



State of Wisconsin
Department of Transportation

Facilities Development Manual

ORIGINATOR Director, Bureau of Highway Development		PROCEDURE 11-30-1
CHAPTER 11	Design	
SECTION 30	Interchange	
SUBJECT 1	Design Elements	

Warranting Guidelines

The criteria to evaluate the need for a rural highway interchange are under study at this time. However, in the interim, it may be necessary to provide a gauge on when to consider construction of an interchange. A Corridors 2020 report provides a general “rule of thumb” that interchanges warrant consideration when the design year mainline and side road combined AADT > 12,000 and the side road traffic AADT > 2,000. The design process will make final determination. Other factors to consider may be economic development, safety, and potential for signals in the near future.

This is interim guidance for rural highways that may apply to four-lane divided expressways or current two-lane facilities where it may be appropriate to reconstruct a short section to four lane divided standards. This does not apply to interstate routes.

General Design

The rest of this procedure deals primarily with interchange ramps, ramp terminals, and cross road details. For information relating to the structures, refer to [Procedure 11-35-1](#).

Guidance concerning sight triangles, exit and entrance ramp tapers, ramp curvatures, and other elements of interchange design are included within this section. For additional information, refer to Chapter X, page 747, GDHS 2001.

Preliminary studies should be made for each highway to establish category of treatment, grade relationship (crossroad over or under), and layout of interchange where required. Layouts of interchanges should include turning diagrams showing design hour volumes.

Interchange Type and Selection

There are several basic interchange configurations to accommodate turning movements at a grade separation. Refer to pages 747 to 756 2001 GDHS for general interchange and grade separation warranting criteria.

The configuration used at a particular site is determined by a number of factors.

- The number or intersection legs,
- Capacity,
- Route continuity,
- Topography,



- Design controls,
- Uniformity of exit patterns,
- Single exits in advance of the separation structure,
- With or without weaving,
- Right-of-way availability
- Potential for stage construction,
- Compatibility with the environment
- Proper signing and intersection treatment at the crossroad.

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, where as the term “service interchange” applies to interchanges that connect a freeway to lesser facilities. System interchange connections should be high speed and free-flow to provide all directional movements. Connections between freeways and other controlled access facilities are analyzed and, where practical, provide all movements unstopped. Refer to pages 808 to 827, GDHS 2001 for general design considerations and specifically Exhibit 10-43 for interchanges that are adaptable on freeways as related to classifications of intersecting facilities in rural, suburban, and urban environments.

At service interchanges the crossroad should pass over the freeway or access controlled highway. There are many reasons for this; the exit ramp is constructed as an upgrade that assists vehicles to decelerate as they approach the intersection, the entrance is constructed on a downgrade that assists vehicles to accelerate as they enter the freeway.

In most cases building the cross road over the mainline offers these added benefits:

- Less earthwork required,
- Lower structure costs
- Lower maintenance costs
- Lower delay costs when the cross road structure is rehabilitated

The side road intersection is typically controlled by stop signs, roundabouts, or signals and will sometimes determine the type of interchange. Signal and roundabout control should be analyzed as equal alternatives until such time when the analysis of capacity, user delay, crashes and available space dictates which treatment is more appropriate for the location. For more information on at-grade intersections refer to [Procedure 11-25-1](#).

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. Furthermore, from the standpoint of driver expectancy, it is desirable that all interchanges have one point of exit located in advance of the crossroad wherever practical. Urban freeway design is more challenging than rural and should:

1. Maintain a basic number of continuous lanes,
2. Provide lane balance at ramp exits and entrances,
3. Provide appropriate ramp spacing, one mile or greater between the gore areas,



4. Maintain route continuity,
5. Use auxiliary lanes as appropriate.

Route continuity and system configuration may also drive the interchange type. Typical urban freeway operational problems that result in high crash locations include; weave sections, limiting geometry (horizontal and vertical), and left side ramps. Left side entrance and exits are not acceptable. A FHWA publication [1] suggests that crashes may be reduced 25-70 percent with the use of right-off, right-on ramps as compared to left side ramps.

Signing and operations are major considerations in the design of the interchanges. The need to simplify interchange design from the standpoint of signing and driver understanding is critical.

