Interchange Ramp Geometrics—Alignment and Superelevation Design

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Several issues associated with the alignment and superelevation design of interchange ramps deserve renewed consideration in the light of research and the changing aspects of the road user. The results of recent studies and a survey of state highway design agencies are summarized. Responsible interchange design requires that the ramp geometrics be analyzed as a three-dimensional system to be certain that the facility will function as anticipated. Interchanges present the motorist with a complex set of decisions that require quick evaluation and action. Designers can reduce drivers' stress at interchanges by keeping the alignment simple and direct, maintaining design consistency, providing sight distances greater than the minimum stopping sight distances, and using above-minimum design criteria for other geometric elements. Some of the major principles that emerge include the following: (*a*) the average age of the general population is increasing, and, therefore, the characteristics of the average driver are changing; (*b*) large truck-trailer combinations are becoming more common and their requirements for proper superelevation rates and longer stopping distances exceed those of the automobile to the extent that previous margins of safety are being eroded; (*c*) spiral curves provide the most appropriate means to effect superelevation and be certain that the roadway and motorist interact in the manner expected; (*d*) superelevation rates for ramps used by large trucks should be based on reduced side friction factors; (*e*) the maximum speed differential between adjacent alignment elements should not exceed 10 mi/hr (16 km/hr); and (*f*) vertical and horizontal coordination is particularly critical when horizontal curves occur at the end of a downgrade and at the top of a vertical curve—conditions typical of interchange ramp design.

The design of interchanges concentrates most of the elements of highway design into one set of interrelated tasks with often conflicting goals. The design principles of two of these elements—ramp alignment and superelevation—should be examined in the light of changing aspects of the road user, the desire of highway design professionals to provide facilities that better meet the demands of the driving public, and recent research that explains the operational aspects of geometric features.

This paper includes information on the horizontal alignment and superelevation design of interchange ramps gathered from published reports and a 1991 survey of governmental design agencies. The information provides an improved understanding of how motorists use the highway and shows how properly designed geometrics can enhance driving and influence safety.

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FACTORS THAT INFLUENCE RAMP ALIGNMENT AND SUPERELEVATION DESIGN

The following key factors directly affect ramp alignment and superelevation design: design consistency and simplicity, the roadway user, design speed, and sight distance.

Design Consistency and Simplicity

Simple and consistent feedback must be presented to the motorist regarding the relationship between each element of the ramp geometry. Drivers' reaction time is slowed when these elements are different from those expected. Increased reaction time can be critical in an interchange area where the drivers' information processing skills are heavily taxed. Accordingly, when complex interchange designs are unavoidable, the designs should provide long sight distances; careful coordination between horizontal and vertical alignment; generous curve radii; and smooth, coordinated transitions.

The Roadway User

Many highways in the past have been designed in anticipation of use by "average" or "reasonably prudent" drivers. This has left little or no margin for the drivers whose capabilities do not meet the highway designer's expectations.

Two groups of drivers with diminished capabilities are receiving increasing attention—the elderly and those impaired by medication, alcohol, or drug usage. Truck drivers also deserve renewed consideration. Results of research on heavy truck performance show that the roadway design features that influence truck behavior need to be reevaluated.

Elderly Drivers

The increasing age of the general population will result in diminished capabilities for the average road user. Designers of interchange alignment can reduce some of the effects of increased age by increasing sight distance and simplifying interchange layout (*1*). Problems associated with the elderly driver that relate to interchange design include the following:

- Decreased visual acuity (beginning at age 40);
- Short-term memory decline, which causes problems in organizing information coming from a variety of sources;
- Reduced decision-making ability;

- Reduced ability to judge vehicle speed;
- Discomfort or pain caused by muscular stiffness and arthritis, reducing the ability to act quickly or look in all traffic directions; and
- Early fatigue, leading to slower reaction times (*2*).

