Minnesota Department of Transportation
Engineering Services Division

Transmittal No. 22-03
December 7, 2022

Road Design Manual

Distribution: **Electronic Distribution Recipients**

Subject: Chapters 2, 5, and 6

MnDOT is transitioning from the Road Design Manual (RDM) to its successor document of road design guidance, the Facility Design Guide (FDG). During this transition, both publications will be active and on line.

As new sections of the FDG are published, the corresponding RDM material will be removed and linked references to the new material inserted in their place. If you are not already on the electronic mailing list for these documents, signing up here will alert you to new updates as they occur.

Our apologies for any inconvenience. Thank you for bearing with us as we renew our road and street design guidance.

Summary of Changes:

1. Content in chapter 2 is being removed to correspond with the publishing of FDG Chapter 3.
2. Content in chapter 3 section 3.01.05 contains outdate research data and is being removed for clarity.
3. Content in chapter 6 section 3.04.01 has been modified. Language adjusts the ramp minimum radius recommendations to be consistent with Standard Plan 5-297.106 and RDM guidance prior to the May 2019 update, which was based on 6% superelevation. Designs based on 8% superelevation are now allowed, but reserved for constrained situations.

Instructions:

1. Record this transmittal letter number, date and subject on the transmittal record sheet located in the front of the manual.
2. Remove from the manual: Pages 2-1(1) through 2-3(16), 2-5(1) through 2-5(4)
   Pages 5-3(6 - 7) and 6-3(4 – 5)
3. Insert into the manual: Pages 2-1(1) and 2-5(1)
   Pages 5-3(6 - 7) and 6-3(4 – 5)

4. The Road Design Manual and associated Transmittal Letters are available online in PDF format at. http://roaddesign.dot.state.mn.us/roaddesign.aspx

5. Any technical questions regarding this transmittal should be directed to Khamsai Yang, Design Standards Engineer, at (651) 366-4708, or by email to DesignStandards.DOT@state.mn.us

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CHAPTER 2
HIGHWAY DESIGN STANDARDS

THE INFORMATION FOR THIS SECTION HAS BEEN INCLUDED IN THE MnDOT FACILITY DESIGN GUIDE. REFER TO THE FOLLOWING WEBSITE:

https://roaddesign.dot.state.mn.us/facilitydesign.aspx

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2-5.0 DESIGN PARAMETERS

2-5.01 Functional Classification

2-5.02 Investment Categories

2-5.03 Types of Highways

2-5.04 Interregional Corridors

THE INFORMATION FOR THIS SECTION HAS BEEN INCLUDED IN THE MnDOT FACILITY DESIGN GUIDE. REFER TO THE FOLLOWING WEBSITE:

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2-5.05 Traffic Characteristics

Traffic operational characteristics have a large impact on the applicable geometric and structural design criteria of a highway. The designer must provide a facility that will reasonably accommodate all anticipated traffic characteristics in the selected design period, (i.e., 20 years from the completion date of the project). The designer should refer to the current Transportation Research Board (TRB) "Highway Capacity Manual" (HCM) for a more detailed description of the operational factors of highways. The MnDOT Traffic Engineering Manual also contains information concerning how to make field measurements of traffic data and how to interpret and apply the data. For bicycle traffic characteristics, see the planning section of the MnDOT Bikeway Facility Design Manual.

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5-3.01.04 Exceptions to Turn Lane Lengths

Ideally the full deceleration length (including taper) and the storage length should be provided. However, conditions may justify a shorter turn lane. Such conditions include:

1. Urban facilities with closely spaced intersections and considerable roadside development, where full length turn lanes are physically unfeasible.
2. Low volume roads, where a full length turn lane is not needed or economically unfeasible to construct.

In such cases, designers may assume that most of the deceleration will be performed in the through-lane. This is a reasonable assumption that does not sacrifice safety.

If the first condition listed above is the one influencing turn lane lengths, designers are encouraged to consider alternative measures such as restricting turning movements, adjusting the signal cycle (done by the District Traffic Engineer), providing dual left-turn lanes (warranted at turning volumes greater than 300 VPH), or providing continuous one-way or two-way left-turn lanes.

When shorter turn lanes are being considered, the following factors should be taken into account:

1. Through-traffic volumes,
2. Traffic composition,
3. Future plans for signalizing a presently unsignalized intersection, and
4. Future land use changes which may alter traffic and turning volumes.

In contrast, it may be justified to provide a turn lane length greater than the typical lengths. Criteria for consideration include: traffic generated by adjacent land use, types of vehicles, deceleration, downgrades, and storage needs. At some sign or signal controlled intersections, heavy through-traffic blocks the entrance of the turning lane. In such cases, designers are encouraged to extend the length of the turning lane to allow turning vehicles to get into the turning lane without having to drive on the shoulder.

Before modifying the length of a turn lane, the District Traffic Engineer should review the specific location and the effect on traffic operation.

