PE

CIVIL ENGINEERING

TRANSPORTATION ENGINEERING REVIEW

Robert W. Stokes, PhD, PE & James H. Banks, PhD

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PE EXAM PREPARATION

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Transportation Engineering

James H. Banks

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The Institute of Transportation Engineers (ITE, 1991a) defines transportation engineering as "the application of technology and scientific principles to the planning, functional design, operation, and management of facilities for any mode of transportation in order to provide for the safe, rapid, comfortable, convenient, economical, and environmentally compatible movement of people and goods. Traffic Engineering is that phase of transportation engineering which deals with the planning, geometric design and traffic operations of roads, streets and highways, their networks, terminals, abutting lands, and relationships with other modes of transportation."

While the professional engineer may be involved in the planning, design, operation and/or management of transportation facilities and services, it is typically only the design-related activities which must be supervised and approved by a registered professional engineer. The general focus of this chapter, therefore, is on the design of transportation facilities. Because travel in this country is largely highway oriented, the specific focus of this chapter is the design of streets and highways. This is not to say that the other aspects of transportation engineering (i.e., planning, operations, and management) are any less important than the design function. They are not. The development of a safe, efficient, and economical transportation system requires the incorporation of all of these aspects of transportation engineering in an integrated, systems approach to solving transportation problems. The emphasis of this chapter on the design of transportation facilities reflects the emphasis given this aspect of transportation engineering in most state professional licensing examinations. Given the very special responsibility of the designer with regard to public safety, this emphasis is entirely appropriate.

The material in this chapter is presented under the following major topics: transportation planning; characteristics of the highway system in terms of drivers, vehicles, roadways and traffic flow; statistical methods; traffic studies; route surveying; geometric design guidelines; highway capacity; traffic control devices; signal timing; pavement design; and special topics. Within many of these topics are example problems which illustrate the fundamental principles and concepts outlined in the text. In the case of those design methods which make extensive use of special design charts, such as AASHTO Geometric Design Guidelines, the Highway Capacity Manual, and Pavement Design Methods, the reader is referred to specific references for additional details and example applications.

The chapter includes a list of recommended references for further study and an extensive set of sample problems which illustrate the applications of concepts

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in their own right, are in most cases intermediate steps in arriving at the ultimate goal of the process, which is the estimation of travel volumes on individual roadways and transit routes in the study area, as obtained from the trip assignment phase of the process.

Even a very general description of the calibration and application of travel demand models in a regional (systemwide) context is beyond the scope of this manual. However, the preceding discussion provides a useful introduction to a simplified, but very important and widely used, application of travel demand models. The application alluded to is commonly referred to as the "traffic impact study." The basic steps in conducting a traffic impact study are presented later in this chapter. [The reader should consult Meyer and Miller (2001), Garber and Hoel (1997), and/or the U.S. Department of Transportation (USDOT, 1977) for an introduction to travel demand models.]

HIGHWAY SYSTEM CHARACTERISTICS AND DESIGN CONTROLS

The characteristics of traffic flows on the highway system are the results of complex interactions between the following three basic elements: drivers, vehicles, and the roadway itself. Effective transportation engineering requires an understanding of these three elements and the traffic flows resulting from their interactions.

Driver Performance

The perception of, and reaction to, cues and stimuli encountered by the driver of a vehicle involve four distinct actions: perception, identification, emotion or decision (determination of appropriate response), and volition (reaction). The total time taken for this sequence is referred to as PIEV (perception, identification, emotion, and volition) time or perception-reaction time. Perception-reaction time is an important factor in the determination of braking distance, which in turn dictates minimum sight distance and the length of yellow phase at signalized interactions. The distance traveled during this time is calculated from the following equation:

$$d_p = 0.278vt \qquad \textbf{(1a)}$$

$$d_p = 1.47vt \qquad \textbf{(1b)}$$

where

d_p = perception-reaction distance (m or ft)
v = speed of vehicle (km/hr)
t = perception-reaction time (sec)
0.278 = conversion factor (km/hr to m/sec)
1.47 = conversion factor (mph to ft/sec)

