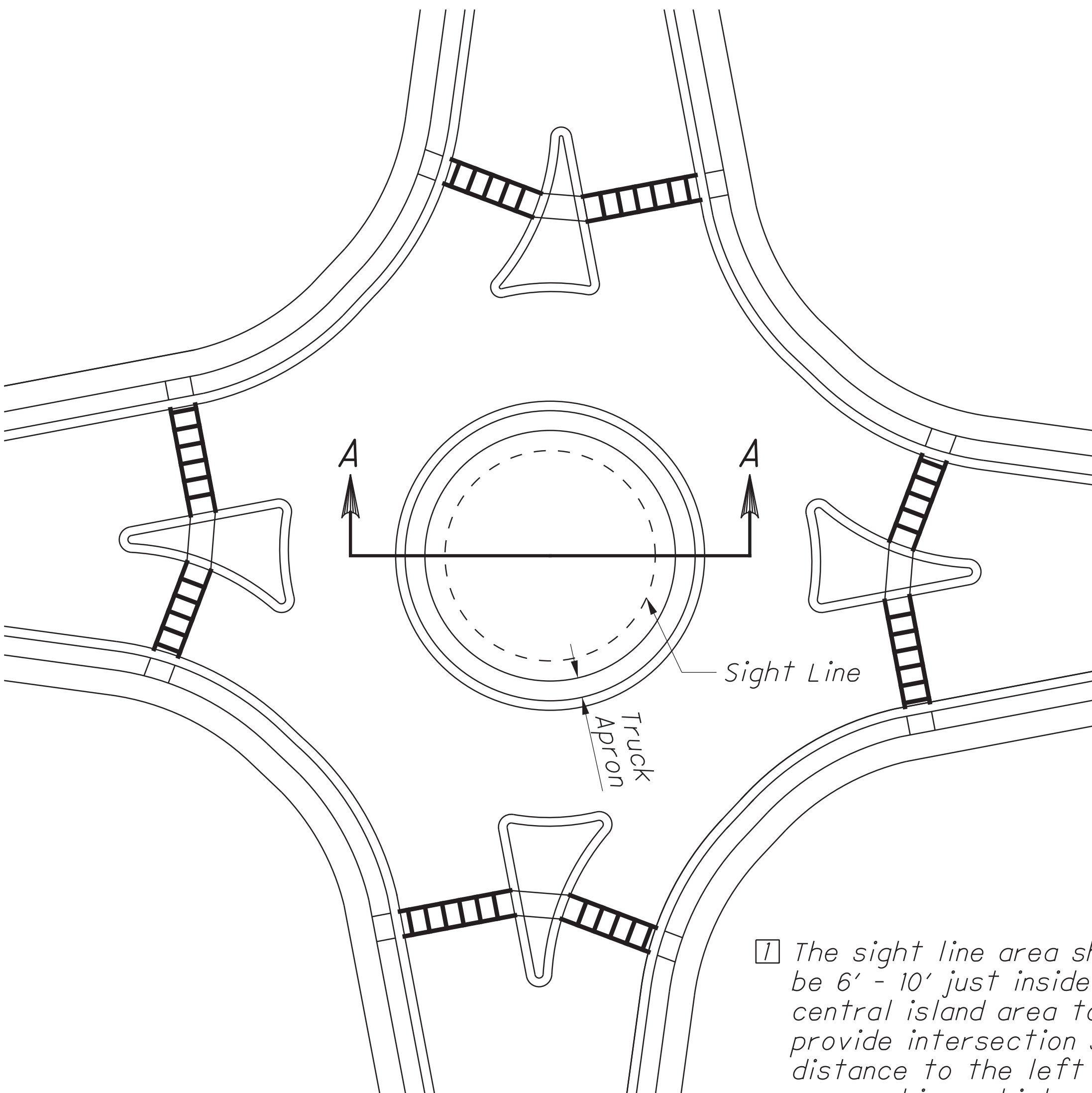
Low plantings not to obstruct driver's view permitted in these areas, except on shoulders and ditches

ROUNDABOUT CENTRAL ISLAND GRADING & LANDSCAPING

905-5

REFERENCE SECTIONS 905.5





1 The sight line area should be 6′ - 10′ just inside the central island area to provide intersection sight distance to the left for approaching vehicles.

SAMPLE TREE PLANTING SPECIFICATIONS

Endorsed by
Ohio Nursery and Landscape Association and ODNR Division of Forestry

Purpose: To increase transplanting success by providing municipalities with the most current and acceptable tree planting procedures. This information, prepared in specification format, will enable communities to convey specific requirements to contractors, developers, and/or volunteers. It contains the fundamental elements necessary to ensure transplanting success, and is intended to be a template that can be expanded to address other project issues.

Endorsement: This information is approved and endorsed by the Ohio Nursery and Landscape Association, and the Ohio Department of Natural Resources Division of Forestry.

Assumptions: All plant material complies with American Standard for Nursery Stock ANZI Z60.1. All plant material has been selected based on site conditions and constraints.

Planting Balled and Burlapped Trees:

-If not readily apparent, locate root flare by removing twine, burlap, and excess soil.
-Dig tree hole at least two times wider than the tree ball, with sides sloped to an unexcavated or firm base. Dig hole to a depth so the located root flare, at the first order lateral root, will be at finished grade.
-Lifting only from the bottom of the root ball, position tree on firm pad so that it is straight and top of root flare is level with the surrounding soil.
-Remove all twine from the root ball. If present, remove and discard at least the top one half of the wire basket. Burlap shall be removed from the top to a point halfway down the root ball and discarded.
-With clean, sharp pruning tools, prune off any secondary/adventitious, girdling, and potential girdling roots.
-Backfill planting hole with existing unamended soil, and thoroughly water.
-Mulch the entire planting surface with composted bark applied no less than two inches (2") deep and no more than three inches (3") deep, leaving three inches (3”) adjacent to the tree trunk free of mulch.

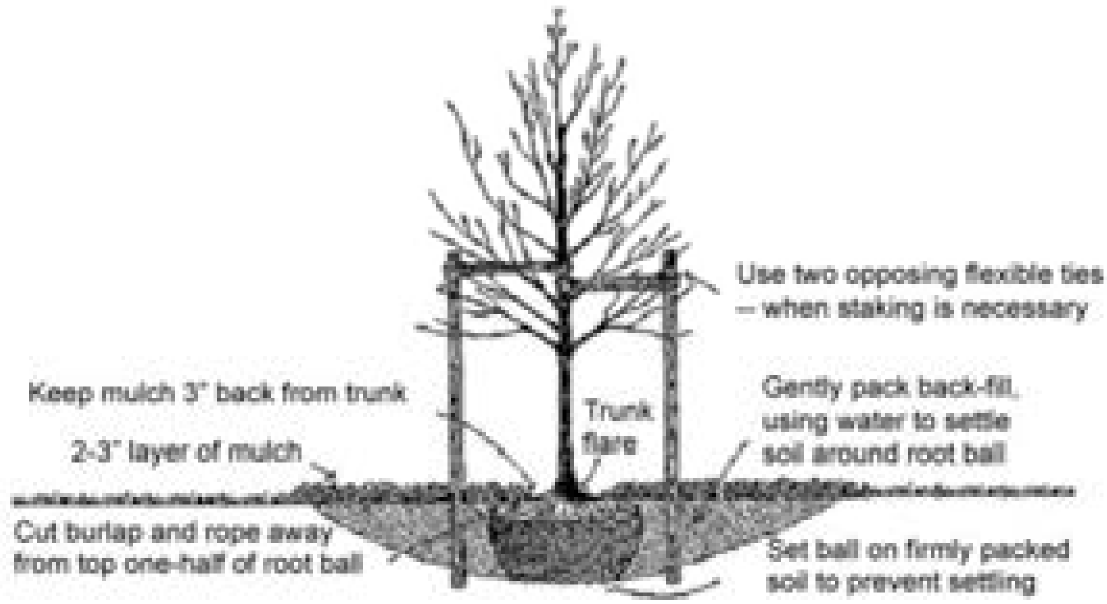
Planting Containerized or Grow Bag Trees:

-If not readily apparent, locate root flare by removing excess soil.
-Dig tree hole at least two times wider than the tree ball with sloping sides. Dig hole to a depth so the located root flare, at the first order lateral root, will be at finished grade.
-Create a firm soil mound at the bottom of the planting hole.
-Remove tree from container or grow bag and completely tease apart root system, repositioning any girdling or potentially girdling roots.
-Spread roots over soil mound so that root flare is at finished grade and the tree is straight.
-With clean, sharp pruning tools, prune off any secondary/adventitious, girdling, and potential girdling roots.
-Backfill planting hole with existing unamended soil and thoroughly water.
-Mulch the entire planting surface with composted bark applied no less than two inches (2") deep and no more than three inches (3") deep, leaving three inches (3”) adjacent to the tree trunk free of mulch.

Planting Bare Root Trees:

-Dig tree hole at least two times wider than the tree ball with sloping sides. Dig hole to a depth so the located root flare, at the first order lateral root, will be at finished grade.
-Create a firm soil mound at the bottom of the planting hole.
-Spread roots over soil mound so that root flare is at finished grade and the tree is straight.

-With clean, sharp pruning tools, prune off any secondary/adventitious, girdling, and potential girdling roots.
-Backfill planting hole with existing unamended soil and thoroughly water.
-Mulch the entire planting surface with composted bark applied no less than two inches (2") deep and no more than three inches (3") deep, leaving three inches (3") adjacent to the tree trunk free of mulch.



1000 Performance Based Project Development (PBPD)

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1000 Performance Based Project Development (PBPD)

1000 INTRODUCTION

Performance Based Project Development (PBPD) is a planning and design philosophy being promoted by the Federal Highway Administration (FHWA) and State Departments of Transportation. The general premise of PBPD is that proposed improvements should be targeted and right sized based on project specific needs.

This philosophy places less emphasis on strict adherence to standards and more significance on safety and operational performance. PBPD is a design up philosophy that makes the necessary improvements to a facility to address specific performance issues. The goal of PBPD is to fix what is broken and to not unnecessarily spend scarce resources solely for the purpose of meeting published standards when those deficient features (as defined by our manuals) are not causing safety, operational or similar problems.

PBPD is a philosophy that will permeate every aspect of the ODOT project development process. PBPD is used to right size the initial scope of a project and will continue through preliminary engineering as well as into detail design. The goal of PBPD is to right size a project scope and the subsequent engineering decisions through the application of data, analysis and engineering judgement.

It is understood that no one project is more important than the overall system. Consequently, savings obtained from targeted PBPD solutions on specific projects can be reinvested in the overall system.

In ODOT, the basic tenants of PBPD will be:

- Safety will not be compromised;
- In many cases, the minimum standard will be the existing condition;
- A project Purpose & Need should be focused to address specific problems supported by data;
- Design solutions should focus on adherence to the project Purpose & Need;
- Solutions should be an optimized combination of mobility, operations and other modes;
- Designs should be consistent with the context of the corridor;
- Designs should strive to maximize benefit/cost.