Large Trucks

Since 1965 the length of the largest design vehicle in the American Association of State Highway Officials' policy on geometric design has more than doubled. Twenty-eight states out of 38 (74 percent) responding to a 1991 survey indicate that they currently consider designs for the WB-62, or larger, design vehicle. The WB-62 is a combination truck with a total length of 69 ft (21 m) and an outside minimum turning radius of 45 ft (13.5 m).

Truck operational characteristics on interchange ramps deserve additional consideration because accident records indicate that as much as 20 percent of truck accidents occur at interchanges. Traditional design guidelines for roadway alignment, based on the operation of automobiles, are not always safe for larger trucks and require modification in some instances.

Design Speed

Design speeds for ramps should bear a relationship to the type of ramp and the design speed of the connecting roadways. AASHTO recommends that ramp design speeds approximate the low-volume running speed of the intersecting highways and offers a range of speeds in Table X-1, p. 960, AASHTO *Policy* (*1*). These values apply to the sharpest or the controlling ramp curve, usually on the ramp proper, but they do not apply to the ramp terminals.

The AASHTO policy provides two sets of design criteria for horizontal curves on ramps: one set for freeway ramps and high-speed arterials and collectors [Tables X-1 and III-6, AASHTO *Policy* (*1*)] and one set for ramps connecting lower-speed arterials and collectors [Table III-17, AASHTO *Policy* (*1*)]. "Low" speed, in this context, refers to speeds of 40 mi/hr (64 km/hr) and less.

The ramp proper should be viewed as a transition area with a design speed equal to the speed of the higher-speed terminal wherever feasible. Few diagonal or loop ramps are long enough to accommodate more than two design speeds. The terminals and the ramp proper should be evaluated as a system to ascertain the appropriate speed for design.

Semidirect and direct ramps, designed to convey large volumes of high-speed traffic generally require considerable expense for their construction principally because of the grade separation structures necessary. Once it is determined that this expense is necessary, every effort should be made to apply a design speed equal to that of the connecting roadways.

Where it is anticipated that a controlled ramp will terminate in a stop sign–controlled intersection, consideration should be given to the possibility that the intersection may be signalized eventually and require geometry and sight distances appropriate for the resulting higher-speed travel.

Sight Distance

Sight distances greater than published minimums are particularly desirable at interchanges because drivers have to make judgments concerning optional travel paths, regulatory information, and often relatively severe geometrics. Accordingly, appropriate sight distances are critical to safe design because roadways with generous sight distances reduce driver anxiety and confusion, allow longer decision and maneuver times, and help compensate for driver and vehicle inadequacies.

Stopping Sight Distance

The values of stopping sight distance presented in the 1990 AASHTO geometric design policy are the minimums for avoidance of a road hazard—not leisurely stops. These minimums are based on the following conditions:

- Brake reaction time is predicated on a 2.5-sec driver reaction time, a time representative of 90 percent of the drivers in a 1971 study by Johansson and Rumar (*3*). A 3.5-sec perception and braking time has been suggested for the elderly driver (*4*).
- Braking distance values are the result of friction factors developed from studies performed from 1948 to 1962. The 1962 studies were performed on high type pavements with a locked-wheel trailer. The values were developed for wet pavement conditions. It is important to note that "truck tire braking capability is approximately 0.7 the frictional capability of passenger car tires. When a truck driver modulates the brakes to prevent spinning or jack-knifing, trucks require stopping sight distances approximately 1.4 times those of passenger cars" (*5*).
- Alignment also affects braking distance. The values provided assume that the vehicle is on a tangent section and not in a horizontal curve. Curves impose greater demands on tire friction than tangents. Therefore, the braking distance increases when the friction requirements of curves and braking are combined (*6*).
- The height of the driver's eye is set at a distance of 3.5 ft (1.07 m), whereas the object (hazard) height is considered to be 6 in. (15 cm) above the road surface. Although the extra eye height of a truck driver, 8 ft (10.4 m), helps compensate for a truck's longer stopping sight distance in some cases, the additional height does not provide an advantage against horizontal height obstructions such as bridge piers, walls, and tree foliage.