To prevent wrong-way movements, confusion, driver frustration and misdirection, all freeway interchanges with non-access-controlled highways should provide ramps to serve all basic directions. When drivers exit the freeway system they should be able to enter again at the same interchange or within a short distance on a frontage road. Do not design at-grade braided ramps at any interchanges. Remove existing at-grade braided ramps¹ on reconstruction projects and relocate the frontage road separate from the ramp.

Provide direct entrance and exit ramps without driveway, side road or other access to ramps. Access within interchange ramps or access within close proximity to ramps on expressways is counter to driver expectancy. See [Procedure 11-5-5, Figure 3](#) for additional information on nearest access from ramp entrance and exit as well as nearest crossroad access outside the ramp/crossroad termini.

The accommodation of pedestrians and bicyclists also should be considered in the selection of an interchange configuration.

The most favored interchange type for Department applications is the conventional diamond interchange. The more common interchange configurations are addressed in the following text. For additional information on interchanges see pages 774-827, GDHS 2001.

Diamond Interchanges

Diamond interchanges are the simplest, generally the least expensive and the most common type of interchange. The conventional and the tight diamond are the preferred WisDOT interchange type for most situations. The more common types of diamond interchanges are:

- (a) Conventional diamond, used in rural conditions or urban conditions where space allows; See GDHS 2001Exhibit 10-16a, page 783
- (b) Tight diamond², primarily used in urban areas where space is limited;
- (c) Split diamond, used where local conditions justify separating the ramps by using one way frontage road pairs; See GHS 2001Exhibit 10-17a.
- (d) Single-point urban diamond (SPUI), used where all four turning movements are controlled by a single traffic signal and opposing left turns operate to the left of each other, and again where space is limited; See GDHS 2001 Exhibit 10-23. The single-point is quite costly to construct. Single-point diamond interchanges should be

¹ Braided ramps are entrance or exit ramps that connect a controlled access highway to a nearby parallel frontage road.

² A tight diamond is a diamond interchange whose ramps are pulled in closer by using retaining walls.



designed such that the mainline passes under the at-grade-intersection. There are two primary reasons for this: (1) by constructing the mainline at the lower level columns may be located in the center of the structure thus reducing the clear span of the structure and substantially reducing girder depth, earth work and the cost, (2) The at-grade-intersection should be located on the top level where it is exposed to an even lighted surface, thus not requiring the driver to go from sunlight into shade and back into sunlight. The eye needs time to adjust to changing light conditions. Turning on a curved path through the intersection with a changing light condition may be unsafe and problematic particularly for older drivers. There are multilevel and other diamond interchange configurations that may be appropriate for certain situations that are explained further in the pages 782 to 791, GDHS 2001.

The capacity of a diamond interchange is limited by the capacity of the at-grade terminals of the ramps at the crossroad. The entrance ramp capacity may be limited by the traffic volume on the mainline. Traffic operations may be improved and interchange life expectancy extended by using a roundabout at the ramp/crossroad termini. The approximate limit of left turning traffic to any intersection approach is with double left turn lanes and that is limited to approximately 600 left turning vehicles for the DHV. Left turn demand greater than 600 DHV should consider a roundabout or directional ramp design to reduce user delay and crashes. A roundabout will usually break up traffic platoons and allow ramp entrance traffic to enter the system in a dispersed manner which may preclude the need for ramp metering. Proper analysis and evaluation of the projected intersection traffic volumes and turning movements will guide the designer to the correct intersection treatment (i.e. stop, roundabout, or signal control).

Three-Leg Designs

An interchange with three legs consists of one or more highway grade separations and one-way roadways for all traffic movements. They are sometimes referred to as “T” and “Y” interchanges. The more common types of three-leg interchanges are:

- (a) Trumpet –A. (page 776, GDHS 2001)
- (b) Trumpet - B. The more direct alignment favoring the heavier volume of the left turn movements and the tight loop favoring the lesser volume.
- (c) Directional-Y (page 777, GDHS 2001). Direct alignment is provided to both left turn movements where volume is anticipated to be high. A review of capacity of tight loop ramps is presented in the Four-Leg Design section.

