

inside lane and the midpoint of the sight line is from 0.5 to 1.5 m [1.5 to 4.5 ft] greater than that for stopping sight distance. It is obvious that for many cut sections, design for passing sight distance should, for practical reasons, be limited to tangents and very flat curves. Even in level terrain, provision of passing sight distance would need a clear area inside each curve that would, in some instances, extend beyond the normal right-of-way line.

In general, the designer should use graphical methods to check sight distance on horizontal curves. This method is presented in Exhibit 3-8 and described in the accompanying discussion.

## General Controls for Horizontal Alignment

In addition to the specific design elements for horizontal alignment discussed under previous headings, a number of general controls are recognized in practice. These controls are not subject to theoretical derivation, but they are important for efficient and smooth-flowing highways. Excessive curvature or poor combinations of curvature limit capacity, cause economic losses because of increased travel time and operating costs, and detract from a pleasing appearance. To avoid such poor design practices, the general controls that follow should be used where practical:

- Alignment should be as directional as practical, but should be consistent with the topography and with preserving developed properties and community values. A flowing line that conforms generally to the natural contours is preferable to one with long tangents that slashes through the terrain. With curvilinear alignment, construction scars can be kept to a minimum and natural slopes and growth can be preserved. Such design is desirable from a construction and maintenance standpoint. In general, the number of short curves should be kept to a minimum. Winding alignment composed of short curves should be avoided because it usually leads to erratic operation. Although the aesthetic qualities of curving alignment are important, long tangents are needed on two-lane highways so that sufficient passing sight distance is available on as great a percentage of the highway length as practical.
- In alignment developed for a given design speed, the minimum radius of curvature for that speed should be avoided wherever practical. The designer should attempt to use generally flat curves, saving the minimum radius for the most critical conditions. In general, the central angle of each curve should be as small as the physical conditions permit, so that the highway will be as directional as practical. This central angle should be absorbed in the longest practical curve, but on two-lane highways the exception noted in the preceding paragraph applies.
- Consistent alignment should always be sought. Sharp curves should not be introduced at the ends of long tangents. Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided. Where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves.
- For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 150 m [500 ft] long for a central angle of 5 degrees, and the minimum length should be increased 30 m [100 ft] for each 1-degree decrease in the central angle. The minimum length for horizontal curves on main highways,  $L_{c \min}$ , should be about three times the design speed expressed in km/h [15 times the

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radius of curvature and minimum sight distance for that design speed, Figure 201.6 gives the clear distance ( $m$ ) from centerline of inside lane to the obstruction.

See Index 1003.1(12) for bikeway stopping sight distance on horizontal curve guidance.

When the radius of curvature and the clear distance to a fixed obstruction are known, Figure 201.6 also gives the sight distance for these conditions.

See Index 101.1 for technical reductions in design speed caused by partial or momentary horizontal sight distance restrictions. See Index 203.2 for additional comments on glare screens.

Cuts may be widened where vegetation restricting horizontal sight distance is expected to grow on finished slopes. Widening is an economic trade-off that must be evaluated along with other options. See Index 902.2 for sight distance requirements on landscape projects.

### 201.7 Decision Sight Distance

At certain locations, sight distance greater than stopping sight distance is desirable to allow drivers time for decisions without making last minute erratic maneuvers (see Chapter III of AASHTO, A Policy on Geometric Design of Highways and Streets, for a thorough discussion of the derivation of decision sight distance.)

On freeways and expressways the decision sight distance values in Table 201.7 should be used at lane drops and at off-ramp noses to interchanges, branch connections, roadside rests, vista points, and inspection stations. When determining decision sight distance on horizontal and vertical curves, Figures 201.4, 201.5, and 201.6 can be used. Figure 201.7 is an expanded version of Figure 201.4 and gives the relationship among length of crest vertical curve, design speed, and algebraic difference in grades for much longer vertical curves than Figure 201.4.

Decision sight distance is measured using the 3 1/2-foot eye height and 1/2-foot object height. See Index 504.2 for sight distance at secondary exits on a collector-distributor road.