Height-of-eye distances vary, even for the automobile driver. Different automobiles position the driver's eye differently, and the eye-height of elderly drivers is lower than 3.5 ft (1.07 m) because the elderly are often shorter in physical stature (*2*).

Decision Sight Distance

Decision sight distance is important when stopping sight distance values do not provide the time necessary to process information and react appropriately. Examples of locations at interchange ramps where decision sight distance is desirable include ramp terminals at the main road, especially at an exit terminal beyond the grade separation and at left exits; ramp terminals at the cross-road; lane drops; and abrupt or unusual alignment changes.

AASHTO recognizes the difficulties presented by providing decision sight distances at the ramp exit terminals beyond a crest vertical curve (e.g., conventional cloverleaf loop ramp exit). As an alternative, AASHTO suggests the stopping sight distance plus 25 percent. Of the state design agencies responding to a 1991 survey about distances used when locating ramp exits beyond a crest vertical curve, 15 (38 percent) use the safe stopping sight distance, 9 (23 percent) use the safe stopping sight distance plus 25 percent, and 12 (31 percent) use decision sight distance. Along the ramp proper, 30 (77 percent) use the safe stopping sight distance values, and only 4 use more generous minimum criteria.

Figure 1 shows the comparative distances required for grade separations for these three sight distance conditions. The comparison assumes a 60 mi/hr (96 km/hr) design speed and an 800-ft (240-m) weaving lane centered at the structure.

Intersection Sight Distance

The sight distances provided at controlled ramp terminals should be determined in the same way as conventional at-grade intersections with consideration for the additional obstacles that are often present, such as bridge parapets, sidewalk or curb adjacent to the bridge parapet, bridge abutment or slopes, concrete median barriers, bridge piers, and retaining walls.

Both the horizontal sight triangle and the vertical sight distance must be checked to ensure that adequate intersection sight distance is available. The stopping sight distance of the vehicle on the cross-road must also be considered. In some cases vertical curves longer than the minimum will have to be designed.

Eye and vehicle elevations must be adjusted for the elevation differences in the roadway cross section and the grades of the intersecting roadways.

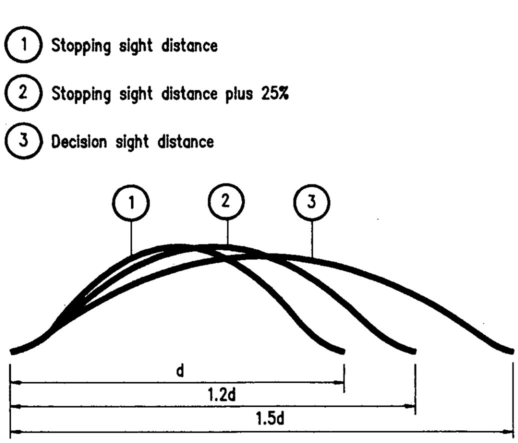


FIGURE 1 Comparative grade separation lengths for various sight distance criteria.

SUPERELEVATION

The proper superelevation rates are decided upon before beginning the ramp alignment design and after determining the traffic characteristics, selecting the design speed, and determining the basic ramp control points at the termini.

Side Friction Factors

Factors Used for Design

The maximum side friction factors (point of impending skid) developed between tires and wet concrete pavements range from about 0.5 at 20 mi/hr (32 km/hr) to 0.35 at 60 mi/hr (96 km/hr). These are not the values used for design, however. The AASHTO *Policy* states, "The speed on a curve, at which discomfort due to the centrifugal force is evident to the driver, can be accepted as a design control for the maximum allowable amount of side friction" (*1*). The margin between the maximum design value and the point of impending skid at 20 mi/hr (32 km/hr) is 0.33 and at 60 mi/hr (96 km/hr) is 0.23. The significance of this margin becomes evident when evaluating the operation of large trucks on interchange ramps.