PBPD may be applicable to any project and can occur at various phases of the project development process. Planning, Preliminary Engineering, Environmental Engineering and Design can all afford PBPD opportunities. Some projects will, however, obviously have more potential PBPD possibilities than others. Path 1 projects by their very nature will have limited PBPD opportunities. Conversely, more complex projects will generally have more opportunities for meaningful PBPD considerations.

1001 APPLICATION TO THE PDP

PBPD can happen throughout project development and can be applied to any number of project processes and elements. The key elements to PBPD will be thoughtful application, balanced consideration and documentation. An effort should be made to stay on the existing alignment (vertical and horizontal) to the maximum extent practical when this will allow for the rehabilitation of existing infrastructure (structures, pavements, median barriers, etc.) rather than construct new infrastructure on a different alignment. In all cases, however, the decisions related to PBPD should consider and balance the benefits, disadvantages, costs & impacts.

1000 Performance Based Project Development (PBPD)

1001.1 Planning

PBPD in the Planning phase of a project does not address specific design criteria; rather, it relates to the scope by establishing the Needs of a project. Thoughtful establishment of a project's Needs allows the project to focus on fixing critical elements and save funding by potentially not fixing those elements that aren't deemed to be unacceptably impactful to safety, operations, system conditions, stakeholder concerns or other similar factors.

During the planning phase technical studies will be used to identify those features that are critical and those features that would only be addressed as budget and impacts (benefit/cost) allow. The product of this effort will be the identification of Primary and Secondary Needs that will be incorporated into the project Purpose and Need. Stakeholder involvement can be a useful tool in identifying the context or relative importance of Primary and Secondary needs as identified through technical studies. Subsequent Preliminary Engineering activities such as the Feasibility Study, AER, IMS, etc. would identify solutions to the Primary and Secondary Needs with a determination if the impacts and cost justify addressing a particular Secondary Need as part of the project.

Primary Needs are those elements of a project that are critical for the project to address in order to satisfy the Purpose and Need. Secondary Needs are essentially elements that may or may not be addressed by the project based upon the cost and impacts they create. The technical studies will be the basis to classify a particular need as being Primary or Secondary. In practical terms, Primary Needs are those that the project must address to the extent feasible. Secondary Needs do not rise to the level of a Primary Need based upon the technical studies (ex. Safety, HCS, HSM, System Conditions, etc.), the context of the project or stakeholder concerns and may ultimately not be addressed by the project depending upon their costs and the impacts they create. A project element may not meet existing standards, policy or guidance and still could be considered a Secondary Need if it is not unduly impacting safety, operations, maintenance or stakeholder concerns.

For example, a common application of this might be a Secondary Need to "Improve Existing Geometrics". In this example some of the existing geometric elements within the project limits may not meet full L&D Volume 1 standards but the analysis/technical studies performed to craft the P&N shows that these deficient elements are not creating any undue safety or operational problems. In this situation, the project would strive to improve (design up) those geometric features identified as Secondary Needs until the cost and impacts are deemed to be impractical in terms of benefit/cost. The term improve should be noted as the goal is to make improvements to the extent practical, not necessarily to meet full standards if the cost or impacts outweigh the benefits.

During identification of Needs and the subsequent formulation of the Purpose & Need no decision is made about if the project will address a particular Secondary Need. That determination will be made during Preliminary Engineering through the Feasibility Study and AER (if applicable) when sufficient alternative information is available to make a balanced decision. The level of effort to determine if a Secondary Need should be addressed should be as minimal as possible in order to make an informed benefit/cost decision. In addition to determining the cost/impact ramifications of addressing a Secondary Need, the original technical studies (ex. existing crashes and operational performance) may be referred to as a potential information source when making these decisions. Some decisions will be obvious to a practical person and minimal effort will be required in order to document the decision to not address a Secondary Need.

Establishment of Secondary Needs creates a powerful opportunity in the subsequent Preliminary Engineering phase for PBPD savings in terms of impacts and cost savings.

The following is a non-all-inclusive list of some potential PBPD opportunities for consideration in Planning:

- Addressing secondary needs
- Design Year
- Design Volumes
- Design Speed (Context of Adjacent Corridor)
- Rehab versus replace

1001.2 Preliminary Engineering

Projects should always address the Purpose & Need to the extent practical and possible. This is especially true of the Primary Needs incorporated into the Purpose & Need document. The Secondary Needs, however, represent a significant opportunity to apply PBPD principals in order to minimize impacts or contain costs.

The Feasibility Study and AER (where applicable) is used to evaluate alternatives to address the Purpose & Need. Included should be an assessment of impacts and costs necessary to address Secondary Needs of the project. PBPD philosophy dictates that the benefits of addressing a Secondary Need is balanced against the impacts and costs in order to make an informed decision. When the benefits outweigh the impacts and costs the Secondary Need should be addressed by the project. In those cases when the impacts and/or costs are deemed to outweigh the benefits the project may opt to not include addressing that particular Secondary Need or to make an improvement that falls short of meeting the standard. This same methodology can be applied to any number of potential project elements identified as Secondary Needs.

It will often not be appropriate for PBPD decisions during Planning and Preliminary Engineering to be made unilaterally at the project level. Some decisions could affect subsequent processes or approvals (ex. a Structure Type Study, IMS or Design Exceptions). It is important for evaluations of PBPD decisions in Planning or Preliminary Engineering to include the appropriate disciplines, process owners and approvers in order to avoid potential back tracking later in the development process. The owners of those subsequent decision points should be involved in the evaluation and acceptance of PBPD decisions during Planning and Preliminary Engineering. For example, an interchange improvement may identify improving one specific movement as being a Primary Need. The Office of Roadway Engineering (ORE) is responsible for the eventual approval of the subsequent IOS/IMS. In this case ORE should be involved in the decision to not address the other movements in order to avoid potential back tracking during evaluation of the IOS/IMS.
The following is a non-all-inclusive list of some potential PBPD opportunities in Preliminary Engineering:

- Operational Performance
  - LOS Criteria
  - 600’ of Access Control along the crossroad for diamond ramps
- Alternatives
  - Facility Type (expressway, freeway, super 2, etc.)
  - Grade separation versus unconventional intersection
- Geometrics
  - Ramp Terminal Spacing
  - Removal of inside merges
  - Removal of left exits or entrance ramps

1001.3 Design

Design in this context refers to Stage 1, Stage 2 and Stage 3 plans. As such, Design centric PBPD opportunities can occur during Preliminary Engineering, Environmental Engineering or Final Engineering depending on the project path/PDP requirements.

PBPD Design is a "design up" philosophy rather than the traditional approach to make every facet of a facility meet every standard even when those deficient standards (as defined by our manuals) are not causing any operational or safety problems.

PBPD in Design is typically associated with meeting specific geometric design standards and guidelines. The application of PBPD in Design requires a departure from the traditional mentality that meeting design standards is a metric, either formally or informally, to measure the success of a given design. Meeting standards is a worthwhile goal, however, in many instances the cost of meeting all of the design standards can be prohibitively expensive or impactful; sometimes to upgrade substandard geometric features that are not causing undue problems.

Design has formally recognized the concept of PBPD for a very long time through the Design Exception (D.E.) process. Where it is not practical to meet one of the Controlling Criteria due to various impacts and/or costs, a Design Exception is a formal approval process to evaluate and document the decisions (Refer to L&D Volume 1, ***Section 105***). Design Exceptions will continue to be the primary method to document deviations from the Controlling Criteria during Design. Deviation from other Design criteria (non-Controlling Criteria) that do not require Design Exceptions should also be evaluated and documented throughout project development.

It should be noted that application of PBPD in Design can still be beneficial when addressing Primary Needs of the project. The project can still address the Primary Need while reducing impacts and/or costs through the application of Design based PBPD to specific design criteria related to the Primary Need.

Proposed deviations from standards, policies and guidelines must be balanced decisions that considers the potential advantages and disadvantages of the proposal. The decision should be based when appropriate upon a quantitative comparison potentially including historical crashes, predicted crashes (HSM), capacity analysis (HCS), turn and vehicle tracking and similar methodologies. The analytical effort should be commensurate with the proposal and in all cases should be documented. For proposals requiring deviation from Controlling Criteria, a Design Exception will be required.

The following is a non-all-inclusive list of some potential PBPD opportunities in Design:

- Geometrics
  - Typical/ Cross Section (for both cost/impact avoidance or context)
  - Median width
  - Roadside grading (safety, clear zone, common or barrier)
  - Cross slope corrections
  - Upgrading entrance ramp terminals including additional acceleration length if needed
  - Upgrading exit ramp terminals by providing the standard diverging curvature
  - Providing the minimum ramp design speed

1002 EVALUATION OF PBPD OPTIONS

1002.1 General

PBPD decisions should be informed, weighing both the positive and negative impacts (B/C). There is no single process that can be used to evaluate PBPD opportunities. The evaluation methodology and level of effort should be commensurate with the complexity of the situation. When appropriate, decisions should be based on comparative quantification through the use of HCS, HSM, vehicle tracking (for narrow lanes), simulation, technical studies, or similar evaluations.

The level of analysis should be commensurate with the proposal and the facility type. For example, the level of analysis to “squeeze in” an additional interstate lane with resulting narrower lane and shoulder widths would be significantly higher than a proposed spot narrowing of a shoulder underneath an overhead structure.

The historical approach of absolute adherence to standards has primarily been used as a proxy measure of a design’s expected safety performance. With the advent of the Highway Safety Manual (HSM) and its predictive safety performance methodologies we can now quantify the expected ramifications of many PBPD opportunities.

1002.2 HSM for Evaluation

HSM is an analytical tool, which in some cases, can be used to compare the expected crashes between different alternatives. HSM, like other analytical tools, should not normally be the sole basis of making decisions. It can, however, be a factor providing a quantified comparison of potential safety performance in terms of expected crashes.

When appropriate and when the situation does not exceed the capabilities of the software (ECAT) or research data set, HSM can be used to compare expected crashes between alternatives. Safety should always be an important consideration, however, that does not mean an HSM analysis cannot predict an increase in crashes on any proposed alternatives. The question becomes what is the magnitude of the predicted crash increases and what are the associated severities.

For example, it may be perfectly appropriate for a PBPD alternative to accept a modest increase in property damage (PDO) crashes if the offsetting benefits afforded by the alternative are commensurately high.

Below is an example HSM analysis for a pilot PBPD project where:
KA= Fatalities and Incapacitating Injuries; B= Visible Injuries; C= Non-Visible Injuries; O=Property Damage

1000 Performance Based Project Development (PBPD)

Project Summary Results (Without Animal Crashes)

	KA	B	C	O	Total
Npredicted - Existing Conditions	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Npredicted - Proposed Conditions	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
				Difference	[illegible]

Project Summary Results (Without Animal Crashes)

	KA	B	C	O	Total
Npredicted - Existing Conditions	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Npredicted - Proposed Conditions	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
				Difference	[illegible]

Countermeasures installed

It should be noted that an increase to the expected crashes predicted by HSM may be potentially mitigated with the application of appropriate safety countermeasures. These countermeasures should be factored into the HSM analysis. In the above "Project Summary Results", the difference between the upper and lower comparisons is the lower has incorporated Safety Countermeasures to reduce the number of predicted crashes.