In most situations, a PIEV time of 2.0 to 2.5 seconds is considered realistic. The American Association of State Highway and Transportation Officials (AASHTO) recommends a value of 2.5 seconds for design purposes (AASHTO, 2001). The AASHTO *Policy on Geometric Design of Highways and Streets* (2001) is *the standard reference* in the field of geometric design. It should be one of the reference documents taken to the exam. Key figures and tables from the Green

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where d_p is the PIEV distance as previously defined. Equation (11) may also be used for estimating vehicle speeds in accident investigations. In this application, the braking distance is given as

$$d_b = \frac{V_0^2 - V^2}{254(f \pm G)} \tag{14a}$$

$$d_b = \frac{V_0^2 - V^2}{30(f \pm G)} \tag{14b}$$

where f is the coefficient of friction and V is the velocity at the time of collision (normally not zero).

In accident investigations, transportation engineers are often called upon to estimate the speeds of the vehicles involved in the accident. A typical approach to this problem is to assume that skid marks on the roadway represent braking distance and solve the basic braking distance equation for the unknown initial speed (the final speed is assumed to be zero). The speed estimated in this manner will always be lower than the actual speed because any reduction in speed before skidding and any speed at impact (if there is a collision) will not be reflected in the length of the skid marks.

Highway Functions and Design Elements

This section summarizes some basic terminology and concepts concerning highway functions, sight distances, and the horizontal and vertical alignments of roadways. The roadway characteristics reviewed here are those related primarily to the characteristics of the driver and vehicle. Detailed treatment of specific aspects of the geometric design of streets and highways is also presented in this chapter.

Functional Classification

Streets and highways serve two distinct and very different functions: (1) through movement, or mobility, and (2) land access. Functional classification is the identification of streets and highways in terms of the degree to which the competing and conflicting functions of movement and access are to be served. Three general classifications are commonly employed: arterials, collectors, and local streets.

Roadways are classified according to their function or use and then designed to fulfill that function. Hence, the functional classification of roadways is the initial requirement for design. An understanding of the following aspects of the concept of functional classification is essential to effective design:

1. The relationship between the three primary roadway classifications and the type of service provided is a continuous one. Travel involves movement through a hierarchy of facility types. This means that each functional class should intersect with facilities of the next higher and lower classifications. Failure to recognize and to accommodate this hierarchy by appropriate design is the principal cause of inefficiency in the roadway system.

2. Functional classification is a function of movement versus access, ranging from little or no restriction of access to complete control of access. Counted or projected traffic volume is not an element in functional classification.

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d_2—distance traveled while the passing vehicle occupies the left lane:

$$d_2 = 0.278vt_2 \tag{18a}$$

$$d_2 = 1.47vt_2 \tag{18b}$$

d_3—distance between the passing vehicle and the opposing vehicle at the end of the maneuver. [This clearance interval (d_3) is assumed to vary from 30 m (100 ft) to 75 m (250 ft).]

d_4—distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane (two-thirds of d_2):

$$d_4 = 2d_2/3 \tag{19}$$

where

t_i = time of initial maneuver (sec)
a = average acceleration (km/hr/sec or mph/sec)
v = average speed of passing vehicle (km/hr or mph)
m = difference in speed of passed and passing vehicles (km/hr or mph)
t_2 = time passing vehicle occupies the left lane (sec)

The "average" acceleration results when a linear (constant) relationship between acceleration and velocity is assumed. See Eq. (2).

The design lengths for passing sight distances for various speeds and the corresponding individual values of d_1, d_2, d_3, and d_4 are shown in Exhibit 3-6 (p. 123) and Exhibit 3-7 (p. 124) in the Green Book.