Four-Leg Designs

Interchanges with loops in all four quadrants are referred to as “full cloverleafs” and all others with loops in one or more quadrants are referred to as “partial cloverleafs” or “par-clo.” The common types of cloverleafs are:

- (a) Full cloverleaf without collector-distributor (**C-D**) road, requires substantial space, may reduce capacity on high volume highways when collector-distributor roads are not used (page 783, GDHS 2001).
- (b) Full cloverleaf with C-D road requires substantial space. The C-D road should be considered when the sum of the traffic on two adjoining cloverleaf loops approaches 1,000 vph. C-D roads allow weaving and slower moving loop ramp traffic to adjust and select the appropriate lane while still separated from the mainline traffic (page 793, GDHS 2001).



- (c) Par-clo is commonly used where the right-of-way is restricted (e.g., with loops on one side of the mainline because of a stream or railroad). For other par-clo configurations see page 795 GDHS 2001.
- (d) Semi-directional interchange, with loops and ramp to accommodate high-volume left turn traffic in one direction (page 799, GDHS 2001).
- (e) Directional interchange, allows for all high speed direct movements from one facility to another. There are many other three and four-leg interchange designs that are covered further in the pages 776 to 827, GDHS 2001.

The volume on a tight loop ramp (25-30 mph design speed) is limited to approximately 12,000 ADT or about 1,200 DHV. A loop ramp should be restricted to one lane unless there is ramp metering or other extenuating circumstances. The entrance to loop ramps should be designed with consistent radii, without compound curves entering the loop from a high-speed condition. Compound curve design is acceptable when leaving the loop and entering the acceleration lane. Two-lane tight loop ramps may cause serious operational problems and are not permitted. In lieu of designing a loop ramp with design year volume greater than 12,000 ADT, consider a directional ramp or roundabout at the termini.

For more assistance contact your BHD project development engineer.

Ramps

The relationship of speed to curvature for ramps is shown in [Figures 2 and 3](#), "Details for Entrance Terminals at Interchanges" and "Details for Exit Terminals at Interchanges.", respectively. The values shown for curve radius and curve length were derived from Exhibit 10-70, page 851 and Exhibit 10 73, page 855, GDHS 2001, and are adequate for main line design speeds through 75 mph (**120 km/h**).

Details for cross road designs, assuming a stopped condition, are shown in [Figures 4, 5 and 6](#).

On diamond type interchanges the length of ramp provided between terminal and gores should be a minimum of 900 feet (**275 m**), up to a maximum of 1,200 feet (**365 m**).

The distance between ramp terminals as measured along the cross road (rural) should be a minimum of 300 feet (**90 m**), up to a maximum of 500 feet (**150 m**), with no ramp terminal being closer than 100 feet (**30 m**) from a structure.

Speed Change Lanes

Speed change lanes (acceleration and deceleration) are needed at both entrance and exit terminals to the main line roadways of interchanges. These lanes should be of sufficient length to allow a driver to make the necessary change between the speed on the highway and the lower speed on the turning roadway or ramp.

Parallel-type entrance ramps are recommended for new interchange construction or for the reconstruction or reconfiguring of existing interchanges. Merge tapers at the downstream end of parallel-type entrance ramps are to have a minimum taper rate of 30:1.

Parallel-type ramps are not to be confused with the auxiliary lanes, which serve a different function than parallel-type ramps. Refer to [Procedure 11-25-35](#) and pages 818-821, 2001 GDHS.

Except in special situations, use the details shown in [Figures 1 and 2](#) of this procedure to design single-lane parallel-type entrance and taper-type exit terminals to the main line roadway. Special situations include, but are not limited to, multi-lane entrance or exit



ramps, and overlapping entrance and exit tapers (as in a full cloverleaf interchange). Refer to pages 860-867, 2001 GDHS, for guidance in special designs.

Right Hand Ramps vs. Left Hand Ramps

Use right hand entrance and exit ramps in the design of new interchanges. Left hand entrance and exit ramps are contrary to driver expectation (refer to page 845, GDHS 2001). An FHWA publication [1] suggests that crashes may be reduced as much as 25-70 percent with the use of right-off, right-on ramps as compared to left hand ramps.

If possible, replace existing left hand entrance/exit ramps with right hand ramps when reconstructing an interchange. If this is impracticable because of unacceptable economic, agricultural, wetland or historical impacts then document and justify this in the Design Study Report. This justification shall include a crash data analysis showing that the existing left hand ramp is not a safety hazard.

