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E 300 GENERAL DESIGN ELEMENTS

The controls and criteria previously presented (traffic data, speed, capacity, etc.) have a direct bearing on the general design elements, with the design speed being the overall control. The general design elements are horizontal and vertical alignment, and sight distance. These elements, as well as superelevation, topography, and meeting existing improvements have a joint relationship and effect on the geometric design. These effects are discussed in the succeeding sections.

E 310 HORIZONTAL ALIGNMENT

The general considerations for determining horizontal alignment are not necessarily the same for rural or outlying areas as for urban areas. Therefore, the discussion of horizontal alignment that follows will include the general design policy as well as the use of superelevation on horizontally curved alignment and its limiting and modifying effect on City Streets.

Alignment should be as direct as possible consistent with topography. A balance should be struck between the two extreme alternatives of constructing a straight line between two points and following every curve or contour in the existing topography. Sudden sharp curves should not be introduced at the end of long tangents or long radius curves. When physical conditions necessitate the use of a lower than standard radius for a given design speed, the substandard radius should be gradually approached by successively sharper curves from the tangent or long radius curved sections. Horizontal alignment should provide at least a safe stopping sight distance, for a given design speed, throughout the entire length of the project. The criteria for sight distance are discussed further in this chapter.

Long radius curves should be used wherever economically feasible, reserving the minimum radius of curvature for use at only the most critical sections, such as where heavy cuts or fills or extensive right of way costs may be involved. A length of arc as long as possible should be used for curves with small deflection angles in order to prevent the appearance of a kink in the road alignment. See Section E 311.6 for minimum length of curve that may be used.

Compound curves (other than certain curb return radii on channelization projects) should be used only where the topography, construction right of way costs, or other conditions make it impractical to use a single curve. Where the use of a compound curve is unavoidable, and the shorter radius is less than 1000 feet the shorter radius should be at least two-thirds the length of the longer radius.

Reverse curves without the use of an adequate length of intervening tangent are undesirable because they produce a relatively abrupt change of alignment, making it more difficult for a driver to follow and stay in the same lane. The proper amount of superelevation may be difficult or impossible to apply throughout the entire length of the reversing system because of insufficient length of tangent to accommodate the superelevation runoff. Superelevation runoff is the general term denoting the change in cross-section from a normal crown section to the fully superelevated section, or vice versa.

A broken-back curve consists of two curves in the same direction connected by a short tangent. This type of alignment is not pleasing in appearance and is difficult to negotiate because it is not generally anticipated by the driver. Unless the use of broken-back curves is unavoidable, the alternate methods of alignment, listed in the order of desirability, that should be used are:

1. Increasing the length of intervening tangent.
2. Providing one single curve.
3. Using a compound curve.

The broken-back curve usually makes it necessary to carry superelevation across the intervening tangent portion. This may or may not be desirable. The first alternative may eliminate or reduce the superelevation of the tangent portion. The last two alternatives enable some degree of continuous superelevation to be maintained. See Section E 311. Where feasible, a curve beginning or ending near a bridge or grade separation should be so located.
that the superelevation transition does not take place on the bridge or the main structure of the grade separation.

### E 311 SUPERELEVATION

Where horizontal curvature is introduced into the alignment, it is desirable to use a radius large enough to permit safe travel at the desired design speed without the use of superelevation. However, to permit safe operation for shorter radius curves, use is made of superelevation. Superelevation used on substandard radii of curvature will permit a more uniform speed in all lanes and will eliminate abrupt changes in the maximum safe speed, particularly on reverse curves.

The relationship between design speed, curvature, and superelevation is given by the formula:

\[
E + F = \frac{0.067v^2}{R} = \frac{v^2}{15R}
\]

Where

- \(E\) = Superelevation rate in foot per foot
- \(F\) = Side friction factor in foot per foot
- \(V\) = Vehicle speed in mph
- \(R\) = Radius of curve in feet

The figures included in this section are based on this formula. Using the street classification and the corresponding design speed for the proposed project, values of radii, superelevation, and other factors may be obtained from these figures, which are briefly described in the following subsections.

#### E 311.1 Side Friction Factors

The maximum safe side friction factors vary from 0.09 foot per foot at 100 mph to 0.30 foot per foot at 20 mph. See Figure E 311.1. The side friction factor at impending skid is also shown on the figure. The factor of safety for the design value of \(F\) varies from 3.33 at 100 mph to 1.67 at 20 mph. The value of \(F\) recommended for design by AASHO is also shown on the figure. It is slightly more conservative than the value recommended by the Bureau of Engineering.

#### E 311.2 Maximum Safe Speed on Horizontal Curves

Figure E 311.2 has been prepared from the exact formula for superelevation, using the recommended value of \(F\) for maximum safe speed and rates of superelevation varying from 0.05 foot per foot to 0.12 foot per foot. This figure should be used for the solution of all problems concerning safe speed.

#### E 311.3 Superelevation and Superelevation Transition

The amount of superelevation and the length of the superelevation transition for radii larger than the minimum are shown graphically on Figure E 311.3. Formulas are given for calculating these values. The method of attaining the maximum superelevation is also shown. On flat grades, this recommended method of revolving the pavement surface about the centerline will result in sumps on the outer edge of the pavement. In order to avoid this condition, the pavement should be revolved about the inside edge rather than the centerline. In this case, the transition should be twice as long as the length shown on the figure. After a superelevation is computed, profiles of the pavement edge should be plotted and any uneven or distorted grades should be changed by using smooth curves.