Stopping sight distances are usually sufficient to allow drivers to safely avoid objects in the roadway. However, there are many "busy" locations where longer sight distances may be desirable. In such cases, *decision sight distance* requirements should be considered to provide the greater sight distances that drivers need. Examples of critical locations where decision sight distance considerations apply are

1. Interchanges, particularly where a "left" exit is located
2. Unusual/complex intersection
3. Changes in cross section, such as those at toll plazas and lane drops
4. Locations where significant "visual noise," such as commercial signs, competes for the driver's attention
5. Locations where unusual or expected maneuvers are required

Exhibit 3-3 (p. 116) in the Green Book contains suggested decision sight distances for various conditions. Because decision sight distance give drivers sufficient length to maneuver their vehicles, process additional information, and adjust their speed, its values are substantially greater than stopping sight distance.

Sight distances can be determined in the design phase by measuring both passing and stopping sight distances on roadway plan and profile sheets. Exhibit 3-8 (p. 129) in the Green Book illustrates this procedure for scaling and recording sight distances on plans.

Sight distance depends on the driver's eye height, the height of the object on the road, and the height of obstructions along the roadside. The 2001 Green Book

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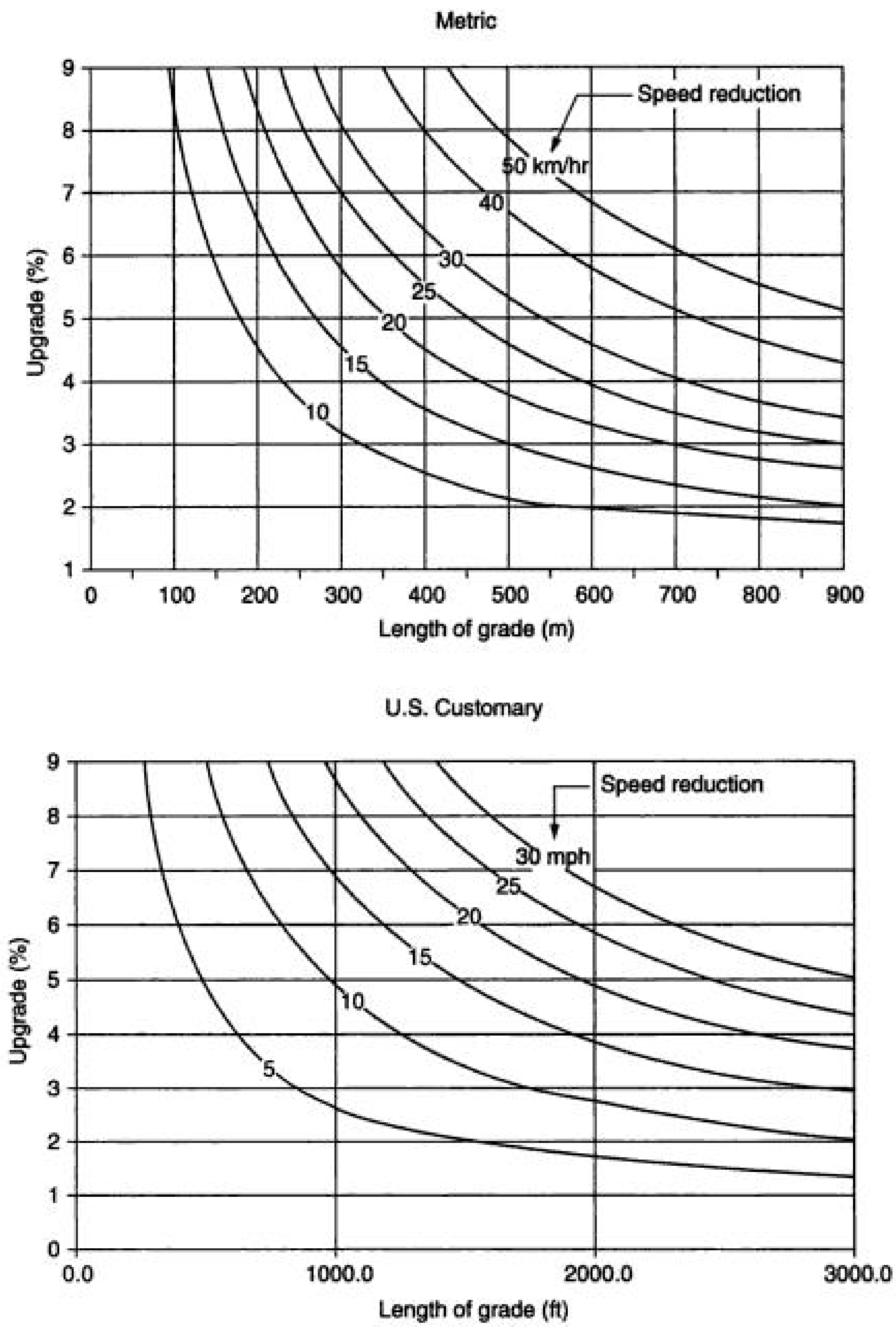