AASHTO recognizes the special conditions that can occur at intersection curves and on ramps, such as less opportunity to develop transition lengths and the controlling geometry of adjacent lanes. On ramps with design speeds of 40 mi/hr (64 km/hr) or less, AASHTO has listed a group of superelevation rates [(*1*) Table IX-12, p. 777] with side friction factors ranging from 0.28 for a 50-ft (15-m) radius curve at 15 mi/hr (24 km/hr) to 0.02 for a 3,000-ft (900-m) radius curve at 40 mi/hr (64 km/hr).

The ramp designer is cautioned to use values in the upper half or third of the range shown wherever possible. Unless the curve is transitioned with spirals, the designer should also calculate the friction factor at the point of curvature to ensure that the suggested maximum side friction factor is not exceeded.

Factors for Large Trucks

The AASHTO friction factors are considered inappropriate for large trucks because the limiting factor in truck operation is "likely to be the rollover limit rather than skidding" (*7*).

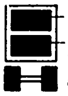
The following formula, based on a formula developed by the University of Michigan Transportation Research Institute can be used to determine the maximum friction factors for large trucks:

$$f_{max} = \frac{RT - SM}{1.15} - e_{pc} \quad (1)$$

where

RT = rollover threshold value in terms of g,
SM = safety margin = $0.10\,g$,
e_{pc} = superelevation at the curve PC, and
1.15 = steering factor.

Rollover threshold values and their associated truck types are shown in Figure 2.

Configuration			Rollover Threshold (g's)
	34 lb/cu ft 80,000 lbs GVW	Full gross, Medium-density Freight	0.34
	30% Wt 70% Wt 73,000 lbs GVW	Typical freight Load	0.28
	18.7 lb/cu ft 80,000 lbs GVW	Full gross, Full cube, Homogeneous Freight	0.24
	80,000 lbs GVW	Full gross Gasoline Tanker	0.32
	80,000 lbs GVW	Cryogenic Tanker	0.26

1 lb = 0.454 kg
1 in = 0.0254 m
1 lb/cu ft = 16.01 kg/cu m

FIGURE 2 Example large-vehicle configurations and rollover threshold values.

Example: A ramp terminal curve to the right is being de-
signed to accommodate a large percentage of trucks. The
design speed is 30 mi/hr. The curve radius selected initially
s 260 ft. Two-thirds of the superelevation runoff length will
be placed on the tangent and one-third on the curve. The
naximum superelevation rate is 0.08, and the normal cross
lope is 0.02. The rollover threshold is selected from Figure
to be 0.28. Determine whether the side friction factor is
xcessive and whether the curve radius should be revised.

Solution: Determine e_{pc}:

$$.02 + 0.6667(0.08 - 0.02) = 0.06$$

ubstituting into Formula 1 gives

$$_{\text{nax}} = \frac{0.28 - 0.10}{1.15} - 0.06 = 0.0965$$

ubstituting this value in the standard formula for curve adius,

$$_{\text{min}} = \frac{V^2}{15(e + f)}$$

ves a revised curve radius of 340 ft. (Note: 1 mi/hr = 1.6 n/hr, 1 ft = 0.3 m, and 15 is a U.S. conversion factor; the etric equivalent is 127.2.)

If the superelevation is developed along a spiral, the term $_c$ can drop out because the maximum superelevation will be lly achieved at the point of greatest demand.

Maximum Superelevation Rates

Values of maximum superelevation for interchange ramps are selected after accounting for the amount and frequency of ice and snow and the expected level of stopped, slow, or fast moving traffic characterized as urban or rural conditions.

Maximum ramp superelevation rates in use where snow and ice can be present are less than or equal to 0.06 or 0.08. Rates of 0.10 to 0.12 are used where snow and ice do not influence design. Urban rates are sometimes reduced to 0.06 or less. Responses to a survey of state design agencies show that the predominant superelevation rate for rural areas is 0.08 followed closely by 0.06; the predominant urban rate is 0.06. The most extreme rates reported are 0.04 (urban) and 0.12 (urban and rural).