#### E 311.4 Minimum Radius and Maximum Transition Length for Limiting Values of \(E\) and \(F\):

Figure E 311.4 gives minimum radii and transition lengths with maximum superelevation of 0.06 foot per foot, which is considered to be the desirable maximum for City streets. Minimum radii and transition lengths are also given for zero superelevation. The value of \(C\) is the rate of increase of the unbalanced centrifugal force in the formula for the length of transition. It is noted that the transition length to safely reverse the unbalanced centrifugal thrust is the same for the maximum superelevation, as well as for zero superelevation. This condition results from the fact that the formula for length is based on the maximum allowable unbalanced centrifugal thrust. From this figure it is possible to calculate the minimum desirable tangent between reversing curves of minimum radii. Since two-thirds of the maximum superelevation should be provided at the B.C. and E.C. of the curves, the minimum tangent length is two-thirds of the sum of the transition lengths. See Figure E 311.5.

#### E 311.5 Design of Horizontal Curves

As an example, Figure E 311.5 shows the application of the superelevation charts to the design of a typical local hillside street.

#### E 311.6 Minimum Length of Curve

Figure E 311.6, below, lists the minimum centerline ra-
Radius of horizontal curvature and the minimum length for a given highway classification, taking into consideration the maximum superelevation of 6 percent and the designated design speed. For smaller central angles, the centerline radius must be increased to maintain the indicated minimum length of curve.

An illustration of the use of this figure is as follows: Assume a local hillside highway classification with a required minimum centerline radius of 132 feet. The figure shows that a minimum of 100 feet of centerline arc length with a minimum central angle of 43.406 degrees must be provided. Where the central angle is less than 43.406 degrees, say 30 degrees, the centerline radius must be increased to a value that may be determined by using the following formula:

$$R = \frac{L}{\Delta}$$

Where:
- $R$ = Centerline radius required in feet
- $L$ = Minimum length of centerline required in feet
- $\Delta$ = Central angle of centerline in radians

Then:

$$R = \frac{100}{0.523598} = 190.989'$$

This means a centerline radius of at least 190.989 feet, say 200 feet, must be provided.

**E 312 IN URBAN AREAS**

In undeveloped areas, such as in new subdivisions, or for a type of project that occurs infrequently within City limits, such as Mulholland Drive, there is an opportunity to adhere closely to the theoretical design requirements previously presented. However, the characteristics of a grid and traffic pattern of most existing streets in urban areas tend to restrict or modify the use of the theoretical or desirable values of the elements of speed, horizontal curvature, and superelevation. For example, the speed is restricted by frequent intersection areas with traffic signal controls, constant turning and cross-traffic movements, dips due to cross-gutters, channelization and median islands with short turning radii, and congestion as a result of the heavy movement of pedestrians and vehicles.

The street alignment and the horizontal curvature, for the most part, are already existing, and the excessive right of way and construction cost may discourage or prohibit realignment for purposes of achieving a flat horizontal curvature. The values of superelevation that may be used are controlled by established street grades (meeting existing improvements) and drainage (extremely flat grades, excessive crossfall and crown sections, etc.). When the horizontal curves on a City street are of large enough radii to permit safe operation with a fully crowned section and at the proper design speed, superelevation is unnecessary and unduly complicates the design.

In general, values chosen from the figures should be tempered by the existing conditions rather than by the indiscriminate use of the theoretical. It should be kept in mind, however, that in no case should deviation from the standards of good design practice be so great as to render operation of pedestrian and vehicular movements unsafe, or to increase materially the City's legal liabilities and maintenance responsibilities.

**MINIMUM LENGTH OF CURVE**

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<th>Minimum Centerline Arc Length</th>
<th>Minimum Central Angle</th>
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<tr>
<td>Major</td>
<td>782'</td>
<td>400'</td>
<td>28.648°</td>
</tr>
<tr>
<td>Secondary</td>
<td>782'</td>
<td>400'</td>
<td>28.648°</td>
</tr>
<tr>
<td>Collector</td>
<td>443'</td>
<td>300'</td>
<td>38.801°</td>
</tr>
<tr>
<td>Local, Flat</td>
<td>212'</td>
<td>200'</td>
<td>54.053°</td>
</tr>
<tr>
<td>Local, Hillside</td>
<td>132'</td>
<td>100'</td>
<td>43.406°</td>
</tr>
</tbody>
</table>

Figure E 311.6
E 320 VERTICAL ALIGNMENT

As with horizontal alignment, there are general design policies for vertical alignment, as well as certain modifications that are required in dealing with urban areas. A general discussion, along with some of these modifications and the different application of vertical curves, are presented in the sections that immediately follow. Other design details and procedures are given in Chapter E 500, Grade Determination.

E 321 BASIC GRADE CONTROLS

Part of this discussion will cover the basic grade controls: maximum grades, minimum grades, and critical lengths of grade. The rest of the discussion, as with basic grade control, will probably have more application to freeways and highways in undeveloped and rural areas than to City streets. The general principles, however, should be practiced, whenever feasible, in City streets.

**E 321.1 Maximum Grades:** The basic grade controls vary with the type of terrain and the design speed. The relation of maximum grade to design speed is shown in Figure E 321.1, below. The figure shows grades for main highways and those longer than 500 feet. Shorter length grades may be 1 percent steeper. Highways with low volume of traffic, lesser designated highways, and extreme cases such as underpasses and bridge approaches may be approximately 2 percent steeper than shown. All these criteria should be used as a guide rather than as an absolute control. See Section E 500, Grade Determination.