Major forks or major freeway splits are not considered ramps and therefore may diverge left or right. Examples of major splits/forks that enter or exit on the left hand side are IH 94 EB/IH 90 EB near Tomah, and IH 894 WB/IH 43 in Milwaukee.

Parallel-Type Entrance Ramps vs. Taper-Type Entrance Ramps

When designing new interchanges or reconstructing interchanges. Use parallel-type entrance ramps in rural or urban applications. Refer to Exhibits 10-69/70/71, pages 849-853, GDHS 2001 for more information.

Taper-type entrance ramps have been used predominantly for interchange design in the past. However, studies [2] have shown that parallel entrance ramps are generally safer than tapered. With tapered entrance the driver has less time and poorer angles in which to use side/rear-view mirrors to monitor surrounding traffic prior to merging. Taper-type entrance ramps can also cause confusion in mainline horizontal curve situations when the driver cannot identify mainline alignment. See [Figure 1](#) for parallel-type entrance ramp design.

Desirably, a curve with a radius of 1000 ft (**300 m**) or more and a length of 200 ft (**60 m**) should be provided in advance of the parallel ramp. If the approach curve is short drivers tend to drive directly onto the mainline without using the acceleration lane, which is undesirable.

As the parallel ramp becomes longer the driver's perception is that they are in a continuing lane. When the parallel entrance ramp design exceeds 800 ft, (**245 m**) pavement marking, arrows, may be necessary to reinforce the lane drop. Consult the District Traffic Section for the need, placement and spacing of the Type 5 arrows.

If it is necessary to provide an acceleration lane longer than computed by applying the adjustment factors in Figure 1 then consult with the district Traffic Section about the use of auxiliary lanes or lane additions.

The length (L) of the parallel entrance ramp is measured from the point where the left edge of the traveled way of the ramp pavement is 3 ft from the right edge of the traveled way of the mainline, refer to Figure 1 and SDD 13C17, to the point where the downstream taper begins. The length (L) is given in Figure 1 for the ramp according to the entering speed (design speed of ramp)³, and the design speed of the mainline. Refer to Chapter 10, pages 849-853, GDHS 2001. Provide a down stream merge taper of 360 ft (**110 m**) in all applications of parallel entrance ramps.

³ The design speed of a ramp is based on the most restrictive curve radius leading into the parallel portion of the ramp. See values for R₁, R₂ and R₃ in Figure 3



Review the DHV traffic on the mainline. If it is anticipated that the freeway will frequently approach capacity or the percent of trucks on the ramp exceeds 10%, then a minimum length (L) of 1200 ft plus the 360 ft merge taper is desirable.

The grade of the mainline parallel ramp area will have an affect on the length of the acceleration lane needed to achieve merging speed. Figure 1 gives multipliers to be used where the grades are + 2.2% to +4% and when the ramp truck volume exceeds 10%.

Consider the following factors when designing parallel entrance ramp terminals.

1. The entering speed of the vehicle - directly related to the radius and design speed of the last curve on the ramp.
2. Type of mainline roadway (freeway, expressway, interstate)
3. Design speed of the main line.
4. Percent trucks using the ramp.
5. The grade of the main line.

Example 1

Given: A freeway with a design speed of 70 mph,

A parallel on-ramp with approach curves $R_1=1095$ ft and R_2 & R_3 = tangent

Mainline grade is 0.5%,

No capacity problems on either the ramp or the mainline.

Find: The length (L) of the parallel entrance ramp.

From the table in Figure 3, this combination of R_1 , R_2 and R_3 will permit a ramp design speed of 55 mph.

From the table in Figure 1 this combination of ramp and mainline design speeds requires a ramp length of 600 ft (**185 m**). To this must be added the 360 ft (**110 m**) merge taper.

Example 2

Given: Same situation as Example 1 except now the mainline grade is +3%.

Find: The length (L) of the parallel entrance ramp.

Start with the same basic length as determined in Example 1 but now apply the adjustment factor for mainline upgrades from +2.2% to +4%. In this case the factor is 1.85.

$600 \times 1.85 = 1110$ ft (**338 m**) To this must be added the 360 ft (**110 m**) merge taper.

With a length of 1110 ft, consult with the district traffic section about the need, placement and number of any Type 5 arrows needed to reinforce the lane drop.