**Table 201.7**  
**Decision Sight Distance**

Design Speed (mph)	Decision Sight Distance (ft)
30	450
35	525
40	600
45	675
50	750
55	865
60	990
65	1,050
70	1,105
75	1,180
80	1,260

## Topic 202 - Superelevation

### 202.1 Basic Criteria

When a vehicle moves in a circular path, it undergoes a centripetal acceleration that acts toward the center of curvature. This force is countered by the perceived centrifugal force experienced by the motorist.

On a superelevated highway, this force is resisted by the vehicle weight component parallel to the superelevated surface and by the side friction developed between the tires and pavement. It is impractical to balance centrifugal force by superelevation alone, because for any given curve radius a certain superelevation rate is exactly correct for only one driving speed. At all other speeds there will be a side thrust either outward or inward, relative to the curve center, which must be offset by side friction.

If the vehicle is not skidding, these forces are in equilibrium as represented by the following simplified curve equation, which is used to design a curve for a comfortable operation at a particular speed:



wide. See Chapter 7 of the Traffic Manual for glare screen criteria.

### 203.3 Alignment Consistency

Sudden reductions in alignment standards should be avoided. Where physical restrictions on curve radius cannot be overcome and it becomes necessary to introduce curvature of lower standard than the design speed for the project, the design speed between successive curves should change not more than 10 miles per hour. Introduction of curves with lower design speeds should be avoided at the end of long tangents, steep downgrades, or at other locations where high approach speeds may be anticipated.

The horizontal and vertical alignments should be coordinated such that horizontal curves are not hidden behind crest vertical curves. Sharp horizontal curves should not follow long tangents because some drivers tend to develop higher speeds on the tangent and could over drive the curve.

See "Combination of Horizontal and Vertical Alignment" in Chapter 3 of AASHTO, A Policy on Geometric Design of Highways and Streets, for further guidance on alignment consistency.

### 203.4 Curve Length and Central Angle

The minimum curve length for central angles less than 10 degrees should be 800 feet to avoid the appearance of a kink. For central angles larger than 30 minutes, a curve is required without exception. Above a 20,000-foot radius, a parabolic curve may be used. Sight distance or other safety considerations are not to be sacrificed to meet the above requirements.

On 2-lane roads a curve should not exceed a length of one-half mile and should be no shorter than 500 feet.

### 203.5 Compound Curves

Compound curves should be avoided because drivers who have adjusted to the first curve could over drive the second curve if the second curve has a smaller radius than the first. Exceptions can occur in mountainous terrain or other situations where use of a simple curve would result in excessive cost. Where compound curves are necessary, the shorter radius should be at least two-thirds the longer radius when the shorter radius is 1,000 feet or less. On one-way

roads, the larger radius should follow the smaller radius.

The total arc length of a compound curve should be not less than 500 feet.

### 203.6 Reversing Curves

When horizontal curves reverse direction the connecting tangents should be long enough to accommodate the standard superelevation runoffs given on Figure 202.5. If this is not possible, the 6 percent per 100 feet rate of change should govern (see Index 202.5(3)). When feasible, a minimum of 400 feet of tangent should be considered.

### 203.7 Broken Back Curves

A broken back curve consists of two curves in the same direction joined by a short tangent. Broken back curves are unsightly and undesirable.

### 203.8 Spiral Transition

Spiral transitions are used to transition from a tangent alignment to a circular curve and between circular curves of unequal radius. Spiral transitions may be used whenever the traffic lane width is less than 12 feet, the posted speed is greater than 45 miles per hour, and the superelevation rate exceeds 8 percent. The length of spiral should be the same as the Superelevation Runoff Length shown in Figure 202.5A. In the typical design, full superelevation occurs where the spiral curve meets the circular curve, with crown runoff being handled per Figure 202.5A. For a general discussion of spiral transitions see AASHTO *A Policy on the Geometric Design of Streets and Highways*. When used, spirals transitions should conform to the Clothoid definition.

### 203.9 Alignment at Bridges

Due to the difficulty in constructing bridges with superelevation rates greater than 10 percent, the curve radii on bridges should be designed to accommodate superelevation rates of 10 percent or less. See Index 202.2 for standard superelevation rates.

Superelevation transitions on bridges are difficult to construct and almost always result in an unsightly appearance of the bridge and the bridge railing. Therefore, if possible, horizontal curves should begin and end a sufficient distance from the bridge so that no part of the superelevation transition extends onto the bridge.