**E 321.2 Minimum Grades:** Flat or level grades on uncurbed pavements in outlying areas may be acceptable when the pavement has sufficient crown to transversely drain the surface runoff. In cut sections or curbed streets, level or extremely flat grades cannot be used, since lateral drainage is blocked. In these cases, adequate longitudinal drainage should be provided. See Chapter E 500, Grade Determination.

**E 321.3 Critical Length of Grade:** The length of steep uphill grade beyond which truck traffic slows down to a speed which is more than 15 mph below the average running speed of all other vehicles is called the critical length of grade. See Figure E 321.3. The following should be used as a guide rather than as an absolute control. The length of the steep grade should not exceed the critical length for a given average running speed where long steep grades must be used and when one of the following conditions is present:

1. The rate of grade cannot be reduced.
2. An additional passing lane is provided on the ascending side of a two-lane highway.
3. An additional passing lane is provided on the ascending side of a four-lane highway approaching maximum capacity.

For further discussion of related design criteria and details, the designer is referred to Section E 020F (1b), Vertical Alignment.

In addition to the above basic controls for vertical alignment, there are several general controls that should be considered:

1. Avoid the use of a series of short breaks in an effort to closely fit the existing terrain. It is desirable to adopt a smooth grade line with gradual grade changes that are consistent with the standards for the particular highway classification under design consideration.
2. In rolling country, a highway following a straight horizontal alignment with a profile that

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**RELATION OF MAXIMUM GRADES TO DESIGN SPEED**

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<th>Type of Topography</th>
<th>Design Speed, mph</th>
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<tr>
<td></td>
<td>30</td>
</tr>
<tr>
<td>Flat</td>
<td>6</td>
</tr>
<tr>
<td>Rolling</td>
<td>7</td>
</tr>
<tr>
<td>Mountainous</td>
<td>9</td>
</tr>
</tbody>
</table>

*Figure E 321.1*
closely adheres to the existing natural ground line will usually produce a "roller coaster" effect or a "hidden dip". Oncoming vehicles occupying hidden dips contribute to passing maneuver accidents, since the driver attempting to pass may be deceived by the view of an open highway beyond the dip which appears free of approaching vehicles. To eliminate or reduce these dips and produce a smoother, more gradual change of grade, several alternatives are open:

a. Sacrifice some of the straight alignment and introduce horizontal curvature in a direction to follow more closely the same elevation contour lines.

b. Where it is desired to maintain a straight alignment and where, as a consequence, the highway is cutting across many contour lines, the use of higher fills and deeper cuts will tend to offset the degree of undulation.

c. Where circumstances dictate, use a combination of these first two alternatives.

3. On undulating grade lines that include long lengths of momentum (downhill) grades, truck traffic tends to operate at a higher speed than on those profiles containing an upgrade but not preceded by a downgrade. In permitting an undulating grade, the designer should weigh the advantage of permitting trucks to increase their average speed on the upgrade against what may be the greater disadvantage of enabling trucks to reach excessive speeds, to the detriment of other traffic, on the downhill side.

4. For esthetic reasons, a broken-back grade line (two vertical curves in the same direction connected by a short section of tangent grade) generally should be avoided, particularly in sags, where both vertical curves are in full view.

5. On long grades it may be preferable to place the steepest grades at the bottom and lighten the grades near the top of the ascent. It may also be more desirable to use, instead of a uniform sustained grade only slightly below the allowable maximum, an interspersion of short intervals of lighter and maximum grades. This is particularly true of low-design-speed highways.

6. Where intersections at grade occur on highway sections with moderate to steep grades, it is desirable to reduce the gradient through the intersection. The intersection may sometimes act as a control, in that the elevations contained therein are relatively fixed. In some cases the intersection grade may be flattened by steepening the grade at some distance before reaching the intersection, and then flattening the approach to and through the intersection without appreciably affecting the intersection control elevations. This flattening effect makes for easier drainage control and vehicular turning movements.

E 322 VERTICAL CURVES

E 322.1 Purpose: A vertical curve is used to avoid the sudden change of direction when moving from one grade to another. If the vertical curve is properly designed, it will provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance. If the curve is too short it will probably sacrifice some of these desirable features. On the other hand, long, flat vertical curves are undesirable, because they may develop poor drainage conditions. In addition, they may discourage some drivers from attempting passing maneuvers even though there may be an adequate passing safety margin.

One of the most important controls is ample sight distance for a given design speed. This factor will be discussed separately in Section E 340.

E 322.2 Properties: That portion of a simple parabolic curve which closely approximates the arc of a circular curve is generally used in highway design. Although highway lengths are measured on a horizontal plane rather than the profile slope, and because highway grades are generally flat, the use of a parabola results in no appreciable error. In addition, the ease of calculations of the vertical offsets from a tangent grade as against the more involved calculations of a circular curve justifies its use. Another advantage of the use of the true parabola is that it permits the sight distance and the speed to be calculated or scaled from charts. The sight distance and speed cannot be calculated for curves with unequal grade breaks or for curves with equal grade breaks including the two end breaks because neither of these curves is a true parabola. This means that time-consuming graphical methods must be resorted to for a determination of sight distance and safe speed. See Section E 340.
E 322.3 Computations: See Figure E 322.3.