Figure 6 Critical lengths of grade for design, assumed typical heavy truck of 120 kg/kW (200 lb/hp), entering speed = 110 km/hr (70 mpm) [*Source:* AASHTO (2001)].

is less than the length of the curve. The equations for determining minimum lengths for various types of vertical curves are

Minimum length of crest vertical curves with $S < L$:

$$L = (AS^2)/658 \quad \text{[for stopping sight distance (m)]} \tag{23a}$$

$$L = (AS^2)/2158 \quad \text{[for stopping sight distance (ft)]} \tag{23b}$$

$$L = (AS^2)/864 \quad \text{[for passing sight distance (m)]} \tag{24a}$$

$$L = (AS^2)/2800 \quad \text{[for passing sight distance (ft)]} \tag{24b}$$

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Example 5

The following travel times were observed for four vehicles traversing a 1.6 km segment of highway.

Vehicle	Time (min)
1	1.6
2	1.2
3	1.5
4	1.7

The space and time mean speeds of these vehicles are approximately:

(a) $\mu_s = 63.8$ mph, $\mu_t = 60.5$ mph

(b) $\mu_s = 64.2$ mph, $\mu_t = 65.4$ mph

(c) $\mu_s = 63.8$ mph, $\mu_t = 69.2$ mph

(d) $\mu_s = 65.4$ mph, $\mu_t = 62.4$ mph

(e) $\mu_s = 61.6$ mph, $\mu_t = 65.4$ mph

Solution

From Eq. (32), the space mean speed is $\mu_s = (nd)/\Sigma t_i = 4(1.6)/(1.6 + 1.2 + 1.5 + 1.7) = 1.07$ km/min = 64.2 km/hr.

From Eq. (31), the time mean speed is $\mu_t = \Sigma(d/t_i)/n = [(1.6/1.6) + (1.6/1.2) + (1.6/1.5)] + (1.6/1.7)]/4 = 1.09$ km/min = 65.4 km/hr.

The correct answer is (b).

The third measure of traffic stream conditions, density (sometimes referred to as concentration), is the number of vehicles traveling over a unit length of highway at a given time. Density is generally expressed in vehicles per km (vehicles per mile) or vehicles per km per lane (vehicles per mile per lane).

The general equation relating flow, density, and speed is

$$q = k\mu_s \tag{34}$$

where

q = rate of flow (veh/hr)
μ_s = space mean speed (km/hr or mph)
k = density (veh/km or veh/mi)

These three parameters (flow, speed and density) are macroscopic measures in that they apply to the traffic stream as a whole. *Spacing* and *headway*, on the other hand, are microscopic measures because they apply to individual vehicles within the traffic stream. Spacing (or *space headway*) is the distance between vehicles in a traffic stream, and headway (or *time headway*) is the time between successive vehicles as they pass a reference point along the roadway. Some important relationships between these macroscopic and microscopic parameters are shown below:

$$\mu_s = qd \tag{35}$$

$$d = (1/k) \tag{36}$$

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where

$P(A)$ = probability event A will occur

n_A = number of ways A can occur

N = total number of possible outcomes

For example, the probability of being dealt an ace from a newly shuffled deck of 52 cards is 4 (number of aces in the deck) divided by 52 (total number of possible outcomes), or 0.08.