Ramp Speeds

Ramp design speed will vary by type of interchange design. This section also addresses a freeway split which by definition is not a ramp. The various scenarios are provided as follows:

1. Freeway Splits
2. Freeway-to-Freeway directional ramp
3. Freeway-to-Service Road off and on ramps

Provide the highest practicable design speed for the ramps as well as attempt to reach the AASHTO suggested upper ranges for the given situation.



Exhibit 10-56, page 830, AASHTO 2001, provides guide values for ramp design speed as related to various mainline highway design speeds (i.e. those in the upper range (85%) of the mainline ramp terminal should be designed to be within 85% of the mainline design speed, middle range within 70%, and lower range within 50%). Guidance is provided for each scenario, including the freeway split.

1) Freeway Splits – Each freeway shall have the same design speed. Freeway split example: Tomah Interchange where I-90 continues westward and I-94 diverges northward.

2) Freeway to Freeway directional ramps – Design directional ramps to be in the upper range, or within 85% of the mainline design speed (within 10mph of mainline highway design speed for 60mph and greater). Freeway to freeway directional ramp example: Portage Interchange where I-39 South connects to I-94 Eastbound.

3) Freeway to Service Road off and on ramps – Design diamond interchanges to be in the upper range, or within 85% of the mainline design speed (within 10 mph of mainline highway design speed for 60mph and greater). Design Tight loop ramps (cloverleaf or partial cloverleaf) in the lower range, or within 50% of the mainline design speed. The minimum design speed on ramps or turning roadways associated with interchanges is normally 30 mph (**50 km/h**). A minimum design speed of 25 mph (**40 km/h**) may be used on loop ramps when the mainline design speed is 50 mph (**80 km/h**) or less. Because of the increased lengths and large areas required, the maximum design speed on loop ramps should be limited to 30 mph (**50 km/h**). Provide proper deceleration distance to transition from the mainline design speed to the ramp design speed while maintaining proper stopper sight distance.

The design speed of ramps approaching their junction with crossroads should be adjusted to fit the conditions existing or desired at these terminals. The details for a stop condition at a crossroad are shown in [Figures 5 and 6](#). For cross road terminal treatments that merge with an arterial, such as at a cloverleaf where the design speed on the arterial is less than 65 mph (**110 km/h**), the length of the taper should be provided in accordance with Exhibit 10-70, page 851, or Exhibit 10-73, page 855, GDHS 2001.

Ramp Alignment

Ramp alignment details and transition lengths for curves on entrance and exit terminals are shown in [Figures 1-3](#). Exit ramp tapers should be located along tangent sections of the main line. Ramp tapers that are located adjacent to a main line section that is on a curve are undesirable because of the confusion created as to which alignment is the main line. Ramp terminals on a curve are addressed on pages 856-860, GDHS 2001. Transition curvature of turning roadways on ramps should be designed with adequate length to facilitate any speed change to the design speed of the succeeding curve. Lengths of transition are also shown in Exhibit 3-45, page 204, GDHS 201.

Ramp Sight Distance

For minimum stopping sight distance along ramps or the turning roadway, refer to Exhibit 3-1, page 112, GDHS 2001. Entrance ramps and merging areas should be visible to approaching main line traffic for a minimum distance equivalent to the design stopping sight distance. The design of the ramp and merging area should take into account the effect of grades, especially where there is a substantial volume of heavy truck traffic (see Exhibit 10-71, page 852, GDHS 2001). Exit ramp tapers should diverge from the main line roadway in such a way that the vertical curvature will not restrict visibility along the ramp to a value less than the stopping sight distance for the ramp design speed. Ramps that "drop out of sight" create a definite problem in driver recognition of queuing and should be avoided. At least



200 feet (**60 m**) of ramp pavement beyond the gore should be visible from the main line at the point where the exit taper begins.

Intersection Sight Distance

The sight distance at a ramp terminal must be adequate to allow safe turning movements. For ramp terminals that merge with a crossroad, such as at a cloverleaf, the sight distance requirements for the main line apply except that the design speed may be lower (refer to "Ramp Speeds" in this procedure).

Grades and Profile

The effect of grades on the length of speed change lanes is shown in [Figures 1 and 2](#). Grades on ramps should not exceed five percent. The maximum acceptable grade is eight percent provided the length of such grade is relatively short.

Profiles for ramps usually consist of a combination of crest and sag vertical curves. The vertical curves should be designed simultaneously with the horizontal alignment to avoid hidden curves for a driver leaving the through highway and turning onto a ramp.