E 322.4 Requirements for Comfortable Riding Qualities: Making riding conditions comfortable as well as safe should always be a part of the street designer's goal. The degree of comfort is affected by the length of curve, the design speed, and the grade differences. The relationship between these factors is determined by the formulas and conditions listed below.

E 322.41 Acceleration Not Perceptible: Where it is unnecessary to provide stopping sight distance equal to the safe stopping distance, as for example on a lighted sag curve, vertical curves should be of sufficient length to produce no perceptible sensation of vertical acceleration. The maximum vertical acceleration which will pass unnoticed on a vertical curve is approximately two feet per second, per second. See Section E 020F (4c). Using a value of 1.79 feet per second, per second for maximum vertical acceleration, the minimum length of vertical curve will be

$$L = \frac{1.2}{AV^2}$$

Where $A$ equals the algebraic difference in grades in percent + 100 and $V$ is the design speed in miles per hour. The length of curve given by this formula should be used only when sight distance requirements do not govern. See Figure E 322.41.

E 322.42 Maximum Acceleration: There will be a few instances, such as approaches to cross gutters and warped surfaces in intersections, where due to space limitations it will be necessary to use vertical curves which produce a definite sensation of vertical acceleration. The maximum vertical acceleration that still provides comfort is between four and five feet per second, per second. Using a value of 4.30 feet per second, per second for acceleration, the minimum length of vertical curve will be $L = 0.50 AV^2$.

E 322.43 Distance Between Grade Breaks: Grade breaks on vertical curves should be computed or plotted on the profile at such intervals that assuming the curve to be constructed as a series of chords, the maximum difference between the chord and the true curve shall not be greater than 0.02 of a foot. The distance between grade breaks which will limit this difference to 0.02 of a foot is given by the following formula:

$$d = 0.4\sqrt{\frac{L}{A}}$$

Where $d =$ The distance between grade breaks in feet,

$L =$ Length of vertical curve

$A =$ Algebraic difference in grades in percent + 100.
**E 330 COMBINATION OF HORIZONTAL AND VERTICAL ALIGNMENT**

The horizontal and vertical alignment of a highway should not be designed independently. Rather, the horizontal and vertical controls are determined, and their influence is considered jointly in order to arrive at the optimum highway location. The general controls to be considered for the proper combination of horizontal alignment and profile are as follows:

1. Long, flat grades at the expense of having to use excessive horizontal curvature, or long, flat curves or tangent alignments with long, steep grades are unsatisfactory extremes in design. A compromise is appropriate when, by sacrificing some of the ideal qualities of either a good horizontal alignment or a good grade, a relatively balanced design is achieved.

2. A vertical curve superimposed upon a curved horizontal alignment or vice versa is more pleasing esthetically than a series of humps visible to the driver for some distance. However, there are some attendant hazards with this type of design, and its effect on traffic operations should be carefully considered, as discussed below.

3. Horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve because it is difficult for the driver to see an alignment change. This is true especially at night where there are no other lights and as the vehicle approaches the summit, the headlight beams go straight ahead into space. The danger is somewhat reduced by commencing the curvature at a distance well before the top of the curve as an advance warning of an alignment change; i.e. the horizontal curve is made longer than the vertical curve.

4. Sharp horizontal curvature should not be used at or near the low point of a pronounced sag vertical curve, because the road ahead is foreshortened and any sharp horizontal curvature appears warped or distorted. In addition, trucks, in particular, have a tendency to pick up speed at the bottom of grades, and vehicular control is more difficult to maintain.

5. On two-lane highways, the need for safe passing sections at frequent intervals, and for an appreciable percentage of the length of the highway, overrides the general desirability of the horizontal and vertical alignment combination.

6. Horizontal curvature and profile should be made as flat as feasible at highway intersections, where sight distance along both highways is important and vehicles may have to slow down or stop.

7. On divided highways, variation in the width of median and the use of separate profiles and horizontal alignments should be considered to obtain the design and operational advantages of independent one-way roadways.
A primary consideration in the design of a highway is the arranging of the geometric elements to obtain adequate sight distance for safe and efficient operation. Sight distance should be considered in the preliminary design stages, when the horizontal and vertical alignments are still subject to adjustment.

Three items will be covered under the subject of sight distance:

1. The distance required for stopping, applicable on all highways.
2. The distance required for overtaking and passing vehicles, applicable on two-lane and three-lane highways.
3. The criteria for measuring these distances in the design of horizontal and vertical alignment.

The figures and equations presented will use the following notations in addition to those previously presented under horizontal and vertical alignment discussions:

- **S** = Sight distance, stopping distance, or stopping sight distance in feet.
- **V** = Design speed in miles per hour.
- **L** = Length of vertical curve in feet (see figures and equations for other specific L designations).
- **a** = Vertical acceleration in feet per second, per second.
- **R = \( \frac{L}{A} \)** = Minimum radius of vertical curve in feet (see figures and equations for other specific R designations).
- **A** = Algebraic difference of grades in percent (see figures and equations for other specific A designations).

**E 341 CRITERIA FOR MEASURING**

Sight distance along the highway is measured from the driver's eye to an object on the traveled way when it first comes into view. Measurement criteria for safe stopping sight distance are different from those for safe passing sight distance. Sight distance controlled by vertical alignment involves different elements than sight distance controlled by horizontal alignment. The height of the driver's eye is assumed to be 3.75 feet above the pavement surface.