Refer to the Safety Analysis Guidelines maintained by the Office of Program Management for detailed analysis requirements.
http://www.dot.state.oh.us/Divisions/Planning/ProgramManagement/HighwaySafety/HSIP/SafetyAnalysisGuidelines/Safety_Analysis_Guidelines.pdf

1003 PBPD EXAMPLES

1003.1 Secondary Needs

Design preference is for ramps to enter/exit from the right hand side due to driver expectation. Converting a left hand ramp to a right hand ramp can be costly or impactful. It might be considered a Secondary Need if the existing left hand ramp is not causing undue safety or operational problems. If the project has interest in potentially retaining a left hand ramp a technical study should be conducted during Planning to determine if the existing left hand ramp is causing undue safety or operational problems. Based upon the results of the technical studies, the project may potentially determine that relocating the ramp to the right side is only a Secondary Need of the project.

The Feasibility Study and/or the AER would determine the actual potential impacts and costs of switching the ramp to the right hand side. A determination about the project addressing this Secondary Need would be made based upon the impacts and costs versus the benefits (B/C).

1003.2 Design Year/LOS

A PBPD Preliminary Engineering opportunity could be related to the proposed alternatives and Design Year/Level-of-Service (LOS) requirements. For example - an expressway with at-grade signalized intersections is suffering from congestion and congestion related crashes. A Primary Need of the project might be to improve corridor capacity. Note that the Primary Need is not to meet a specified LOS, but to improve capacity. This provides for flexibility in identifying and evaluating alternatives. In these types of situations it would traditionally be common and logical to propose variations of grade separations as the alternatives. The cost of these new interchanges could, however, be prohibitively expensive with little likelihood of funding being available for the foreseeable future.

A PBPD alternative might be to also propose alternative intersection designs such as a Superstreet signalized intersection. The Superstreet operation would not provide the same capacity as a grade separation and it might not potentially meet Design Year LOS criteria, however it does meet the Purpose and Need of the project by improving corridor capacity. It could work acceptably for many years and it would provide significant capacity improvements as compared to the existing (no-build) traditional traffic signals. The Superstreets would also be much less expensive and have the potential to obtain the necessary construction funding. In this case, the Superstreet meets the Primary Need (though not to the extent of a grade separation) by improving corridor capacity but it could potentially fail to meet 20 year LOS criteria. In the context of PBPD, a determination would have to be made if the benefits provided outweigh the costs and impacts of the grade separation.

1003.3 Bottleneck Projects

Bottlenecks are capacity constraints typically contained in a fairly small geographic area. They can be caused by insufficient capacity due to the number of basic mainline lanes, weaves, merges or other geometric features that cause traffic operational problems.

A potential PBPD solution might be to “squeeze in” an additional lane which may result in less than standard lane widths, shoulder widths or barrier offsets. The Performance of the proposed resulting typical section would be the basis of determining if this type of alternative should be approved, specifically:

1. Capacity analysis can be used to predict the operational benefits;
2. HSM can be used to predict the expected safety performance of the proposed typical section versus the existing (example – expected crash performance related to narrower lanes, shoulders and offsets to barriers versus benefits from reduced congestion);
3. Design vehicle tracking could be investigated to ensure there is not infringement on adjacent lanes;
4. Investigation of hydraulic spread on a reduced shoulder width could be necessary;
5. Others analysis and considerations as the situation and proposal require;

Like all PBPD decisions, the results of the alternative analysis would be the basis of deciding if the benefits of a “squeezed in” lane outweigh the negatives.

1003.4 Context

PBPD can be applied to establish a project’s design criteria based upon the surrounding context or stakeholder input. An example of this might be a corridor that is winding with narrow lanes and shoulders throughout its length. It may not make sense and may potentially be prohibitively expensive and impactful to improve one section of that roadway to meet full geometric standards. In this case, the project might set geometric goals to improve the geometric design to a practical extent while not attempting to meet full standards. In the end, the corridor will always be a winding road with little likelihood of ever being relatively straight and flat (meeting our standards). Meeting standard in one small section of the corridor, far exceeding the context of the rest of the corridor, might provide relatively little benefit at a relatively large expense. The cost of fully meeting standards would deprive the necessary construction funds from being available to meet other system needs.

It may be practical and appropriate to set design criteria, operational performance requirements and other project level metrics based upon the context of the corridor as well as stakeholder input.

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(APPENDIX A & C HAVE BEEN REMOVED)

APPENDIX B: ROADWAY SAMPLE PLAN NOTES

NUMBER	DATE	NAME	REFERENCED SECTION
R111	07/2013	Connection Between Existing and Proposed Guardrail	n/a
R112a	07/2013	Item 606 – Anchor Assembly, MGS Type B	603.3.2
R113a	07/2013	Item 606 – Anchor Assembly, MGS Type E	603.3.3
R116	07/2014	Paving Under Guardrail	n/a
R118	10/2009	Item Special – Mailbox Support	n/a
R123	04/2011	Item 606 – Impact Attenuator, Type 1 (Unidirectional or Bidirectional)	603.4.1
R124	04/2011	Item 606 – Impact Attenuator, Type 2 (Unidirectional or Bidirectional)	603.4.2
R125	04/2011	Item 606 – Impact Attenuator, Type 3 (Unidirectional or Bidirectional)	603.4.3
R127	04/2011	Item 606 – Cable Guardrail	602.2.2.2

Appendix B: Roadway Sample Plan Notes

R111 – CONNECTION BETWEEN EXISTING AND PROPOSED GUARDRAIL

WHEN IT IS NECESSARY TO SPLICE PROPOSED GUARDRAIL TO EXISTING GUARDRAIL, ONLY THE EXISTING GUARDRAIL SHALL BE CUT, DRILLED, OR PUNCHED. THE CONNECTION SHALL BE MADE USING A W-BEAM, BEAM SPLICE AS SHOWN IN AASHTO M 180-12, EXCEPT THE BEAM WASHERS ARE NOT TO BE USED. PAYMENT SHALL BE INCLUDED IN THE CONTRACT PRICE FOR THE RESPECTIVE GUARDRAIL ITEMS.

Designer Notes: Use this note when connections are required between existing and proposed guardrail runs. Locations shall be noted on the plans.
Use Standard Drawing MGS-4.3 Guardrail Transitions when connecting MGS to Type 5 Guardrail.

R112a – ITEM 606 – ANCHOR ASSEMBLY, MGS TYPE B

THIS ITEM SHALL CONSIST OF FURNISHING AND INSTALLING ANY OF THE GUARDRAIL END TERMINALS FOR TYPE MGS GUARDRAIL AS LISTED ON ROADWAY ENGINEERING'S WEB PAGE UNDER ROADSIDE SAFETY DEVICES FOR APPROVED GUARDRAIL END TREATMENTS. INSTALLATION SHALL BE AT THE LOCATIONS SPECIFIED IN THE PLANS, IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

REFER TO THE MANUFACTURER'S INSTRUCTIONS REGARDING THE INSTALLATION OF, AND THE GRADING AROUND, THE FOUNDATION TUBES AND GROUND STRUT. THE TOP OF ANY FOUNDATION TUBE SHOULD BE LESS THAN 4 INCHES ABOVE THE GROUND. THE PLACEMENT OF THE FOUNDATION TUBES SHOULD BE AN APPROPRIATE DEPTH BELOW THE LEVEL LINE IN ORDER TO MAINTAIN THE FINISHED GUARDRAIL HEIGHT OF 31 INCHES FROM THE EDGE OF THE SHOULDER.

ON-SITE GRADING IS REQUIRED IF THE TOP OF THE FOUNDATION TUBES OR TOP OF THE GROUND STRUT DOES PROJECT MORE THAN 4 INCHES ABOVE THE GROUND LINE.

THE FACE OF THE TYPE B IMPACT HEAD SHALL BE COVERED WITH TYPE G REFLECTIVE SHEETING, PER CMS 730.19.

PAYMENT FOR THE ABOVE WORK SHALL BE MADE AT THE UNIT PRICE BID FOR ITEM 606, ANCHOR ASSEMBLY, MGS TYPE B, EACH, AND SHALL INCLUDE ALL LABOR, TOOLS, EQUIPMENT AND MATERIALS NECESSARY TO CONSTRUCT A COMPLETE AND FUNCTIONAL ANCHOR ASSEMBLY SYSTEM, INCLUDING REFLECTIVE SHEETING AND ALL RELATED HARDWARE, GRADING, EMBANKMENT AND EXCAVATION NOT SEPARATELY SPECIFIED, AS REQUIRED BY THE MANUFACTURER.

Designer Notes:

1. The length of need (LON) point is at post number 3; therefore, after calculating the required LON for the guardrail, deduct the last 25'-0" of the unit (from post #3 to post #9) from the length of need for the guardrail. The designer should show the LON point on all guardrail runs in the plans.
2. Pre-approved shop drawings are reviewed and are on the Office of Roadway Engineering's web page under Roadside Safety Devices.
3. These end treatments are gating systems.
4. The standard offset at post #1 for the B is 4'-0". This offset can be reduced to a minimum of 3'-0" at locations where the 4'-0" offset is impractical.
5. Use this plan note in conjunction with Type MGS Guardrail.

R113a – ITEM 606 – ANCHOR ASSEMBLY, MGS TYPE E

THIS ITEM SHALL CONSIST OF FURNISHING AND INSTALLING ANY OF THE GUARDRAIL END TERMINALS FOR TYPE MGS GUARDRAIL AS LISTED ON ROADWAY ENGINEERING'S WEB PAGE UNDER ROADSIDE SAFETY DEVICES FOR APPROVED GUARDRAIL END TREATMENTS. INSTALLATION SHALL BE AT THE LOCATIONS SPECIFIED IN THE PLANS, IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

THE FACE OF THE TYPE E IMPACT HEAD SHALL BE COVERED WITH A SHEET OF TYPE G REFLECTIVE SHEETING, PER CMS 730.19.

REFER TO THE MANUFACTURER'S INSTRUCTIONS REGARDING THE INSTALLATION OF, AND THE GRADING AROUND THE FOUNDATION TUBES AND GROUND STRUT. THE TOP OF ANY FOUNDATION TUBE SHOULD BE LESS THAN 4 INCHES ABOVE THE GROUND. THE PLACEMENT OF THE FOUNDATION TUBES SHOULD BE AN APPROPRIATE DEPTH BELOW THE LEVEL LINE IN ORDER TO MAINTAIN THE FINISHED GUARDRAIL HEIGHT OF 31 INCHES FROM THE EDGE OF THE SHOULDER.

ON-SITE GRADING IS REQUIRED IF THE TOP OF THE FOUNDATION TUBES OR TOP OF THE GROUND STRUT DOES PROJECT MORE THAN 4 INCHES ABOVE THE GROUND LINE.

PAYMENT FOR THE ABOVE WORK SHALL BE MADE AT THE UNIT PRICE BID FOR ITEM 606, ANCHOR ASSEMBLY, MGS TYPE E, EACH, AND SHALL INCLUDE ALL LABOR, TOOLS, EQUIPMENT AND MATERIALS NECESSARY TO CONSTRUCT A COMPLETE AND FUNCTIONAL ANCHOR ASSEMBLY SYSTEM, INCLUDING ALL RELATED TRANSITIONS, REFLECTIVE SHEETING, HARDWARE, GRADING, EMBANKMENT AND EXCAVATION NOT SEPARATELY SPECIFIED, AS REQUIRED BY THE MANUFACTURER.