Superelevation and Cross Slope

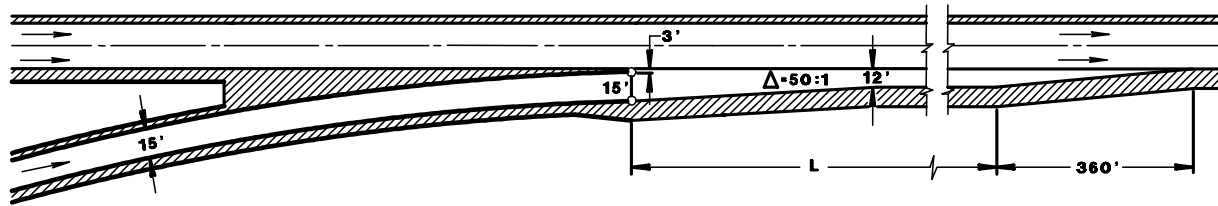
The maximum superelevation rate for ramps and ramp terminals is 6 percent. The one exception is existing ramp roadways with cross slopes based on an 8 percent maximum superelevation. In these cases the existing 8 percent rate may be retained when such roadways are resurfaced or rehabilitated. Guidance on development of superelevation at turning roadways and terminals is included in GDHS, 2001 Chapter 9, Intersections, Superelevation For Curves At Intersections and, Chapter 10, Grade Separations and Interchanges, Ramps.

The maximum algebraic difference in pavement cross slope at turning roadways should not exceed 5 percent. Where necessary, the divergence angle between the ramp and through lanes should be increased to limit the roll-over rate to 5 percent.

References

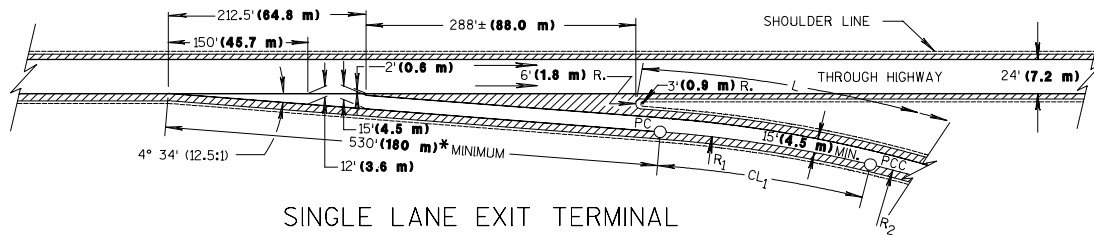
- [1] US Department of Transportation, Federal Highway Administration. Safety Effectiveness of Highway Design Features, Volumes 1 to VI. Washington DC, 1992. Volume IV specifically discusses interchanges.
- [2] Transportation Research Record 1385, Ramp Exit/Entrance Design

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**PARALLEL ENTRANCE RAMP**

Entering Speed (Design Speed) of Ramp (mph)	Main Line Design Speed		Adjustment Factor for Upgrade on Mainline
	70 mph	55 mph to 65 mph	
	L feet (m)	L feet (m)	+2.2% to + 4% *
30	1350 (410 m)	1120 (340 m)	1.60
35	1230 (375 m)	1000 (305 m)	1.65
40	1000 (305 m)	800 (245 m)	1.70
45	820 (250 m)	600 (185 m)	1.75
50	600 (185 m)	600 (185 m)	1.80
55	600 (185 m)	600 (185 m)	1.85

* For mainline upgrades less than +2.2 %, no adjustment is necessary. For mainline grades exceeding + 4%, see Exhibit 10-71 on page 852 of GDHS 2001. Also discuss this with your Project Development Engineer in the BHD Design Services Section.



Ramp Design Speed PC	Radius		Ramp Design Speed PCC	Curve Length
	R ₁	R ₂		CL ₁
55 mph (90 km/h)	Tangent Alignment			
50 mph (80 km/h)	850' (250 m)	510' (135 m)	40 mph (60 km/h)	150' (50 m)
45 mph (70 km/h)	660' (195 m)	380' (135 m)	35 mph (60 km/h)	150' (50 m)
40 mph (60 km/h)	510' (135 m)	273' (90 m)	30 mph (50 km/h)	150' (50 m)

L = 900' (280 m)

LEGEND



Paved Shoulder

L

Minimum ramp distance from gore to the intersection of the ramp with the crossroad.