On crest vertical curves, sight distance is limited by some point on the pavement surface. The height of object used to measure safe stopping sight distance on crests is 0.50 foot. The height of object for passing sight distance which represents the height of an oncoming vehicle in the opposing lane in a passing maneuver is 4.0 feet.

The average height of a truck driver's eye above the pavement is 6 feet. On sag vertical curves where sight distance is limited by an overcrossing and where a sufficient percentage of trucks are using the highway (no percentage figures available), this additional height should be considered for design purposes. See Subsection E 342.2. The height of the object is assumed as two feet above the pavement (representing the height of the taillight of a forward-moving vehicle).

For safe stopping distance on horizontal curves, the following height criteria are generally used at the midpoint of the sight line where the cut slope or obstruction usually interferes with sight: 3.75 feet for height of eye, 6 inches for height of object, and 2 feet for height above the pavement at the centerline of the inside lane radially opposite the point of obstruction. See Figure E 342.4A.

**E 342 SAFE STOPPING DISTANCES**

The minimum stopping sight distance is equal to the safe stopping distance. It is the sum of two distances:

1. The distance \( D_1 \) covered by a vehicle from the instant the driver sights an object to the instant the brakes are applied.
2. The distance \( D_2 \) required to stop the vehicle from the instant the brakes are applied.

The time required for the first distance also consists of two components and is called the perception time. This is the sum of the times that elapse from the instant an object appears to the driver to the instant of realization that a stop must be made. The amount of perception time varies with the operator, the road conditions, and the particular situation involved. AASHO considers 1.5 seconds as a sufficient time for most drivers. The second time interval is called brake reaction time and
is the time required to actually apply the brakes. AASHO uses one second as a safety factor to include most drivers. A constant value of 2.5 seconds for the total brake reaction and perception time is assumed by AASHO for all ranges of design speed in their development of values for minimum stopping sight distance.

Figure E 342 gives safe stopping distances for speeds from 20 to 70 miles per hour and formulas for calculating these distances. It also includes curves that provide a correction for the greater or lesser lengths traveled by a vehicle operating on descending or ascending grades. The formulas include a coefficient of friction between the tires and the roadway. An additional safety factor has been added by assuming a wet pavement and a correspondingly lower friction factor. Curves based on these equations are plotted on various figures included in this section.

The minimum stopping sight distance values are based on passenger car operation. Generally, trucks require a longer distance to stop, for a given speed, than do passenger vehicles. AASHO does not provide any values for the additional lengths that would be required at the various speeds. However, trucks generally travel slower, and the operator, seated at a higher level, is provided with a greater vertical sight distance than passenger vehicle operators. Therefore, no distinction is usually made for sight distance requirements between trucks and passenger vehicles. However, where truck traffic represents a considerable percentage of the traffic (no percentage figures available), efforts should be made to provide longer sight distances, particularly on downgrades, where truck traffic tends to increase speed.

E 342.1 Stopping Sight Distance on Crest (Summit) Vertical Curves: The algebraic difference in intersecting grades at the crest of a vertical curve is the basic limiting factor of the available sight distance. Figure E 342.1 gives the length of vertical curve necessary to provide the safe stopping sight distance required for a given algebraic difference in grade and for a given design speed. The algebraic formulas for calculating sight distances are included in the figure.

E 342.2 Headlight Sight Distance on Sag Vertical Curves: The minimum length of safe vertical curve should provide headlight visibility for unlit or poorly lit highways for a distance that is at least equal to the safe stopping distance for the design speed of the highway. The headlights are assumed to be 2.0 feet above the pavement surface and have a maximum deviation of the beam above the horizon of one degree of arc. The headlight beams may be cut off by the sag in grade and/or any overcrossing structures. In the case of the truck driver, the sight distance may be less than that of the passenger vehicle operator because the view, as provided by the higher eye level of the truck driver, is cut off sooner by an overcrossing structure. This factor should be considered in determining the length of sag vertical curves, with overcrossings, that have a large proportion of truck traffic (truck percentages not available).

Figure E 342.2 shows a series of curves for determining the length of sag vertical curves. These lengths will provide the necessary sight distance for a given algebraic difference in grade and a designated design speed. Formulas which may be used for determining the required length of vertical curves are also included.

E 342.3 Minimum Radii for Stopping Sight Distance on Vertical Curves: All of the formulas used in the investigation and calculation of vertical curves contain the ratio \( \frac{L}{A} \). It can be shown that \( \frac{L}{A} \) is equal to the minimum radius of a parabola. The minimum radius of a parabola occurs at the point where the slope of the tangent to the vertical curve is zero. On a summit vertical curve connecting a plus and a minus grade it is located at the highest point on the curve. On a sag vertical curve with the same conditions it will be located at the lowest point on the curve. On vertical curves connecting two plus or two minus grades it would occur not on the finite curve but on the imaginary prolongation of the curve at the point where the tangent grade is zero. Since both the sight distance and the riding qualities of a vertical curve are functions of the minimum radius, it is possible to specify minimum radii for vertical curves which will provide any desired sight distance or comfortable speed. See Figure E 342.3A, below. The use of the figure is illustrated by the following examples:

1. Given a crest vertical curve connecting a +6 percent grade and a −2 percent grade. Design speed is 50 miles per hour. Calculate the minimum
length of vertical curve for the stopping sight distance.

\[ A = \frac{6 - (-2)}{100} = 0.08 \]

From the figure opposite design speed of 50 miles per hour and under the column headed "Crest Curve — 6" Object", read \( R = 8763 \) feet. Then \( L = 8763 \times 0.08 = 701.04 \) feet, which is the minimum length of vertical curve to provide a stopping sight distance for 50 miles per hour.