Designer Notes:

1. The length of need (LON) point for both systems is at post number 3; therefore, after calculating the required LON for the guardrail, deduct the last 37'-6" of the unit (from post #3 to post #9) from the length of need for the guardrail. The designer should show the LON point on all guardrail runs in the plans.

2. Pre-approved shop drawings are reviewed and are on the Office of Roadway Engineering's web page under Roadside Safety Devices.

3. These end treatments are gating systems.

4. A Type C delineator should be installed on a flexible post at the head of all Type E units located on the right side of the through roadway in areas that have known snowdrift/piling problems, or per District policy. A Type D delineator should be installed on a flexible post at the head of all Type E units located on the left side of the through roadway. Delineators shall be itemized separately and shall comply with Standard Construction Drawing TC-61.10 and CMS 620.

5. Use this plan note in conjunction with Type MGS Guardrail.

Appendix B: Roadway Sample Plan Notes

R116-PAVING UNDER GUARDRAIL

THIS OPERATION SHALL INCLUDE PREPARATION OF THE GRADED SHOULDER USING ITEM 209, LINEAR GRADING, AS PER PLAN AND PAVING UNDER THE GUARDRAIL USING 411 ASHPALT CONCRETE INTERMEDIATE COURSE, TYPE 1, (448), UNDER GUARDRAIL, AS PER PLAN.

ITEM 209, LINEAR GRADING, AS PER PLAN SHALL CONSIST OF EXCAVATING TOPSOIL, AND PLACING GRANULAR MATERIAL.

ALL COLLECTED DEBRIS AND TOPSOIL, INCLUDING RHIZOMES, ROOTS AND OTHER VEGETATIVE PLANT MATERIAL SHALL BE REMOVED AND DISPOSED OF AS SPECIFIED IN 105.17.

THE REMOVED MATERIAL SHALL BE REPLACED WITH COMPACTABLE GRANULAR MATERIAL CONFORMING TO 703.16 PLACED TO GRADE AS DETAILED ON THE TYPICAL SECTION OR AS APPROVED BY THE ENGINEER.

ALL EQUIPMENT, MATERIALS AND LABOR REQUIRED TO PERFORM THE WORK OUTLINED ABOVE SHALL BE INCLUDED FOR PAYMENT UNDER ITEM 209, LINEAR GRADING, AS PER PLAN.

PAVING UNDER GUARDRAIL SHALL CONSIST OF PLACING ITEM 441 TO THE DEPTH SPECIFIED USING ONE OF THE FOLLOWING METHODS:

METHOD A:

1. SET GUARDRAIL POSTS
2. PLACE ITEM 441

METHOD B:

1. PLACE ITEM 441
2. BORE ASPHALT AT POST LOCATIONS (MAY BE OMITTED IF STEEL POSTS ARE USED)
3. SET GUARDRAIL POSTS
4. PATCH AROUND POSTS. THE MATERIALS USED FOR PATCHING SHALL BE AN ASPHALT CONCRETE APPROVED BY THE ENGINEER. PATCHED AREAS SHALL BE COMPACTED USING EITHER HAND OR MECHANICAL METHODS. FINISHED SURFACES SHALL BE SMOOTH AND SLOPED TO DRAIN AWAY FROM THE POSTS.

ALL EQUIPMENT, MATERIALS AND LABOR REQUIRED TO PERFORM THE WORK OUTLINED ABOVE, WITH THE EXCEPTION OF SETTING GUARDRAIL POSTS, SHALL BE INCLUDED FOR PAYMENT UNDER ITEM 441, ASPHALT CONCRETE, INTERMEDIATE COURSE, TYPE 1(448), UNDER GUARDRAIL, AS PER PLAN.

Designer Notes: Quantities for Item 441 should be calculated in Cubic Yards. The asphalt concrete thickness should be shown on the typical sections. The depth may vary according to project requirements, but shall be a maximum of 3 inches. The area to be paved shall be from the edge of the paved shoulder to the break point between the graded shoulder and the foreslope. The slope shall be the same as the graded shoulder slope. The designer may specify either paving Method A or B, or leave the option to the contractor. Guardrail shall be paid for under Item 606.

R118 – ITEM SPECIAL – MAILBOX SUPPORT

THIS WORK SHALL CONSIST OF FURNISHING AND ERECTING MAILBOX SUPPORTS AND ANY ASSOCIATED MOUNTING HARDWARE IN ACCORDANCE WITH PLAN DETAILS, AND ATTACHING AN OWNER-SUPPLIED MAILBOX AT LOCATIONS SPECIFIED IN THE PLAN, OR OTHERWISE ESTABLISED BY THE ENGINEER.

WOOD POSTS SHALL BE NOMINAL 4 INCHES BY 4 INCHES SQUARE OR 4.5 INCHES DIAMETER ROUND, AND CONFORM TO 710.14.

STEEL POSTS SHAL BE NOMINAL PIPE SIZE 2 INCHES I.D., AND CONFORM TO AASHTO M 181.

ALL HARDWARE INCLUDING BUT NOT LIMITED TO PLATES, SCREWS, BOLTS, AND ETC. SHALL BE COMMERCIAL-GRADE GALVANIZED STEEL.

POSTS SHALL BE SET PER THE FIRST PARAGRAPH OF 606.03, AND SHALL IN NO INSTANCE BE ENCASED IN CONCRETE.

SUPPORT HARDWARE SHALL ACCOMMODATE EITHER A SINGLE OR A DOUBLE MAILBOX INSTALLATION, AND NO MORE THAN TWO BOXES MAY BE MOUNTED ON A SINGLE POST.

THE MAILBOX SHALL BE SECURELY AND NEATLY ATTACHED BY THE CONTRACTOR TO THE NEW SUPPORT. THE CONTRACTOR SHALL FURNISH ALL NECESSARY ATTACHMENT HARDWARE (NUTS, BOLTS, PLATES, SPACERS, AND WASHERS) AS NECESSARY TO ACCOMMODATE THE COMPLETE INSTALLATION.

IN THE ABSENCE OF A NEW BOX SUPPLIED BY THE OWNER, THE CONTRACTOR SHALL SALVAGE THE EXISTING BOX AND PLACE IT ON THE NEW SUPPORT. DUE CARE SHALL BE EXERCISED IN SUCH AN OPERATION, AND THE CONTRACTOR SHALL BE RESPONSIBLE FOR REPAIRING OR REPLACING ANY BOX DAMAGED BY IMPROPER HANDLING ON HIS PART, AS JUDGED AND DIRECTED BY THE ENGINEER.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR COORDINATING WITH THE LOCAL POST MASTER REGARDING THE TIMING OF THE MOVEMENT OF ANY MAILBOX TO A NEW LOCATION.

PAYMENT UNDER THIS ITEM SHALL BE LIMITED TO FINAL PERMANENT INSTALLATIONS, TEMPORARY INSTALLATIONS SHALL BE IN ACCORDANCE WITH 107.10. HOWEVER, THE SAME MATERIAL AND SIZE LIMITATIONS AS FOR PERMANENT INSTALLATIONS SHALL APPLY.

MAILBOX SUPPORTS, COMPLETE IN PLACE, WILL BE PAID FOR AT THE CONTRACT UNIT PRICE PER EACH, FOR ITEM SPECIAL MAILBOX SUPPORT SYSTEM, (SINGLE) (DOUBLE).

Designer Notes: The above note should be used for the replacement of existing mailbox supports constructed of materials which may be considered “hazardous” because they exceed the size stated with the note. See Figure 803-1 in Volume One (Roadway Design) of the Location and Design manual for more information.

R123 – ITEM 606 – IMPACT ATTENUATOR, TYPE 1 (UNIDIRECTIONAL OR BIDIRECTIONAL)

THIS ITEM SHALL CONSIST OF FURNISHING AND INSTALLING ANY ONE OF THE TYPE 1 IMPACT ATTENUATORS AS LISTED ON THE OFFICE OF ROADWAY ENGINEERING'S WEB PAGE. INSTALLATION SHALL BE AT THE LOCATIONS SPECIFIED IN THE PLANS, IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

THE FACE OF THE TYPE 1 IMPACT HEAD SHALL BE COVERED WITH A SHEET OF TYPE G REFLECTIVE SHEETING, PER CMS 730.19. PAYMENT FOR THE ABOVE WORK SHALL BE MADE AT THE UNIT PRICE BID FOR ITEM 606, IMPACT ATTENUATOR, TYPE 1(UNIDIRECTIONAL OR BIDIRECTIONAL), EACH, AND SHALL INCLUDE ALL LABOR, TOOLS, EQUIPMENT AND MATERIALS NECESSARY TO CONSTRUCT A COMPLETE AND FUNCTIONAL IMPACT ATTENUATOR SYSTEM, INCLUDING ALL RELATED TRANSITIONS, HARDWARE, REFLECTIVE SHEETING AND GRADING, NOT SEPARATELY SPECIFIED, AS REQUIRED BY THE MANUFACTURER.

Designer Notes:

1. After calculating the required Length of Need for the guardrail, deduct the last 12'-6" of the unit from the length of need for the guardrail. The designer should show the LON point on all guardrail runs in the plans. Refer to the approved products listed on the Office of Roadway Engineering's Web Page.

2. The 6'-3" section directly behind the Type 1 shall be parallel to the centerline of the unit. A maximum flare of 3 degrees (20:1) is permissible. A cross slope of no more than 8% (5 degrees) is recommended.

3. Bidirectional should be specified for locations where traffic is expected to be in opposing directions on either side of the barrier. Unidirectional shall be specified when traffic is expected to move in the same direction on both sides of the barrier.

4. All curbs and islands should be removed for optimum impact performance.

5. More information is located in Section 600 of the Location and Design Manual Volume 1.

Appendix B: Roadway Sample Plan Notes

R124 – ITEM 606 – IMPACT ATTENUATOR, TYPE 2 (UNIDIRECTIONAL OR BIDIRECTIONAL)

THIS ITEM SHALL CONSIST OF FURNISHING AND INSTALLING ANY OF THE TYPE 2 IMPACT ATTENUATORS AS LISTED ON THE OFFICE OF ROADWAY ENGINEERING'S WEB PAGE (REFER TO THE POSTED SHOP DRAWINGS FOR THE MOST CURRENT APPROVED PRODUCT MODELS). WHEN BI-DIRECTIONAL DESIGNS ARE SPECIFIED, THE CONTRACTOR SHALL SUPPLY APPROPRIATE TRANSITIONS. THE FACE OF THE IMPACT HEAD SHALL BE COVERED WITH TYPE G REFLECTIVE SHEETING, PER CMS 730.19.