R₂

Radius of the major internal segment of the loop.

* This metric length is based on a mainline design speed of 68 MPH (110 km/h) and a minimum ramp design speed at PC of 37 mph (60 km/h)

NOTES:

The length of the deceleration lane is based on ramp grades of 0 to 2%. Refer to Exhibit 10-73 page 855 GDHS 2001, for length adjustment factors to be used when ramp grades exceed $\pm 2\%$.

If the ramp speed and radii relationships listed in the table cannot be attained due to area R/W restrictions, consideration should be given to collector-distributor roads. This permits further speed reductions before entering the ramp loop.

The radii of the horizontal curves are rounded and based on a maximum superelevation rate of 6% and the speeds shown.

Ramp geometrics are adequate for mainline design speeds through 65 MPH.

Minimum Shoulder Treatments

Mainline

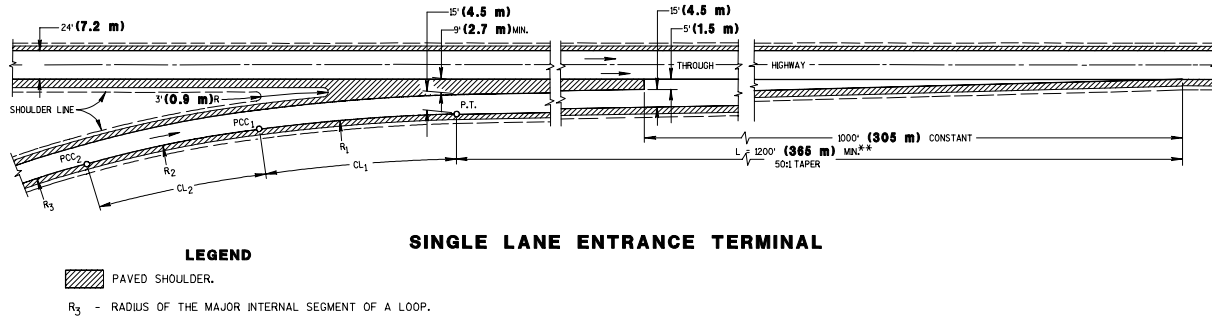
Left - 6'(1.8 m) total / 3'(0.9m) paved [4'(1.2 m) for interstate]

Right- 10'(3.0 m) total / 8'(2.4 m) paved

Ramps

Left - 4'(1.2 m) total / 3'(0.9 m) paved

Right - 8'(2.4 m) total / 5'(1.5 m) paved



SINGLE LANE ENTRANCE TERMINAL

Ramp Design Speed PCC ₂		Radius			Curve Length	
(mph)	(km/h)	R ₃	R ₂	R ₁	CL ₂	CL ₁
55	90	TANGENT ALIGNMENT				MIN. 200' (60 m)
50	80					MIN. 200' (60 m)
45	70	660' (195 m)	660' (195 m)	850' (260 m)		150' (45 m)
40	60	510' (135 m)		850' (260 m)		150' (45 m)
35	60	380' (135 m)		850' (260 m)	150' (45 m)	150' (45 m)
30	50	273' (90 m)		850' (260 m)	150' (45 m)	150' (45 m)
		*250' (84 m)	* Minimum Desirable			

Minimum Shoulder Treatments

Mainline

Left - 6' (1.8 m) total / 3' (0.9 m) paved [4' (1.2 m) for interstate]

Right - 10' (3.0 m) total / 8' (2.4 m) paved

Ramps

Left - 4' (1.2 m) total / 3' (0.9 m) paved

Right - 8' (2.4 m) total / 5' (1.5 m) paved

NOTES:

Ramp design speeds at PCC₂ are based on governing radii R₃ or R₁. Assuming SE = 6% refer to Exhibit 3-14 page 145, GDHS 2001.