5-3.01.05 Continuous One-Way or Two-Way Left Turn Lanes (COWLTL; CTWLTL)

This section has been removed. Contact MnDOT’s Geometric Design Unit for guidance.
https://www.dot.state.mn.us/design/geometric/index.html

5-3.01.06 Double Left-Turn Lanes

Double left-turn lanes should be considered when the turning volume equals or exceeds 300 vehicles per hour (VPH), when the requirements for storage makes the turn lane extremely long, or when geometrically a required length for the single left-turn lane cannot be provided. Double left-turn lanes operate at approximately 1.8 times the capacity of a single left-turn lane. Because of high volumes associated with double left-turn lanes, fully protected signal phasing is required.

The following design guidelines for double left-turn lanes are from NCHRP 279, Intersection Channelization Design Guide:

1. The throat width for turning traffic is the most important design element. Drivers are most comfortable with extra space between the turning queues of traffic. Because of the off tracking characteristics of vehicles and the relative difficulty of two abreast turns, a 36 ft throat width is desirable for two lanes of turning traffic. In very constrained situations, a 30 ft throat width is an acceptable minimum.
2. Designers should check for possible conflicts involving left-turns opposing double left-turns. For proper design, use the swept path of semi-trailer and a 14 ft strip placed alongside on the inside of the turn for a passenger vehicle. Turn templates should be used to check the design.
3. Consideration should be given to providing pavement markings to separate the turn lanes. The MN MUTCD recommends 2 ft long dashed lines with 4 ft gaps to channel turning traffic. These channelization lines should be carefully laid out to reflect off-tracking and driving characteristics.

5-3.01.07 Continuous Right-Turn lane (CRTL)

A continuous right-turn lane is essentially a combination of right-turn acceleration and deceleration lanes that are extended to accommodate several closely spaced driveways. For proper operation, continuous right-turn lanes should not be longer than 0.25 miles. Continuous right-turn lanes are desirable where speeds are greater than 30 mph, the roadway volumes heavy, and turning demands high.

1. The use of continuous turn lanes may reduce the frequency and severity of rear-end conflicts by removing turning vehicles from higher speed through lanes.
2. The need and application should be site specific and should be based on analysis of rear-end accidents, turn volumes and operation.
3. The design of a typical section and tapers should be the same as that for a typical right-turn lane.

5-3.02 Channelization

Large all-paved intersections may cause the driver to make conflicting movements. Channelization separates and clearly defines points of conflict within the intersection. Channelized islands should be placed so that the proper course of travel is immediately obvious, easy to follow, and of unquestionable continuity. This can be achieved with painted, flush, or curbed islands. The islands also provide pedestrian refuge and a location for traffic control devices. The following is general guidance for channelized intersections:

1. There is a practical limit to how much channelization is appropriate for a given intersection before driver confusion results.
2. Curbed islands should only be used on multi-lane highways and the more important 2-lane facilities.
3. Curbed islands whose area is less than 50 sq ft should not be constructed. The minimum width and length of median islands is 4 ft and 25 ft respectively. Where signs or ramp meters are to be used, the minimum width should be 8 ft.

6-3.04 Design Elements

6-3.04.01 Design Speed

The design speed of the ramp proper should conform to the expectations of drivers and fit the constraints and topography of each location. In practice this involves designing the curve adjoining the mainline terminal to a certain percentage of the mainline design speed, depending on context, degree of constraint, and construction cost. The other portions of the ramp are designed based loosely on an assumed speed profile along its length. For ramps that terminate at an intersection, uniform deceleration to a stop condition is usually appropriate. Direct and outer connections are most often designed for a constant speed. For semi-direct connections, the portion between the mainline terminals is usually designed to a somewhat lower speed than the terminal curves, typically dictated by site specifics and interchange configuration.

For a given mainline design speed, Table 6-3.04A gives the corresponding ranges of ramp design speed and associated recommended radii, applicable to the first/last curve adjoining the mainline terminal (but not the transitional curves to/from the main curve). As noted, these recommended radii are based on a superelevation rate of 0.060 (ft/ft), which conforms to historical practice. Applying these values as a typical design approach maintains consistency among the majority of interchanges on the highway system. For situations where additional flexibility is needed, minimum radii based on the maximum superelevation rate of 0.080 are provided in Section 5B.3 of the *Facility Design Guide*.

1. On diagonal ramps (such as ramps in diamond or parclo interchanges), the desirable minimum design speed is the value from the middle range of Table 6-3.04A. This applies to all but the most constrained situations (buttonhook configurations and urban core locations, for example). The minimum design speed is the value from the lower range of the same table. To avoid excessive interchange footprints, design speeds in the high range are not recommended for diagonal ramps having reversing curvature, particularly those in parclos.
2. For loops, AASHTO recommends a design speed no less than 20 mph (110-ft radius) for use with high-speed highways and encourages above-minimum designs in less constrained locations. Radii between 140 ft (22.5 mph) and 170 ft (25 mph) have exhibited good performance with typical freeway design speeds (50 mph to 70 mph) where spiral or robust circular transition treatments are used (see 6-3.04.02). A maximum practical radius is 250 ft (30 mph), above which space requirements and travel times become excessive.
3. On semi-direct connections (as shown in Figure 6-1.03K) as well as outer connections in cloverleaf and semi-directional interchanges, the minimum design speed is the value from the middle range of Table 6-3.04A. This also applies to two-lane semi-direct connections.
4. A direct connection (as shown in Figure 6-1.03L) often carries a mainline route or has comparable significance or traffic demand. In these cases, a uniform design speed along its entire length based on guidelines for mainline highways may be appropriate. A value somewhat lower than for an open-road condition is often justified, however, to fit configuration and constraint. The minimum design speed for any direct connection is the value from the middle range of Table 6-3.04A, not less than 40 mph.