In situations involving the probability of the occurrence of multiple events, the following rules apply.

Rule 1. Addition rule for mutually exclusive events (A or B can occur, but not simultaneously):

$$P(A \text{ or } B) = P(A) + P(B) \tag{49}$$

Rule 2. Multiplication rule for independent events (two events could both occur, but the occurrence of one does not influence the occurrence of the other):

$$P(A \text{ and } B) = P(A)P(B) \tag{50}$$

Rule 3. General addition rule (A or B can occur, but outcomes are not necessarily mutually exclusive):

$$P(A \text{ or } B) = P(A) + P(B) - P(A \text{ and } B) \tag{51}$$

Rule 4. General multiplication rule (probability of two events occurring when they are not independent, i.e., the probability of one event occurring differs depending on whether the other has occurred):

$$P(A \text{ and } B) = P(A)P(B \mid A) \tag{52}$$

where $P(B \mid A)$ is read as "the probability of B given the occurrence of A."

The basic rules of probability outlined above assume that the distribution of the values of the variables in question is known. Usually, this is not the case, and a procedure for computing probabilities when dealing with unknown populations is needed. That is, it would be very useful to be able to write mathematical equations, graphs, or tables to describe the population in question. A number of *theoretical distributions* are often used to describe the possible values of a variable and the probability that each value will occur. The theoretical distributions most commonly used in transportation engineering are (1) the Normal distribution, (2) the Poisson distribution, (3) the Exponential distribution, and (4) the chi-square distribution. [The chi-square distribution is commonly used in statistical tests (goodness-of-fit tests) rather than in traffic models directly. The Exponential distribution is used primarily in microscopic traffic flow models. Therefore, the discussion in the section is limited to the Normal and Poisson distributions.]

The Normal Distribution

The theoretical distribution most frequently used by engineers and scientists in the Normal distribution. Many variables such as heights, test scores, and linear dimensions in general conform to a Normal (or approximately normal) distribution. Many non-Normal distributions can be transformed to induce normality (for example, by

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null hypothesis. The test is performed on the null hypothesis. When we reject the null hypothesis, we accept the alternative hypothesis as being true. In making this decision, we incur two possible types of error: (1) Type 1 error (concluding the hypothesis is false when it is really true), and (2) Type II error (concluding the hypothesis is true when it is really false). These errors are commonly referred to as α and β, respectively (Daniel, 1978). The basic steps in hypothesis testing are outlined below.

Step 1: State the statistical hypotheses. If the parameters of interest are the means of two populations, the following hypotheses could be considered:

$$H_0: \mu_1 = \mu_2, \qquad H_1: \mu_1 \neq \mu_2$$
$$H_0: \mu_1 \leq \mu_2, \qquad H_1: \mu_1 > \mu_2$$
$$H_0: \mu_1 \geq \mu_2, \qquad H_1: \mu_1 < \mu_2$$

The first case is an example of a "two-sided" or "two-tailed" hypothesis. In this case, we are asking, "Can we conclude that the two populations have different means?" If the issue is which population has the larger mean, then the second or third pair of statements would be appropriate. These represents "one-sided" or "one-tailed" hypotheses. In hypothesis testing, the alternative hypothesis is the statement of what we expect to be able to conclude. If the question is whether population 1 has a larger mean than population 2, then H_0: $\mu_1 \leq \mu_2$ would be tested. If this H_0 can be rejected, we accept H_1: $\mu_1 > \mu_2$ as true.