2. Given a sag vertical curve connecting a +5 percent grade and a -4 percent grade. Design speed is 50 miles per hour. Calculate the minimum length of curve required for a lighted highway.

\[ A = \frac{5 - (-4)}{100} = 0.09 \]

From the figure opposite 50 miles per hour design speed and under the column headed "Speed-No Apparent Acceleration," read \( R = 3000 \) feet. Then \( L = 3000 \times 0.09 = 270 \) feet, which is the minimum length of a sag vertical curve to be used for 50 miles per hour on a lighted highway.

3. Given a crest vertical curve, design speed of 50 miles per hour, length of 600 feet, and an algebraic difference in grades of 7 percent. Can the stopping sight distance for a 6-inch object be satisfactory for the design speed?

\[ R = \frac{L}{A} = \frac{600}{10} = 8571' \]

From the figure opposite 50 miles per hour design speed and under the column headed "Crest Curves — 6" Object", read \( R = 8763 \) feet, which is the minimum radius required and which is greater than the radius calculated. The length of vertical curve is unsatisfactory. Therefore, either a greater length of vertical curve must be used or the algebraic difference in grades must be reduced. The minimum radii for stopping sight distance on vertical curves may be determined graphically by referring to Figure E 342.3B.

### MINIMUM RADII FOR VERTICAL CURVES

<table>
<thead>
<tr>
<th>Class of Highway</th>
<th>Design Speed — MPH</th>
<th>Safe Stopping Sight Distance — Ft.</th>
<th>Safe Stopping Sight Distance — Ft.</th>
<th>Safe Stopping Sight Distance — Ft.</th>
<th>Safe Stopping Sight Distance — Ft.</th>
<th>Safe Stopping Sight Distance — Ft.</th>
<th>Maximum Comfortable R = 0.50y</th>
<th>Maximum Comfortable R = 0.50y</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary</td>
<td>50</td>
<td>350</td>
<td>8,763</td>
<td>13,629</td>
<td>7,692</td>
<td>13,629</td>
<td>3,000</td>
<td>1,250</td>
</tr>
<tr>
<td>Secondary</td>
<td>50</td>
<td>350</td>
<td>8,763</td>
<td>11,613</td>
<td>7,692</td>
<td>11,613</td>
<td>3,000</td>
<td>1,250</td>
</tr>
<tr>
<td>Collector</td>
<td>40</td>
<td>275</td>
<td>5,410</td>
<td>6,532</td>
<td>5,550</td>
<td>6,532</td>
<td>1,920</td>
<td>800</td>
</tr>
<tr>
<td>Local—Flat</td>
<td>30</td>
<td>200</td>
<td>2,861</td>
<td>3,952</td>
<td>3,636</td>
<td>3,952</td>
<td>1,080</td>
<td>450</td>
</tr>
<tr>
<td>Local—Hillside</td>
<td>25</td>
<td>160</td>
<td>1,609</td>
<td>2,903</td>
<td>2,727</td>
<td>2,903</td>
<td>750</td>
<td>315</td>
</tr>
</tbody>
</table>

*Use this column when curve is adjacent to an intersection.*
curs, the safe stopping sight distance (non-passing sight distance), on horizontal curves for a given design speed determines the minimum horizontal radius to be used.

From the equation or curves shown on Figure E 342, the safe stopping sight distance can be obtained for a given design speed. Referring to Figure E 342.4A, an algebraic or graphic solution can be found, using the safe stopping sight distance S previously obtained, knowing either the available horizontal radius of curvature R or the available clear distance D as measured radially from inside edge of pavement or curb line.

Since extreme accuracy is not necessary, simple but approximate equations as shown on Figure E 342.4A and in the following discussion have been provided. The values secured in using these equations are slightly in error but are on the side of safety. If the height criteria are used for horizontal curves as stipulated in Section E 341, small height variations, such as nonlevel pavement cross-sections, do not usually affect design except where the sight restriction is a cut slope or other variable-height object.

The curves on Figure E 342.4A are limited to those cases in which the required sight distance is less than the length of curve provided. The approximate formula when \( S \leq L \) is:

\[
D = \frac{S^2}{8(R + \frac{W}{2})} - \frac{W}{2}
\]

Although these curves are plotted for lane widths of 12 feet, approximate answers may be obtained for other typical lane widths.

As noted above, the graph in the figure is for sight distances less than the length of curve provided. No such graph is available at this time for sight distances greater than the length of curve provided. However, it should be emphasized that there is an approximate formula for use in such cases. The approximate formula when \( S \geq L \) is:

\[
D = \frac{2SL - L^2}{8(R + \frac{W}{2})} - \frac{W}{2}
\]

The following examples illustrate the application of these formulas for a given set of conditions.

Given: Primary hillside collector street with a 74-foot-width right of way and a 60-foot-width roadway. See Figure E 113, Standard Street Dimensions. Figure E 311.6 indicates the use of a minimum centerline radius R of 443 feet where a curved street alignment is necessary. A minimum sight distance of 275 feet is required for a design speed of 40 miles per hour. See Figure E 342.4A. Other considerations assumed are a 6 percent pavement superelevation, a 2:1 side slope, a 2.5 percent sidewalk grade, and an 8-inch curb face.