PAYMENT FOR THE ABOVE WORK SHALL BE MADE AT THE UNIT PRICE BID FOR ITEM 606, IMPACT ATTENUATOR, TYPE 2 [(SPEED (IN MPH), HAZARD WIDTH (IN INCHES)), (UNIDIRECTIONAL OR BIDIRECTIONAL)], EACH, AND SHALL INCLUDE ALL LABOR, TOOLS, EQUIPMENT AND MATERIALS NECESSARY TO CONSTRUCT A COMPLETE AND FUNCTIONAL IMPACT ATTENUATOR SYSTEM, INCLUDING ALL RELATED BACKUPS/BACKSTOPS, TRANSITIONS, HARDWARE AND GRADING, NOT SEPARATELY SPECIFIED, AS REQUIRED BY THE MANUFACTURER. INSTALLATION SHALL BE AT THE LOCATIONS SPECIFIED IN THE PLANS, IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

<u>Designer Notes:</u>
These systems are non-gating and redirective therefore the entire length of the unit can be included as part of the calculated length of need.

The most current approved products and models are updated regularly online, as such, individual products should generally not be listed on the plans.

This note should be used for the protection of Type 5 Barrier Design Guardrail, concrete median barrier and other fixed objects.

If cross slopes are steeper than 8% (12:1) or if the cross slope varies by more than 2% over the length of the unit, a leveling pad may be used.

Rear fender panels may slide 60 inches rearward upon impact, so ensure the specified width is adequate.

Bidirectional should be specified for locations where traffic is expected to be in opposing directions on either side of the barrier. Unidirectional shall be specified when traffic is expected to move in the same direction on both sides of the barrier.

Each of the Type 2 products have a wide variety of related units (families), typically covering various design speeds (number of bays) and protected widths. The designer should also identify on the project plans for each unit specified on the plan any contingencies needed to construct a complete device. They include:

- Design speed (The designer must specify Test level 3 (TL-3) configurations for installations on the NHS)
- Width of hazard
- Available foot print area for the product
- Foundation type (asphalt, concrete, bridge deck)
- Transition type (concrete barrier or guardrail)
- Backup support (A standard concrete backup is detailed on SCD RM-4.6. Otherwise, specify an independent stand-alone anchorage like the product's own concrete backup, or its tension strut backup)
- Any unique characteristics of the site (curb, expansion joints, etc.)

R125 – ITEM 606 – IMPACT ATTENUATOR, TYPE 3 (UNIDIRECTIONAL OR BIDIRECTIONAL)

THIS ITEM SHALL CONSIST OF FURNISHING AND INSTALLING ANY OF THE TYPE 3 IMPACT ATTENUATORS AS LISTED ON THE OFFICE OF ROADWAY ENGINEERING'S WEB PAGE (REFER TO THE POSTED SHOP DRAWINGS FOR THE MOST CURRENT APPROVED PRODUCT MODELS). WHEN BI-DIRECTIONAL DESIGNS ARE SPECIFIED, THE CONTRACTOR SHALL SUPPLY APPROPRIATE TRANSITIONS. THE FACE OF THE IMPACT HEAD SHALL BE COVERED WITH TYPE G REFLECTIVE SHEETING, PER CMS 730.19.

PAYMENT FOR THE ABOVE WORK SHALL BE MADE AT THE UNIT PRICE BID FOR ITEM 606, IMPACT ATTENUATOR, TYPE 3 [(SPEED (IN MPH), HAZARD WIDTH (IN INCHES)), (UNIDIRECTIONAL OR BIDIRECTIONAL)], EACH, AND SHALL INCLUDE ALL LABOR, TOOLS, EQUIPMENT AND MATERIALS NECESSARY TO CONSTRUCT A COMPLETE AND FUNCTIONAL IMPACT ATTENUATOR SYSTEM, INCLUDING ALL RELATED BACKUPS/BACKSTOPS, TRANSITIONS, HARDWARE AND GRADING, NOT SEPARATELY SPECIFIED, AS REQUIRED BY THE MANUFACTURER. INSTALLATION SHALL BE AT THE LOCATIONS SPECIFIED IN THE PLANS, IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

Designer Notes: These systems are non-gating and redirective therefore the entire length of the unit can be included as part of the calculated length of need.

The most current approved products and models are updated regularly online, as such, individual products should generally not be listed on the plans.

This note should be used for the protection of Type 5 Barrier Design Guardrail, concrete median barrier and other fixed objects.

If cross slopes are steeper than 8% (12:1) or if the cross slope varies by more than 2% over the length of the unit, a leveling pad may be used.

Rear fender panels may slide 60 inches rearward upon impact, so ensure the specified width is adequate.

Bidirectional should be specified for locations where traffic is expected to be in opposing directions on either side of the barrier. Unidirectional shall be specified when traffic is expected to move in the same direction on both sides of the barrier.

Each of the Type 3 products have a wide variety of related units (families), typically covering various design speeds (number of bays) and protected widths. The designer should also identify on the project plans for each unit specified on the plan any contingencies needed to construct a complete device. They include:

- Design speed (The designer must specify Test level 3 (TL-3) configurations for installations on the NHS)
- Width of hazard & available footprint area for the product
- Foundation type (asphalt, concrete, bridge deck)
- Transition type (concrete barrier or guardrail)
- Backup support (A standard concrete backup is detailed on SCD RM-4.6. Otherwise, specify an independent stand-alone anchorage like the product's own concrete backup, or its tension strut backup)
- Any unique characteristics of the site (curb, expansion joints, etc.)

  The REACT 350 is 48 inches tall, if sight distance is needed where the attenuator will be installed the designer shall note the REACT 350 is not allowed at that location.

Appendix B: Roadway Sample Plan Notes

R127 – ITEM 606 – CABLE GUARDRAIL

THIS ITEM SHALL CONSIST OF FURNISHING AND INSTALLING ANY ONE OF THE HIGH TENSION FOUR CABLE GUARDRAIL SYSTEMS AS LISTED ON THE OFFICE OF ROADWAY ENGINEERING'S WEB PAGE. PAYMENT FOR THE ABOVE WORK SHALL BE MADE AT THE UNIT PRICE BID FOR ITEM 606, CABLE BARRIER WITH CONCRETE LINE POST FOUNDATION, AND ITEM 606 CABLE BARRIER, ANCHOR ASSEMBLY AND SHALL INCLUDE ALL LABOR, TOOLS, EQUIPMENT AND MATERIALS NECESSARY TO CONSTRUCT A COMPLETE AND FUNCTIONAL HIGH TENSION CABLE GUARDRAIL SYSTEM NOT SEPARATELY SPECIFIED, AS REQUIRED BY THE MANUFACTURER. THE LENGTH OF THE TENSIONED CABLE NECESSARY TO INSTALL A FUNCTIONAL ANCHOR SYSTEM SHALL BE INCLUDED IN ITEM 606, CABLE BARRIER WITH CONCRETE LINE POST FOUNDATION.

INSTALLATION SHALL BE AT THE LOCATIONS SPECIFIED IN THE PLANS, IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

SYSTEMS SHALL HAVE A MAXIMUM DEFLECTION OF 8 FEET AND THE MAXIMUM LONGITUDINAL DISTANCE BETWEEN POSTS SHALL BE 15 FEET.

INSTALLATION WILL BE A FOUR CABLE HIGH TENSION SYSTEM INSTALLED IN SOCKETED POSTS FOUNDATION WITH A FOUR FOOT WIDE "NO MOW STRIP".

CONTRACTOR SHALL PROVIDE BI-DIRECTIONAL DELINEATORS ON THE POSTS AT A MINIMUM INTERVAL OF 100 FEET AND ON ALL ANCHOR TERMINALS.

TRANSITIONS TO W-BEAM GUARDRAIL ARE NOT ALLOWED.

REFER TO MANUFACTURER FOR MAXIMUM OFFSET FROM BREAK POINT.

TORPEDO OR BULLET SPLICES ARE NOT ALLOWED. ALL CABLE SPLICES SHALL BE A SWAGED OR OPEN BODY DESIGN THAT ALLOWS FOR ANNUAL INSPECTION BETWEEN THE WEDGE AND STRANDS OF CABLE.

POSTS ARE SET IN SOCKETED CONCRETE FOUNDATIONS AND SHALL NOT BE PERMANENTLY INSTALLED UNTIL THEIR RESPECTIVE RUNS OF TENSIONED CABLE GUARDRAIL ARE READY FOR FINAL CONNECTION TO THE END TERMINAL ASSEMBLY. THE CONTRACTOR SHALL REPLACE ANY POSTS DAMAGED DURING INSTALLATION AS DETERMINED BY THE ENGINEER AT NO ADDITIONAL COST TO THE STATE.

<u>Designer Notes</u>:
High tension cable barrier systems shall only be installed to meet the requirements of Location and Design Manual Section 601.2 Median Barrier Warrants.

The most current approved products and models are updated regularly online, as such, individual products should generally not be listed on the plans.

Designer should look at the entire corridor before selecting which side of the median the cable will be installed on. At breaks in the runs of cable such as turnarounds the layout of the cable should limit the gating potential of the cable end treatments. Installing the end treatments behind the trailing bridge parapets can eliminate the gating part of the end treatments. When overlapping cable runs eliminate all of the gating part of the end treatments. Review Figure 602-3 and 602-4 of L&D Vol. 1 for appropriate layouts. Additional information is provided in Location and Design Manual Volume 1 Section 600 and the manufacturer.

Additional pay items primarily used in maintenance projects may include:

Appendix B: Roadway Sample Plan Notes

606E55020	SPECIAL – CABLE BARRIER, REPLACEMENT CABLE
606E55100	SPECIAL – CABLE BARRIER, CONCRETE LINE POST FOUNDATION
606E55110	SPECIAL – CABLE BARRIER, CONCRETE ANCHOR FOUNDATION WITH SLEEVE
606E55120	SPECIAL – CABLE BARRIER, CONCRETE SOCKETED FOUNDATION
606E55130	SPECIAL – CABLE BARRIER, TERMINAL POST, CAST IN PLACE
606E55140	SPECIAL – CABLE BARRIER, ANCHOR POST
606E55160	SPECIAL – CABLE BARRIER, TERMINAL STRUT
606E55170	SPECIAL – CABLE BARRIER, TURNBUCKLE
606E55180	SPECIAL – CABLE BARRIER, SPLICE
606E55190	SPECIAL – CABLE BARRIER, POST REFLECTOR
606E55200	SPECIAL – CABLE BARRIER, TENSIONING
606E55210	SPECIAL – CABLE BARRIER, ANCHOR RECONSTRUCTED
606E55220	SPECIAL – CABLE BARRIER, ANCHOR POST RESET

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PROCEDURES FOR DEVELOPING DESIGN DESIGNATIONS FOR NON-INTERSTATE BRIDGE REPLACEMENT/REHABILITATION PROJECTS

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF TECHNICAL SERVICES

NOVEMBER 17, 1993
REVISED SEPTEMBER 11, 1998

PROCEDURES FOR DEVELOPING DESIGN DESIGNATIONS
FOR NON-INTERSTATE BRIDGE

REPLACEMENT/REHABILITATION PROJECTS

The procedures contained in this revised procedural manual are to be used to develop design designations for non-interstate bridge replacement/rehabilitation projects. Such bridge projects may be located on U.S. highways, state routes, county routes, township routes, or local streets. These procedures replace those found in the manual dated November 17, 1993.