Refer to Chapter 3 for criteria pertaining to superelevation rates and transitions. Generally apply superelevation to the first/last curve adjoining the mainline terminal per Table 3-3.02A and the selected ramp design speed; however, curves less than 300 feet in length may be sloped at the normal cross slope rate to avoid near-continuous transitioning though the curve. To simplify design, secondary curves on diagonal ramps should be superelevated only as necessary to limit side friction to f_{max}, based on Figure 3-3.03A and an assumed speed at that point on the ramp. Loops should always receive full superelevation (0.06 to 0.08 (ft/ft, m/m)).

Table 6-3.04A
RAMP DESIGN SPEEDS

Highway Design Speed (mph)	40	45	50	55	60	65	70	75
Ramp Design Speed (mph)								
High Range (85%)	35	40	45	50	50	55	60	65
Middle Range (70%)	30	30	35	40	45	45	50	55
Low Range (50%)	20	22.5	25	27.5	30	30	35	40
Corresponding Minimum Recommended Radius (ft) (rounded) Based on 0.06 (ft/ft) Superelevation								
High Range	380	510	660	835	835	1,050	1,350	1,650
Middle Range	275	275	380	510	660	660	835	1,050
Low Range	115	150	185	225	275	275	380	510

6-3.04.02 Horizontal Alignment

Horizontal alignment will be largely determined by the selected design speed. The minimum radii given in Table 6-3.04A are computed based on the values of maximum side friction for rural roadways presented in Table 3-2.03A; the values therein should be extrapolated for design speeds under 30 mph (50 km/h). Ramps and loops are to be designed using these rural side friction values regardless of the cross section design (rural or urban). All ramps should be as directional and flat-curved as possible. This applies, for example, on diagonal ramps at cloverleaf interchanges. The diagonal ramp should be as directional as possible, but may be allowed to follow a reverse curve path around the loop if site conditions are restrictive. Loops pose particular problems. The preferred design is to provide a 3-centered compound curve, the center curve being the minimum radius. The arrangement may be symmetrical or asymmetrical as may be appropriate for any variance in design speed between the two intersecting highways. A 3-centered arrangement allows for a transition between the mainline to the sharpest part of the loop curve, and it eases the acceleration and deceleration problems at either ramp end. For ramps and loops, the ratio of the flatter radius to the sharper one should not exceed 2:1. The length of the flatter transition curve should allow for a desirable acceleration/deceleration rate of 2 mph/sec (3 km/h/sec), and a minimum rate of 3 mph/sec (5 km/h/sec). It is also acceptable to provide a loop of constant-radius curvature. Desirable stopping sight distances should be used to check horizontal curvature. The values and the methodology are presented in Section 3-2.0. The desirable first curve of an exit ramp and the last curve of the entrance ramp are described in Sections 6-2.03 and 6-2.04. See Figure 6-2.03C.

6-3.04.03 Vertical Alignment

Maximum grades for vertical alignment cannot be as definitely expressed as for highway mainline, but preferably should not exceed 5 percent. General values of limiting gradient for ramps are shown in Table 6-3.04B, but for any one ramp the gradient to be used is dependent upon a number of factors peculiar to that site and quadrant alone. These factors include the following:

1. The flatter the gradient on the ramp, the longer it will be.
2. The steepest gradients should be designed for the center part of the ramp. Landing areas or storage platforms at at-grade intersections with ramps should be as flat as possible, as discussed in Section 5-2.02.
3. Short upgrades of 7 to 8 percent permit safe operation without unduly slowing down passenger cars. Short upgrades of up to 5 percent do not unduly affect trucks and buses.
4. Downgrades on ramps should follow the same guidelines as upgrades. They may, however, safely exceed these values by 2 percent, with 8 percent considered the desired maximum.
5. Ramp gradients and length can be significantly impacted by the angle of intersection between the two highways and the direction and amount of gradient on the two mainlines.

Ramp profiles usually have vertical curves at either end, with a straight grade in the center portion. The vertical curves should have designs which meet the criteria for desirable stopping sight distance as presented in Section 3-4.0. If vertical curves are designed at the mainline/ramp junctions, they should meet the design speed of the ramp.

Table 6-3.04B (U.S. Customary)
RAMP GRADIENT GUIDELINES

RAMP DESIGN SPEED (mph)	15	20	25	30	35	40	45	50
MAXIMUM GRADE (%)	8	8	7	7	6	6	5	5

Table 6-3.04B (Metric)
RAMP GRADIENT GUIDELINES

RAMP DESIGN SPEED (km/h)	30	40	50	60	70	80
MAXIMUM GRADE (%)	8	7	7	6	5	5