Find: The horizontal distance from the curb to the point where the 2:1 slope through the obstruction intersects the 2.5 percent sidewalk slope, and determine if available clear horizontal distance D in feet from the curb line to the obstruction at height of the line of sight is adequate (does not require a sight distance easement). See Figure F 342.4B.

From the formula:

\[
L_L = \frac{R_L x L_S}{R_3}
\]

Where

- \( L_L \) = Length of curb lane centerline
- \( R_L \) = Radius of curb lane centerline
- \( L_S \) = Length of street centerline
- \( R_3 \) = Radius of street centerline

Then:

\[
L_L = \frac{418 \times 300}{443} = 283.07\text{'}
\]

Since the required sight distance of 275 feet is less than the centerline length of curb lane provided, the value D is obtained from the formula:

\[
D = \frac{S^2}{8(R + \frac{W}{2})} - \frac{W}{2} = \frac{275^2}{8(413 + \frac{10}{2})} - \frac{10}{2} = 17.6\text{'}
\]

From the curves, using a 12-foot-width lane, a clear horizontal distance D of 18.6 feet would have been required.

It can be seen from the figure that due to the slope of the existing ground, there is an insufficient distance D available. For most practical purposes, the approximate D distance is determined as in the above example and rounded off to the nearest higher even foot. Either the slopes are then graded back (in the above case to a D distance of 18 feet) in newly developed areas (such as new subdivisions) or, if the property has been previously developed, a sight distance easement is acquired for this grading. In order to calculate more precisely where the 2:1 slope through the obstruct-
tion intersects the 2.5 percent sidewalk slope, proceed as follows:

Assume a straight pavement grade around the curve along the lane centerline, between the driver and the object on the pavement. The average height of the driver's eye above the pavement (3.75 feet) and the object on the pavement (0.5 feet) is:

$$\frac{3.75' + 0.5'}{2} = 2.13'$$

By adding an assumed pavement elevation of 100.00 feet to this, an elevation of the line of sight of 102.13 feet is found. From this we obtain the following:

1. Side $AW = 17.6' \times \tan \angle AYW (0.50) = 8.80'$
2. Side $AC = 102.13' - \text{top of curb elevation of } 100.37' = 1.76'$
3. Side $CW = 8.80' - 1.76' = 7.04'$
4. Angle $BXC = 0.025$ (sidewalk slope)
5. Angle $CXW = 0.50 - 0.025 = 0.475$
6. Side $BX = \frac{7.04'}{0.475} = 14.82'$
7. Elevation of point $X = 100.37' + (14.82' \times 0.025) = 100.74'$
8. Side $GX = 102.13' - 100.74' = 1.39'$
9. Side $GY = \frac{1.39'}{\tan \angle AYW (0.50)} = 2.78'$

From the calculations made, the 2:1 slope through the obstruction intersects the sidewalk slope 14.82 feet from the curb. The existing ground at the average height of the line of sight is higher than 102.13 feet at a point 17.6 feet back from the curb. Therefore, where the grade and/or alignment of the street cannot be adjusted, either the slopes have to be graded or a retaining wall must be constructed (whichever is determined to be more economical) to provide this clear distance.

The City's minimum standards for the design of local hillside streets in new subdivisions or in existing streets do not usually require additional easements for sight distance when grading streets and side slopes or constructing a retaining wall at the theoretical grading line. This method should be used also to determine whether sight distance easements are required in improving streets with existing substandard right of way widths or alignments. See Figure E 113, Standard Street Dimensions. The following example illustrates these points, since minimum standards are included in the calculations.

**Given:** Local hillside street with a design speed of 25 miles per hour and a sight distance $S$ of 150 feet, a minimum centerline radius $R$ of 132 feet, a minimum curve length $L$ of 100 feet, a superelevation of 6 percent, and a new subdivision with side slopes graded at a 2:1 slope.

**Find:** The clear horizontal distance $D$ from the inside edge of the inner riding lane to the 2:1 slope at the line of sight, and the sight distance $S$ when the required sight distance is greater than the centerline curve length $L$ of the inner riding lane. See Figure E 342.4C.

1. As in the previous example, the elevation of the average of the eye level of the driver and the height of the object is 102.13 feet. This average elevation is radially opposite the point of obstruction on the centerline of the inner riding lane. Also, the elevation on the pavement at this point is 100.00 feet. Extending the 6 percent superelevation down from the 100-foot elevation to the flow line and adding an 8-inch curb face to this gives a top of curb elevation of:

$$100.00' - 0.06(8' + 5') + 0.67' = 99.89'$$

2. The elevation at the toe of slope, point $A$, is:

$$99.89' + (0.025 \times 5') = 100.02'$$

3. The distance $AC$ is:

$$102.13' - 100.02' = 2.11'$$

4. The horizontal distance $BC$ equals twice $AC$ when there is a 2:1 slope. Therefore:

$$2.11' \times 2 = 4.22'$$

5. When checking a riding lane that is not adjacent to the curb, assume, for calculating purposes, that the inner edge of the inner riding lane is the theoretical curb line and proceed as in the previous example. Therefore, in this case the assumed curb radius is 122 feet, and the clear distance $D$ is:

$$8' + 5' + 4.22' = 17.22'$$

The sight distance provided by these factors should be checked using the formula when $S$ is equal to or less than the length of the curve to see whether the sight distance overlaps the center length of the riding lane.
6. Length $L$ along the centerline of the inner riding lane is:

$$\frac{127.1}{322} \times 100' = 96.21'$$

7. Using the formula:

$$D = \frac{S^2}{8(R+w)} - \frac{W}{2},$$

where

$$S = \sqrt{8\left(D + \frac{W}{2}\right)\left(R + \frac{W}{2}\right)} = 150.25'$$