Maximum superelevation rates for open roads are appropriate for directional and semidirectional ramps and other ramps with design speeds greater than 40 mi/hr (64 km/hr). Rates for ramps with speeds of 40 mi/hr (64 km/hr) or less are shown in Table IX-12, p. 777 of the AASHTO *Policy* (*1*). The highest rates listed in this table should be used whenever feasible.

More recent research shows that the margin of safety implied by the AASHTO side friction factors is eroded by a significant number of drivers who do not track the designed circular curve path, but follow a sharper curve path (*6*). Accordingly, the report states that more superelevation is required than is called for by AASHTO policy to produce the intended lateral tire accelerations at design speed for these drivers on an AASHTO criteria highway curve.

The designer should also consider the grades approaching the curve when selecting superelevation rates and curve radii. The steeper negative grades (downgrades) found in interchanges can lead to speeds higher than the chosen design speed—especially on loop ramps.

Superelevation Transition

Superelevation Transition Length

For ramp design, it is most convenient to determine superelevation transition lengths by establishing the relative slopes between the ramp pavement edges. AASHTO recommends values for relevant edge of pavement grades and relates them to design speed (AASHTO *Policy*, Table III-14, p. 177). These lengths roughly equal the distance traveled in 2 sec at the given speed.

Superelevation Transition Location

Traditionally, the superelevation runoff length has been located on a spiral curve leading into the circular curve. If no spiral is present, the roadway changes from requiring no superelevation (on the tangent) to requiring full superelevation instantaneously beyond the point of curvature (PC). Theory seems to favor putting most of the superelevation runoff on the tangent section so that the vehicle is positioned for the lateral forces exerted by the curved path. Current practice substantiates this rationale. Of the state design agencies responding to a survey, most (90 percent) used this general position.

With the short-length curves often encountered in ramp design, it can be tempting to begin the superelevation runoff exiting the curve immediately after the runoff entering the curve. A length of at least 50 ft (15 m) of full superelevation should be provided, however, to allow development of the vertical curves mentioned above. Without this distance, the pavement edge profile grade would change 1 percent without a vertical curve.

Superelevation Development at Free-Flow Ramp Terminals

Free-flow ramp terminals frequently join the main road in a curved alignment. The methods of developing ramp superelevation are described in the AASHTO *Policy* (*1*, pp. 966–969). The gore neutral area can be used to facilitate elevation changes between the ramp and the main road, but it should always slope away from the main line. The only exception is if the ramp joins the main road on the high side of a superelevated curve and both main road and ramp are curving in the same direction.

The maximum limits of algebraic difference between the cross-slope grades of the ramp and the main road must not be exceeded. Table IX- 14, p. 785, AASHTO *Policy* (*1*) lists these maximum algebraic differences and relates them to design speed. The most frequently cited value among the design agencies surveyed was 0.05 regardless of design speed. Other values ranged from 3 percent at 60 mi/hr (96 km/hr) to 9 percent at 15 mi/hr (24 km/hr).

The most difficult ramp terminal arrangement to design properly is one in which the ramp joins the right side of a main road curve to the left. It is difficult to achieve the proper superelevation on the ramp while not violating the crown crossover grade restrictions. This can only be resolved satisfactorily during the initial planning stages of the interchange. If the main road must curve to the left, it should be with as flat a curve as possible. For example, the radius of a main line curve with a design speed of 60 mi/hr (96 km/hr), e = 0.08, maximum, at an exit ramp with a 50-mi/hr (80-km/hr) design speed should be no greater than 4,279 ft (1289 m) or 0.75 degrees.

Superelevation and Large Trucks

Adequate superelevation enhances large truck operation, but only if the roadway is properly designed and constructed. Superelevation helps prevent truck rollover by tilting the truck in the direction opposing the lateral acceleration forces. It is not effective, however, unless it is developed early in the curve, where the truck will typically receive the highest lateral acceleration (*8*).