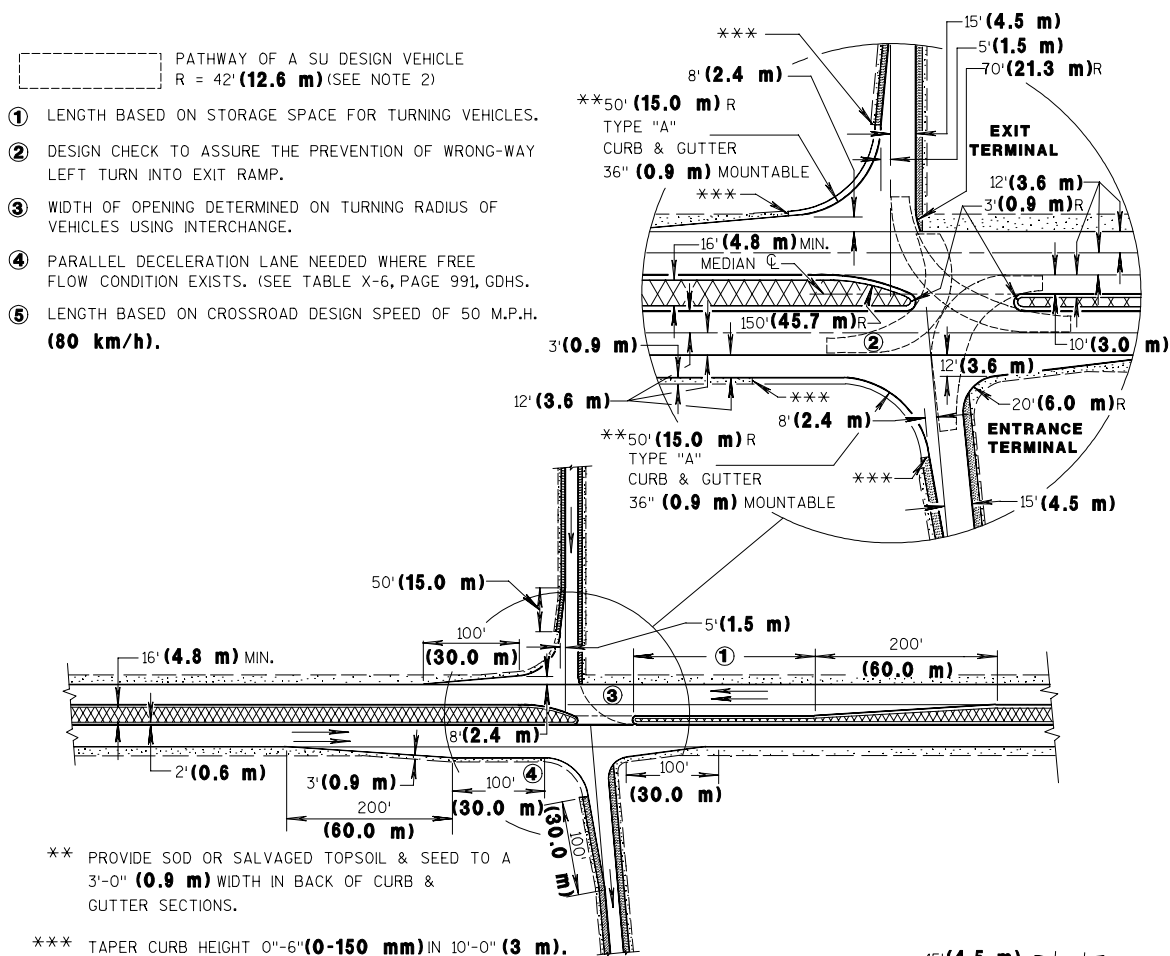
Minimum acceleration lane, taper length (L), based on Exhibit 10-70, page 851 GDHS 2001.

For acceleration lanes having grades in excess of ±2% refer to Exhibit 10-71 page 852, GDHS 2001, for length adjustment.

**When design speed at PCC₂ is 40 MPH (60 km/h) or less, adjust acceleration length (L) as follows: 1250' (380 m) (40 MPH) (60 km/h), 1300' (395 m) (35 MPH) (60 km/h), and 1400' (425 m) (30 MPH) (50 km/h).

Ramp geometrics are adequate for mainline design speeds through 65 MPH. (110 km/h).





PLAN VIEW

NOTES:

USE THIS DETAIL FOR FREE FLOW CONDITIONS UP TO 25 M.P.H. (40 km/h).

SEE TABLE III-17, PAGE 197, GDHS FOR MINIMUM RADIUS (R) NECESSARY FOR DESIRED TURNING SPEED.