Since 150.25 feet is greater than 96.21 feet, the sight distance overlaps the centerline length of the inner riding lane. The formula where $S$ is equal to or greater than the curve length should be used instead:

$$D = \frac{2SL - \frac{L^2}{2}}{8(R+w)},$$

where

$$S = \frac{8\left(D + \frac{W}{2}\right)(R + \frac{W}{2}) + \frac{L^2}{2L}}{165.43'}$$

Since 165.43 feet is greater than 150.25 feet, it can be seen that grading the slope at 2:1 or constructing a retaining wall (if more economical) provides a sight distance greater than the required sight distance. Therefore, no additional sight distance easement is required.

However, if it is an existing street with existing improvements, check whether the existing ground can remain undisturbed (no sight distance easement required) and yet provide adequate clear distance $D$. Assume that a wall or other obstruction exists on the theoretical grading line (1 foot in back of the property line). The available clear distance $D$ would then be measured from this grading line to the theoretical curb line (the inner line of the riding lane). Therefore, referring to Figure E 342.4C, $D$ is 13 feet. Using the formula:

$$D = \frac{2SL - \frac{L^2}{2}}{8(R+w)},$$

where

$$S = \frac{8\left(D + \frac{W}{2}\right)(R + \frac{W}{2}) + \frac{L^2}{2L}}{165.43'} = 143.15'$$

The sight distance of 143.15 feet furnished is less than the sight distance of 150.25 feet required. However, there is a sufficient safety factor involved in arriving at the required sight distances for given speeds for most practical purposes. This is true for all minimum standard City streets, as shown on Figure E 113, Standard Street Dimensions. The horizontal sight distance provided by the above conditions obviates the necessity for obtaining sight distance easements on private property. It should also be noted that a car parked on the inside curb lane near the end of the line of sight will reduce the sight distances calculated above by approximately one-third. Since parking restrictions are difficult to enforce on residential streets, the above facts should emphasize the undesirable effects produced by using minimum design standards.

When changes of grade coincide with horizontal curves (either a crest vertical curve or a sag vertical curve with an overhead obstruction such as a bridge), the vertical sight distance is the controlling factor and should exceed the horizontal sight distance requirements. In some cases, despite the use of a minimum horizontal radius of curvature, it may be necessary, in order to improve the line of sight clearance, to cut back the slope or natural growth or, where feasible, remove or modify existing structures.

**E 343 PASSING SIGHT DISTANCE FOR TWO-LANE HIGHWAYS**

On two-lane highways, the overtaking of slower moving vehicles must be accomplished on a lane regularly used by opposing traffic. Therefore, the driver of the overtaking vehicle must see far enough ahead to permit enough time for a safe completion of the passing maneuver. Sufficient distance must be available to enable the passing vehicle to return to the right lane without cutting off the passed vehicle and before meeting the opposing traffic.

The minimum passing sight distance is based on the following assumptions:

1. A single vehicle is passing a single vehicle.
2. The overtaken vehicle travels at a uniform speed.
3. While the passing vehicle is in the opposing lane its average speed will be 10 miles per hour faster than the overtaken vehicle.
4. There is adequate distance between the oncoming vehicle in the opposing lane and the over-
taken vehicle at the time the passing vehicle returns to its own lane.

The minimum length of passing sight distance for a two-lane highway is determined by the sum of the four distances $d_1 + d_2 + d_3 + d_4$, as shown in Figure E 343. The distance-time relationships presented below are based on field observations of driver behavior during passing maneuvers where:

$d_1 =$ the distance traveled during perception and reaction time and during the initial acceleration to the point of encroachment on the left lane.

$d_2 =$ the distance traveled while the passing vehicle occupies the left lane.

$d_3 =$ the distance between the passing vehicle at the end of its maneuver and the opposing vehicle.

$d_4 =$ the distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or two-thirds of $d_2$. For additional information, refer to Section E 020F(1b).

E 343.1 Passing Sight Distance on Crest Vertical Curves: In determining the required passing sight distance on a crest vertical curve, it is assumed that an object being passed on the highway has a height above the pavement of four feet.

Figure E 343.1 shows a series of curves for determining minimum passing sight distance where the sight distance is either greater or less than the length of curve. Formulas for determining this distance are also included on the figure.

Generally, it is impractical to design crest vertical curves to provide for passing sight distance because of the high cost where cuts are involved and the difficulty of fitting the required long vertical curves to the terrain of existing improvements for high-speed roads.

There are no widely used criteria for passing sight distance on sag vertical curves. The controls previously discussed are:

1. Headlight sight distance.
2. Rider comfort.
3. Drainage control.
4. Pleasing appearance.

These are all taken into consideration, with the headlight sight distance control usually prevailing.

E 343.2 Passing Sight Distance on Horizontal Curves: There are few projects in the City in which provisions must be made for passing sight distance on horizontal curves. This situation may occur in the more rural areas, where prevailing traffic density is low and intersections are widely spaced. Where this type of project is encountered, the minimum passing sight distance that must be provided for a two-lane highway is about four times greater than the stopping sight distance for the same design speed. It follows that the clear distance $D$ between the curb line and the obstruction for normal highway cross-sections in cut should be from four to twelve feet greater than those previously presented for stopping sight distance.