Traffic forecasts for bridge projects on the Interstate System must continue to be provided or certified by the Office of Technical Services. For these bridges, the district offices may either forward design designations to Technical Services or request that the Technical Services provide them. Design designations for bridge projects that include more than simple replacement/rehabilitation must also be provided by Technical Services. This would include bridges with major approach work, bridges on new alignments, or bridges that are part of major capacity addition projects.

Bridge projects within a metropolitan planning organization (MPO) area should be coordinated with the MPO. Since a bridge project involving federal funds must be included on the MPO's transportation improvement program (TIP), coordination should take place at the time the project is added to the TIP.

This responsibility was originally delegated to the district offices in November, 1993, based on approval from FHWA in a letter dated September 29, 1993 (see Appendix A). The original version of this manual was designed to provide a cookbook approach to developing design designations for bridge projects for use by the district offices. The changes are intended to make the projections more accurate by giving the district offices more flexibility in developing them.

The major changes in this edition of the manual are as follows:

- The elimination of generalized growth rates by county and their replacement with a statewide continuous range of rates to provide flexibility in the selection of an accurate rate specific to the site and the individual project;

- Changes in the terminology used to refer to the year of construction from "Current Year" to "Opening Year" to eliminate confusing "current" with the current calendar year or with the year of the most recent count data;

- Changing the Design Year to be either 12 or 20 years after the Opening Year, consistent with the draft Pavement Design Manual (paragraph 2.02.1.1);

- Providing a range of values for the selection of the K factor, the calculation of the DHV on the worksheet, and the replacement in the design designation of K with the DHV;

- Three choices for the D factor, depending on whether the bridge is within or outside an MPO's boundaries and whether the bridge is one-way or two-way;

- Providing a "comments" section on the worksheet for use in documenting the selection of the growth rate, for substituting refined output from the MPO models, for noting more detailed available truck information (the "B" and "C" components), and/or for noting directional imbalances in the ADT such as those found on bridges on routes over freeways between ramp termini;

- Dropping the request for forwarding completed design designations to Technical Services; and

- The update of terms to reflect current terminology in use in the department.

The worksheet, itself, is now larger, the equivalent of two 8 ½ by 11 inch sheets. However, the form can be reproduced as a two-sided 8 ½ by 11 inch form or side by side on an 11 by 17 inch sheet, etc.

Any comments or questions on the use of this manual, including the discovery of any errors or inconsistencies, should be directed to the Office of Technical Services at (614) 644-8195.

Worksheet Instructions (Note: the worksheet is found on pages 6 and 7.)

1A. Enter the PID.

1B. Enter the Count-Route-Log. If the project is not on the State System, enter an appropriate project identifier.

2A. Enter the Existing ADT. The ADT selected should be the most recent, accurate, seasonally adjusted 24-hour volume available. The most recent ADT may be obtained from the latest Traffic Survey Report (TSR) if the project is on the State System. Other data sources may be used (ODOT data obtained since the last TSR, count data from county engineers, MPOs, consultants, cities, etc.). Partial-day counts may be expanded to 24-hours using average values for the proportion of each hour in the daily total. Expansion tables and season adjustment factors can be obtained from the Office of Technical Services' Traffic Monitoring Section. If the available count data is three (3) years or older, consideration should be given to obtaining a new count.

2B. Enter the 24-hour B&C volume (trucks). If no data is available, leave this box blank.

2C. Enter the Existing Year. This is the year the count was taken. For TSR data, assume this is the year of the report (e.g., for a report published in 1996, assume the data is from 1996) unless the specific ADT is known to come from a count taken in an earlier year.

3. Enter the Opening Year. The Opening Year is the year construction will be completed and the bridge will reopen to traffic.

4. Enter the Design Year. The Design Year is either 12 or 20 years after the Opening Year. This is determined by the scope and intent of the project and is unlikely to be an option available to the user of this manual. Most projects will have a 20-year life; a 12-year design year would occur only when the bridge is part of an overall 12-year pavement rehabilitation project.

5A. Enter the number of years from the Existing Year to the Opening Year. Enter the difference between the Opening Year and the Existing Year: (3) – (2C).

5B. Enter the number of years from the Existing Year to the Design Year. Enter the difference between the Design Year and the Existing Year: (4) – (2C).

6. Select a growth rate. The growth rate is to be selected from the continuous range of rates shown on the worksheet. The range of rates for each category is subjective, as are the categories, themselves. Judgement must be used in selecting an appropriate rate. If the project lies within an MPO area, manually adjusted output from a travel demand forecasting model provided by the MPO may be used in place of the growth rate. A rate derived from a regression analysis of historical traffic volumes over at least a twelve year period (equivalent to three traffic survey reports – five preferred) may be used as a tool for selecting the growth rate. It is important to recognize that a high rate derived from a regression analysis, based on only a few data points may not be sustainable when projected 20 or more years into the future. The implicit growth rate based on the Design Year ADT and the Existing Year ADT should be calculated and evaluated against the rates shown. The use of model output in place of the given rates should be noted in "Comments" (Section 15) of the worksheet.

7. Enter the Opening Year Factor. This factor is calculated as follows: [(6) x (5A)] + 1. Multiply the growth rate by the difference between the Opening Year and the Existing Year, then add 1.

8. Enter the Design Year Factor. This factor is calculated as follows: [(6) x (5B)] + 1. Multiply the growth rate by the difference between the Design Year and the Existing Year, then add 1.

9. Enter the Opening Year ADT. The Opening Year ADT is obtained by multiplying the Existing ADT by the Opening Year Factor: (2A) x (7).

10. Enter the Design Year ADT. The Design Year ADT is obtained by multiplying the Existing ADT by the Design Year Factor: (2A) x (8).

11A. Enter K. The K factor is selected from the chart on the worksheet. The volume groupings shown are subjective. When count data exists, it is possible to estimate K by dividing the peak hour volume by the ADT. However, K is to reflect the 30th highest hour of the year. For a count on a given day, there is no way to know how the peak hour for that day compares to the 30th highest hour, but "true" K would almost always be higher than this estimated K.

11B. Enter the DHV. The DHV (Design Hourly Volume) is obtained by multiplying the projected Design Year ADT by the K Factor: (10) x (11A).

12. Enter the D factor. The D factor is assumed to be .55 for projects outside an MPO's boundaries and .60 for projects on or within an MPO's boundaries, except for a one-way bridge, in which case the D factor is always 1.00. The D factor, representing the directional distribution in the design hour, is used to calculate the Directional Design Hourly volume (DDHV). Like the K factor, it can also be estimated from available count data.

    The directional distribution in the ADT is entirely different from D. In the ADT, the directional split is usually close to 50/50. If known to vary significantly from 50/50/, such as between the ramps on a bridge on a roadway over a freeway, then the directional distribution should be noted in the "Comments" section of the worksheet.

13. Enter the T24 factor. T24 represents the proportion of B&C commercial vehicles in the ADT. T24 is calculated based on the Existing Year data and assumed to apply to the Design Year. Information is seldom available that warrants selecting a T24 value for the Design Year that differs from T24 as calculated from the Existing Year data. T24 is calculated as: (2B)/(2C). If no count data exists, assume T24 = .03 or obtain new count data that provides truck data.

14. Enter the TD factor. TD is the proportion of B&C commercial vehicles in the design hour. If the number of trucks in the peak hour is included in any available count data, an estimate of TD can be calculated directly. However, TD is usually close to 60 percent of the T24 value, which an acceptable approximation for use here. TD is calculated as (13) x .6.

15. The comments section may be used for noting the substitution of MPO model output for volume estimates based on growth rates, the B and C components of the truck traffic, a significant departure from the expected 50/50 split in the daily directional distribution rate, or anything else the use wishes to document.

The Design Designation is summarized at the end of the worksheet from the above information. The design values (D, T24, and TD) are commonly listed as percents rather than decimal proportions. DHV is usually shown on the plans instead of K, although to assess the reasonableness of the DHV, it is usually easier to think in terms of K.

References:

1. Pavement Design and Rehabilitation Manual, Draft, Office of Materials Management, transmitted on August 10, 1998.

2. Guidelines for Developing Design Year Traffic on Local Roads and Streets, prepared by the Bureau of Technical Services, December, 1976.

3. Procedures for ODOT District Offices for use in Developing Design Designations for Non-Interstate Bridge Replacement/ Rehabilitation Project, Bureau of Technical Services, November 17, 1993.

4. Traffic Survey Reports prepared by the Bureau/Office of Technical Services, 1975-1998 (some in preparation, older reports exist).

BRIDGE PROJECT DESIGN DESIGNATION WORKSHEET			
1A	Enter the PID:		1A
1B	Enter the County-Route-Log or other identifier:		1B
2A	Enter the Existing ADT (Total Vehicles):		2A
2B	Enter 24-hour B&C (commercial) volume if available:		2B
2C	Enter the Existing Year:		2C
3	Enter the Opening Year:		3
4	Enter the Design Year:		4
5A	Enter the number of years from the Existing Year to the Opening Year: **(3) - (2C) =**		5A
5B	Enter the number of years from the Existing Year to the Design Year: **(4) - (2C) =**		5B
6	Select a growth rate from the following range of rates: Stable .0025-.0050 Moderate .0100-.0200 Low .0050-.0100 High .0200-.0300		6
7	Enter the Opening Year Factor: **[(6) x (5A)] + 1=**		7
8	Enter the Design Year Factor: **[(6) x (5B)] + 1=**		8
9	Enter the Opening Year ADT: **(2A) x (7) =** Round to nearest 100 vehicles (nearest 10 vehicles if < 1000)		9
10	Enter the Design Year ADT: **(2A) x (8) =** Round to nearest 100 vehicles (nearest 10 vehicles if < 1000)		10
11A	Enter K, selected from the following table of Design Year ADT: < 1000 .12 5001 – 15000 .10 1001 - 5000 .11 15001 < .09		11A
11B	Enter the DHV: **(10) x (11A) =**		11B
12	Enter the D Factor (for DDHV): within an MPO area: .60 outside an MPO area: .55 any one-way bridge: 1.00		12
13	Enter the T24 factor (the proportion of B&C vehicles in ADT): **[(2B)/2A)]** or **.03** if (2B) is blank		13
14	Enter the TD factor (the proportion of B&C vehicles in the design hour): **(13) x 0.6**		14

BRIDGE PROJECT DESIGN DESIGNATION WORKSHEET

15 COMMENTS		15
DESIGN DESIGNATION (summarized from above) PID		**1A**
County-Route-Log		**1B**
Opening Year ADT =		**9**
Design Year ADT =		**10**
K =		**11A**
D =		**12**
T24 =		**13**
TD =		**14**

OHIO DEPARTMENT OF TRANSPORTATION

INTER-OFFICE COMMUNICATION

TO: **D-1 Distribution**

FROM: **Cash Misel, Assistant Director, Planning and Production Management**
Mary Ellen Kimberlin, Assistant Director, Highway Management

DATE: **April 30, 2002**

SUBJECT: **Guidelines For Identifying Acceptable Locations For The Disposal of Waste Material And Construction Debris or The Excavation of Borrow Material Within ODOT Right-of-Way**

Attached for your immediate use, are the above referenced guidelines to be used in evaluating projects for acceptable locations within ODOT Right-of-Way for the disposal of waste material or the excavation of borrow. These guidelines should be used to evaluate sites during design of a project or to evaluate contractor proposed sites after sale of the project. The guidelines use the 2001 AASHTO Design Criteria and the attached figures from the ODOT L&D Manual, Volume 1 have been revised to conform to the 2001 AASHTO Design Manual. The ODOT L&D Manual, Volume 1 will be updated to the 2001 AASHTO Design Criteria in the near future. Please share these guidelines with your staff.