Locating the superelevation runoff beyond the point of curvature can result in the friction factor exceeding the AASHTO recommended maximum until full superelevation is reached, making smooth truck operation more difficult (*7*). The superelevation rate and the maximum side friction factor should be checked against the rollover threshold values for large trucks as explained in the previous section on side friction factors.

This check is particularly important at the end of long downgrades. The increase in speed often caused by downgrades should not be greater than the design speed of the curves that follow. Long, steep negative grades (greater than 5 percent) require the drivers of large trucks to be extra cautious with braking and encourage speeds above the design speed. When the ramp ends in a horizontal curve under these conditions, jackknifing or rollover becomes much more likely.

HORIZONTAL ALIGNMENT

Basic Alignment Principles

The following basic alignment principles contained in the AASHTO *Policy* (*1*) have been tailored to apply particularly to interchange ramps:

- Intersection alignment—The controlled ramp intersection angle should be as close to 90 degrees as possible, and the ramp should not intersect the cross-road at a sharp curve. The construction of short-radius curves to achieve right-angle intersections should be avoided.
- Curvature rate—The maximum degree of curvature should be used only in the most unresolvable situations, not as a means of achieving the most expedient or least costly design.
- Central angles—Large central (deflection) angles, greater than 45 degrees, are inherent in some ramp configurations, but they should be avoided where feasible. Small central angles should be absorbed by the longest curve practicable.
- Curve length—Adequate curve length is sometimes slighted, especially at small directional changes. Although the desirable curve lengths recommended by AASHTO for small central angles on open roads may not be achievable on ramps, curves must be long enough to provide proper superelevation.
- Consistent alignment—Consistent alignment, alignment expected by the driver, is facilitated by design that relates the horizontal elements to one another by design speed. The maximum speed changes between successive design elements have been recommended by various studies and design agencies. The most common value used in the United States for ramp design is less than or equal to 10 mi/hr (16 km/hr). German design guidelines for open roads recommend a change less than or equal to 6 mi/hr (10 km/hr) (*9*). Swiss design standards show a speed change equal to or less than 12 mi/hr (20 km/hr) (*10*) as being satisfactory to provide a good speed change relationship between horizontal alignment components.

Consistent alignment avoids introducing sharp curvature at or near the top of a crest vertical curve where the beginning of the curve cannot be perceived by the driver. The horizontal curve should be longer than the vertical curve. Vehicles beginning a horizontal curve near the low point of a sag vertical curve could be traveling too fast to track the horizontal curve properly.

Sharp curvature following a long tangent is not expected by drivers, who have difficulty judging speed reduction requirements and can enter the curve at too high a speed.

• Reverse curves—Reverse curves should always be separated by tangents of a length adequate to provide proper superelevation runoff. Ideally the tangent length will be long enough to accommodate both the runoff and the tangent runout. If this distance is not available, the pavement edges stay near the same elevation for a long distance, resulting in decreased transverse drainage. If the section is not long enough for the tangent runout, increase the runoff lengths until they meet in an instantaneous level section (*1*).

Drivers generally expect curves to reverse direction rather than curve in the same direction. Therefore "broken-back" curves should be avoided on diagonal ramps where such a change is not expected.

• Compound curves—Compound curves with large radii differences are inconsistent with driver expectations. It is generally accepted practice to allow the ratio of the flatter-curve radius to the sharper-curve radius to vary by no more than 2:1. If this is done, however, the arcs' lengths should be sufficient to enable motorists to decelerate and accelerate at a reasonable rate over the range of speeds.

Loop Ramps

Loop ramps have the most complicated horizontal geometrics of the ramp types mentioned. These ramps are particularly difficult because one goal is to keep them as short as possible to keep the interchange footprint small and reduce travel time. Considering these factors, AASHTO states that the practical size of loop ramp radii should be 100 ft (30 m) to 150 ft (45 m) for minor movements on highways with design speeds of 50 mi/hr (80 km/hr) or less, and 150 ft (45 m) to 250 ft (75 m) for more important movements on highways with greater design speeds (*1*).