Any questions should be directed to the Office of Roadway Engineering Services.

Attachment

GUIDELINES FOR IDENTIFYING ACCEPABLE LOCATIONS FOR THE DISPOSAL OF WASTE MATERIAL AND CONSTRUCTION DEBRIS OR THE EXCAVATION OF BORROW MATERIAL WITHIN ODOT RIGHT-OF-WAY

PURPOSE

This guide provides the criteria to be used when evaluating a project for acceptable locations for the disposal of waste material and construction debris or the excavation of borrow material within highway rights-of-way.

REFERENCES

1. Ohio Department of Transportation, "Location & Design Manual (LDM), Volume I & III".

2. Ohio Department of Transportation, "Construction and Materials Specifications (CMS)".

3. American Association of State Highway and Transportation Officials (AASHTO), "A Policy on Geometric Design of Highways and Streets, 2001 ".

4. FHWA Policy, "Recycled Materials Policy", dated February 7, 2002.

SCOPE

All Districts, Divisions and Offices of the Ohio Department of Transportation (OOOT) involved in the design, construction and maintenance of roadways and all consultants and contractors who provide similar services to ODOT.

BACKGROUND

The use of ODOT right-of-way for disposal of waste material and construction debris or the excavation of borrow material is now prohibited, unless locations are identified in the plans (see CMS Sections 104.03, 105.16, 105.17 and 107.11). With the increased need to remove and replace the pavements of our Interstate and Freeway System as the pavements approach or exceed their design life, the disposal of the existing pavement, much of it concrete, that cannot be recycled or used as part of the new pavement structure has become a problem. These guidelines have been developed to give designers the criteria that should be used in the evaluation of a project for acceptable waste or borrow areas within the right-of-way of a project.

DEFINITIONS

<u>Clear Zone:</u> The desirable unobstructed area along a roadway, outside the edge of pavement, available for the safe recovery of vehicles that have left the traveled way. (Section 600.2, LDM)

<u>Safety Grading</u>: The shaping of the roadside using 6:1 or flatter slopes within the clear zone area and 3:1 or flatter foreslopes and recoverable ditches extending beyond the clear zone. (Figures 307-1 and 307-2, LDM)

<u>Clear Zone Grading</u>: The shaping of the roadside using 4:1 or flatter foreslopes and traversable ditches within the clear zone area. (Figure 307-3, LDM)

Decision Sight Distance: The distance needed for a driver to detect, recognize and select an appropriate course of action for an unexpected or otherwise difficult-to-perceive condition in the roadway. (Section 201.5 and Figure 201-5, LDM)

PROJECT EVALUATION

Waste Disposal Areas

All projects with large amounts of cut and fill or projects with pavement removal, particularly non-recyclable concrete pavement, should be evaluated for acceptable disposal areas within the right-of-way. Acceptable disposal areas would preferably enhance the safety of the roadway and should not provide a less safe highway than now exists. The total width of existing right-of-way should be considered. Examples of roadway safety enhancements would include the use of safety grading where clear zone grading or less now exits, the use of clear zone grading where something less exists and the elimination of barrier. In accordance with Section 307.21 of the LDM, all interstate and interstate look alike roadways should use safety grading. If safety grading now exists, consider the possibility of extending it to the right-of-way line. If clear zone grading now exists, consider the use of safety grading or consider the possibility of extending clear zone grading to the right-of-way line. Existing barrier locations should be evaluated to see if the application of safety grading, or at a minimum clear zone grading, would eliminate the need for barrier. Adjustments to drainage or drainage structures may also be required. Not all acceptable disposal areas will enhance the safety of the roadway. Areas that do not affect the safety of the roadway (areas outside a safety graded or clear zone graded section) and do not affect wetlands or other environmental regulations but are within the right-of-way of the project should also be considered as acceptable disposal areas.

Although interchange infields seem like obvious or ideal areas to dispose of waste material, great care not to restrict sight distances is required.

Exit Ramps - Decision stopping sight distance, Avoidance Maneuver A or B, as per Figure 201-5 of the LDM should be provided for the design speed of the ramp (Figure 404-1 and Section 404.2 of the LDM). Fills may be placed in the infield areas as long as the decision stopping sight distance is provided and 6:1 or flatter slopes are provided in the gore areas (Section 307.5.3 of the LDM). Fills within the infields of diamond interchanges should not affect the intersection sight distance at the intersection of the crossroad and the exit ramp.

Entrance Ramps - Decision sight distance, Avoidance Maneuver C or E, as per Figure 201-5 of the LDM should be provided for the design speed of the ramp (Figure 404-1 and Section 404.2 of the LDM). The decision sight distance is measured from a point on the ramp where a driver on the ramp has an unobstructed view of vehicles on the mainline to a point on the ramp where the driver no longer has a lane width available on the ramp and must start to merge. This is the distance that the merging ramp driver has to decide where he can safely merge into the mainline traffic. This distance should also be unobstructed for the mainline driver to react to the ramp vehicle by either a lane or speed change.

Loop Ramps - The infields of loop ramps generally should not be filled unless it is to eliminate barrier or provide safety graded slopes. Loop ramps have a higher than average number of run off the road accidents due to the sharp curvature and high speeds. When the infields of these ramps are filled, not only are sight distances decreased but the driver also loses a sense of how sharp the curvature of the ramp is when he cannot see the entire ramp but only a small portion of it. If considered an acceptable fill site, then at a minimum, decision sight distance, Avoidance Maneuver A or B for the exit end of the ramp and Avoidance Maneuver C or E for the entrance end of the ramp, as per Figure 201-5 of the LDM should be provided for the appropriate design speed of the ramp (Figure 404-1 and Section 404.2 of the LDM).

Fill Restrictions - Fill heights greater than 10 feet should be reviewed by the Office of Geotechnical Engineering. Slopes should not exceed 4:1 for ease of maintenance. Fill material and fill construction shall be in accordance with the Construction and Materials Specifications, Item 203.

Borrow Areas

All projects requiring borrow should be evaluated for acceptable borrow areas within the right-of-way. The same criteria used to evaluate the waste disposal areas should be used to evaluate borrow areas within the right-of-way. The safety of the highway should be enhanced, if possible. Consider applying safety grading when something less than safety grading exists or clear zone grading when something less than clear zone grading exists.

The determination as to whether or not to allow the disposal of waste material or the excavation of borrow within the right-of-way of a project should be made as soon as possible in the project development process. Possible waste areas or borrow areas within the project right-of-way should be identified during the field review prior to final scope preparation so that the evaluation of these areas can be included as part of the scope for the project. If during plan development these areas are found to be acceptable as waste areas or borrow areas, then they shall be identified in the construction plan along with their limits. Acceptable locations should be identified on the schematic plan, plan and profile sheets or in a general note (see Location & Design Manual, Volume III, Section 1303.20 and Appendix B, Sample Plan Note G105). If the project has no acceptable waste areas or borrow areas within the project right-of-way, then it shall be stated on the construction plans by plan note, that an evaluation has been completed and no acceptable waste areas or borrow areas exist within the right-of-way of the project. Another consideration should be the impact of the allowed waste area or borrow area on future projects. One should not allow the placement of fill or excavation for borrow in an area that would require its removal or fill in the near future by another project. Environmental regulations, public involvement commitments, erosion control, the effects on utilities and the effects on drainage should also be considered. CMS Section 105.16 addresses erosion control and environmental regulations controlling borrow and waste areas. Coordination with utilities will be required and drainage structures may need extended or adjusted to grade.

DECISION SIGHT DISTANCE

201-5

REFERENCE SECTION **201.5**

HEIGHT OF EYE 3.50' HEIGHT OF OBJECT 2.00'

DESIGN SPEED (mph)	DECISION SIGHT DISTANCE (FT)				
	AVOIDANCE MANEUVER				
	A	B	C	D	E
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445

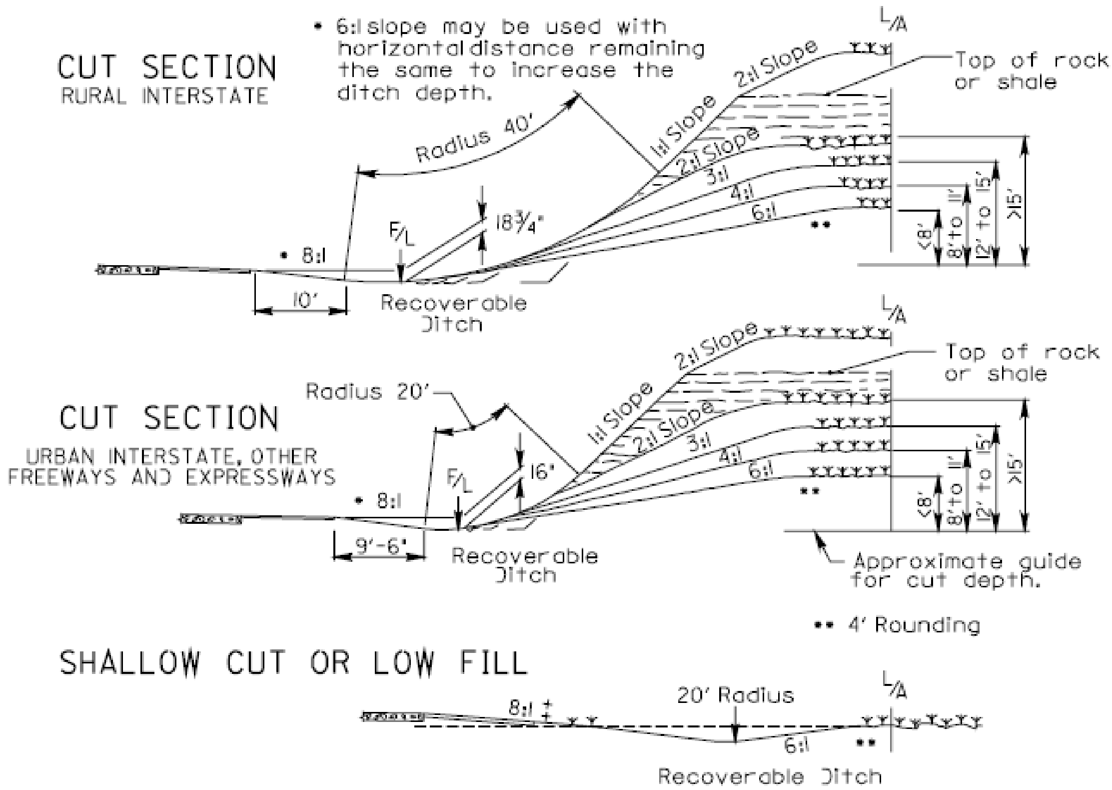
The Avoidance Maneuvers are as follows:

- A - Rural Stop
- B - Urban Stop
- C - Rural Speed/Path/Direction Change
- D - Suburban Speed/Path/Direction Change
- E - Urban Speed/Path/Direction Change

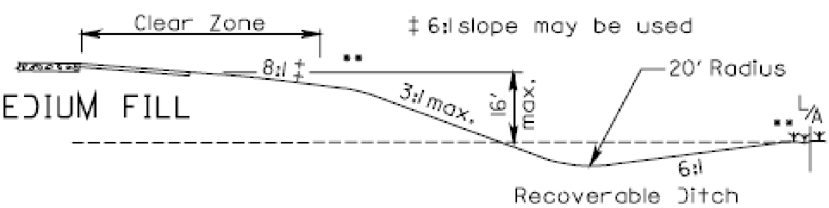
Decision Sight Distance (DSD) is calculated or measured using the same criteria as Stopping Sight Distance; 3.50 ft eye height and 2.00 ft object height.
Use the equations on Figures 203-3, 203-6 and 201-2 to determine the DSD at vertical and horizontal curves.