There are three basic schemes of curve arrangement for loverleaf loop ramps: single radius curve, flat-sharp-flat radius compound curves, and sharp-flat-sharp radius compound urves. Of these, the flat-sharp-flat combination is the most widely used, followed closely by the single radius curve. The harp-flat-sharp combination is tempting to use because it equires less land area than the other types (for comparable veaving lengths), but it is difficult for drivers to properly udge the safe speeds required. After passing through the first urve marked with a low advisory speed, the driver may inrease speed on the flatter curve to prepare for merging, being nable to perceive that the next (sharp) curve is more denanding (*11*). This speedup can be especially likely for trucks egotiating downgrades or accelerating to merge.

piral Curves

piral curves have been used effectively in highway design for ecades. They have been reserved for use on high-speed roadays or by only the most conscientious designers, however, ecause they have been unwieldy to calculate and stake in e field.

Every motorist drives spiral curves on every roadway partly ecause vehicles must track a spiraled path and partly because ivers apparently desire to do so. The actual observed path mulates a true spiral (clothoid) curve of the type used in highway alignment design. The motorist then tracks a radius of circular curvature sharper than that of the road centerline before following a path generally concentric with the roadway, if the curve is sufficiently long (*6*).

Motorists are able to drive a spiraled path because the pavement is usually wide enough to accommodate such a path on a combined tangent and circular curve. If the pavement is not wide enough, lane encroachment occurs, and drivers tend to track a curve radius even more severe than described above.

Interchange ramp alignment with its relatively severe geometrics seems particularly appropriate for the use of spirals. The complexity of design and stakeout is no longer an issue. Computer-aided design and automated field survey equipment have eliminated the tedious calculations previously required.

The advantages of spirals are significant:

- Spiral curves significantly reduce side friction demand for operating speeds at or above design speeds.
- The changes in lateral acceleration and truck roll angle are smoother, requiring less driver correction.
- Spiral curves follow the driver's natural path.
- Spirals provide the appropriate location for superelevation transition.

Of the state design agencies responding to a survey, only 12 (32 percent) require spiral curves for interchange ramps. The common ramp pavement width of 16 ft (4.8 m) provides space for the motorist to drive in a spiral path of his own making, but it does not improve the superelevation distribution or reduce side friction demand.

Simple Curves

The radius of curvature (or degree of curve) is directly related to the design speed by the practical limits of superelevation rate, side friction factors, and, for large trucks, the rollover threshold. Curves sharper than the minimum radius of curvature as determined by the standard formula would require superelevation rates higher than practical or operation beyond the safe limit of tire friction.

Curve lengths should be long enough to provide the proper superelevation runoff plus a central portion at full superelevation at least 50 ft (15 m) long. AASHTO provides minimum curve lengths for compound curves on the basis of a maximum deceleration rate of 3 mi/hr (4.8 km/hr). These lengths enable the compound curve to provide a smoother transition into a sharper curve.

CONCLUSION

Successful alignment and superelevation design balances the desirable features of high design speed, long sight distance, gentle curvature, and flat grades against the constraints imposed by the natural environment, available right-of-way, social and political considerations, physical obstructions, and limited economic resources. All of these constraints push planners and designers toward using the minimum design criteria in a situation where above-minimum criteria are particularly needed. The multiple demands on drivers' attention and reaction times can be mitigated by good geometric design.

Before final design it is imperative that the designer go beyond the standard criteria and analyze the operation of each geometric element three-dimensionally in relationship to the other elements. By considering the interrelationship of horizontal alignment, vertical alignment, and superelevation and the effects they have on the driver, the designer can understand how the ramp will operate under actual conditions and how to best modify the geometrics to result in a safe, efficient design.

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