APRIL, 2002

SAFETY GRADING SECTIONS	307-1E
	REFERENCE SECTIONS 307.21



Slope transition between low fill design and medium fill design shall be such that the flowline of the roadside ditch does not turn toward the roadway.

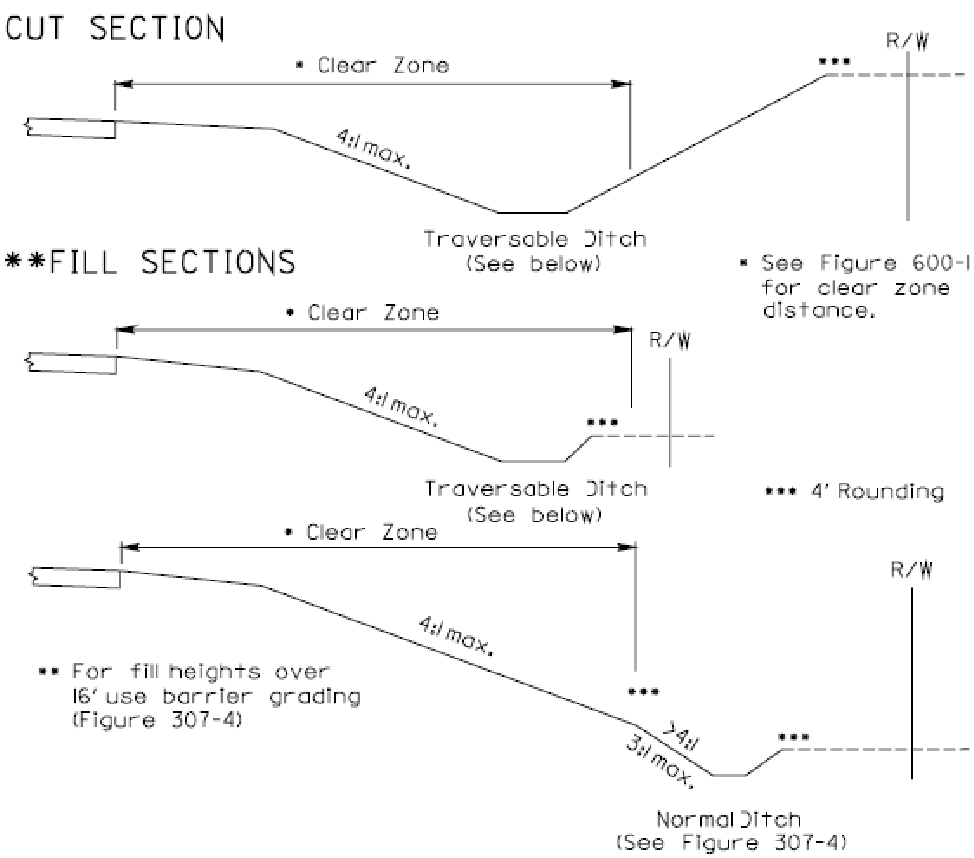


Application of these sections may vary to avoid frequent slope changes and to maintain reasonably straight ditches.

See Figure 307-2 for Recoverable Ditch details.

APRIL, 2002

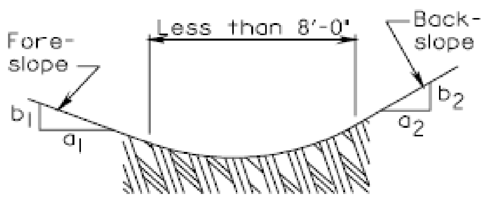
CLEAR ZONE GRADING SECTIONS (TRAVERSABLE DITCH SECTIONS)	307-3E
	REFERENCE SECTIONS 307.21, 601.1

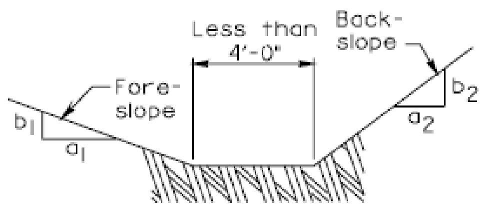


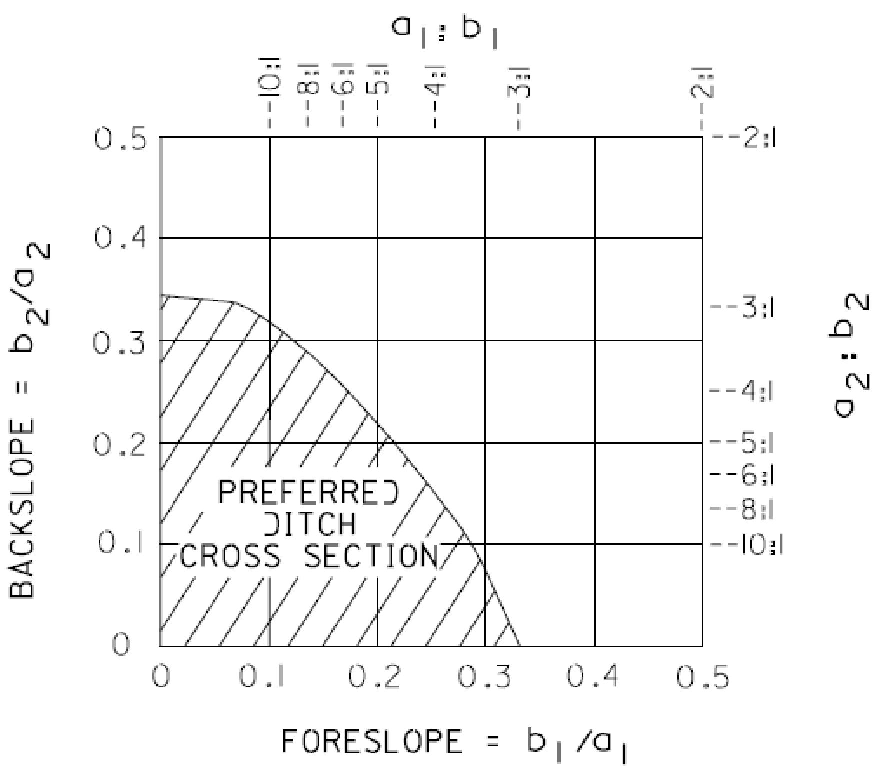
FOR ACCEPTABLE COMBINATIONS OF FORESLOPE AND BACKSLOPE FOR TRAVERSABLE DITCHES SEE FIGURES 307-10 AND 307-11.

Minimum Ditch Depth
Cut: 1.5', Fill: 1.0'

PREFERRED CROSS SECTIONS FOR DITCHES WITH ABRUPT SLOPE CHANGES	307-10E April 1999







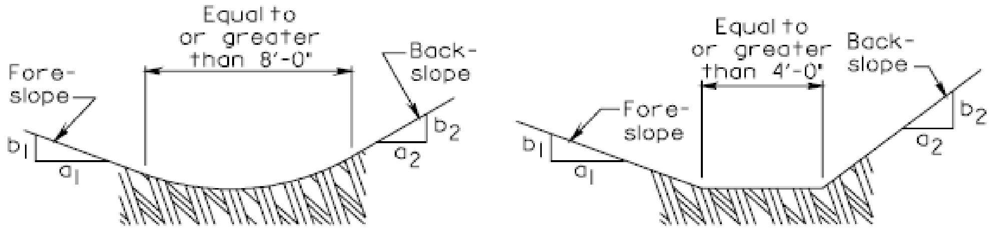
This chart is applicable to vee ditches, rounded ditches with bottom widths less than 8'-0", and trapezoidal ditches with bottom widths less than 4'-0".

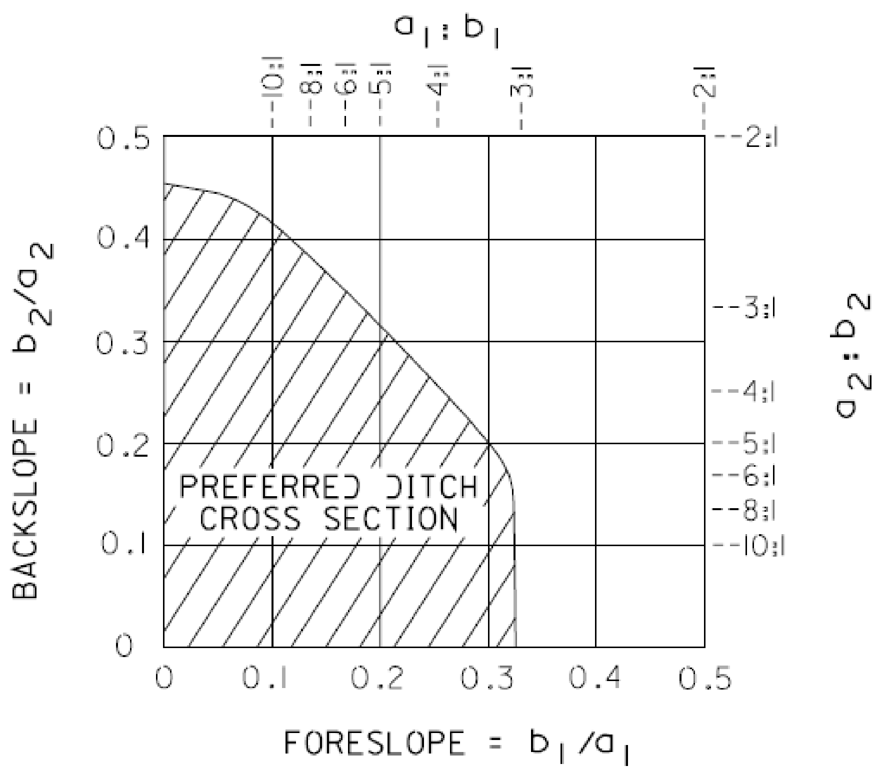
Ditches that fall within the shaded areas are considered traversable and are preferred for use within the clear zone.

PREFERRED CROSS SECTIONS FOR DITCHES WITH GRADUAL SLOPE CHANGES

307-11E

April 1999





This chart is applicable to rounded ditches with bottom widths of 8'-0" or more, and to trapezoidal ditches with bottom widths equal to or greater than 4'-0".

Ditches that fall within the shaded areas are considered traversable and are preferred for use within the clear zone.

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