Step 2: Calculate the test statistics. To test the hypothesis, the analyst selects an appropriate test statistics and specifies its distribution when H_0 is true; that is, the test procedure is based on the underlying distribution of the statistics used to estimate the parameters in question. For example, *testing the significance of the difference between means from two independent samples* may be based on the t statistic,

$$t = (\bar{x}_1 - \bar{x}_2) \big/ \left[s_p^2 (1/n_1 + 1/n_2)\right]^{1/2} \qquad \textbf{(69)}$$

where $\bar{x}_1$, $\bar{x}_2$ and n_1, n_2 refer to the means and sample sizes of the two groups, and s_p^2 is obtained by pooling the two sample variances s_1^2 and s_2^2:

$$s_p^2 = \left[(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2\right] / (n_1 + n_2 - 2) \qquad \textbf{(70)}$$

Step 3: State the "decision rule." (The decision rule is usually formulated even prior to stating the statistical hypotheses. Its location in the sequence of steps presented here is largely illustrative.) The issue here is to determine whether the magnitude of the test statistic computed from sample data in step 2 is sufficiently extreme (either too large or too small) to justify rejecting H_0. Two basic approaches are commonly used to formulate the decision rule. In the first, the analyst rejects H_0 if the probability of obtaining a value of the test statistic of a given or more extreme value is equal to or less than some small number α (referred to as the level of significance). Commonly used values for α are 0.10, 0.05, or 0.01. The second approach involves stating the decision rule in terms of critical values of the test statistic. Because critical values are a function of the level of significance, the two approaches are equivalent. Tabulated values for α and the corresponding critical values for commonly used probability distribution can be found in many statistics textbooks.

Step 4: Apply the decision rule. If the probability of obtaining the computed or a larger value of the test statistic is $\leq \alpha$, reject H_0 and conclude that H_1 is true.

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"links," are computed as follows:

$$R_I = (A \times 10^6)/V_I \tag{82}$$

$$R_S = (A \times 10^6)/VMT \tag{83}$$

where

R_I = intersection accident rate (accidents per million vehicles entering the intersection annually)
A = annual number of accidents
V_I = annual traffic entering the intersection
R_S = roadway section accident rate (accidents per million veh-km or veh-mi of travel)
VMT = annual veh-km or veh-mi of travel = AADT × 365 days/year × section length in km or mi

Other accident measures include the severity index (SI) and equivalent property damage only ($EPDO$).

$$SI = \text{number of fatalities/total number of accidents} \tag{84}$$

$$EPDO = PDO + (INJ \times w_I) + (FAT \times w_F) \tag{85}$$

where

PDO = number of property-damage-only accidents
INJ = number of injury accidents
w_I = injury accident weight (cost of injury accidents/cost of PDO accidents)
FAT = number of fatal accidents
w_F = cost of fatal accidents/cost of PDO accidents

The traffic engineer is frequently called upon to identify sites with higher-than-normal numbers of accidents and to design and implement accident reduction measures. One approach to identifying high accident locations is expected value analysis (Garber and Hoel, 1997).

$$EV = \bar{x} \pm Zs \tag{86}$$

where

EV = expected range of accident frequency
$\bar{x}$ = average number of accidents per locations
s = estimated standard deviation of accident frequencies
Z = number of standard deviations corresponding to the required confidence level

Locations with accident frequencies higher than the expected value are considered high accident sites for that specific accident type and should be considered candidates for various accident reduction measures.

Once an accident problem site has been identified, and an improvement has been implemented, the engineer must conduct an evaluation to determine whether or not the improvement has been effective in reducing accidents. In such cases, a before-after analysis is conducted. A procedure commonly used in before-after studies is the test of the significance of differences in means from two independent samples, as described in the previous section. In this procedure, if the absolute value of the difference in the average number of accidents before and after the improvement is $>Zs_p$ (where s_p = the pooled standard deviation of the two samples),

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Step 9: Capacity analysis. The adequacy of the existing street system to accommodate the traffic volumes estimated in step 8 is assessed using standard capacity analysis techniques. The results of the capacity analysis provide the basis for determining any on- and/or off-site improvements that might be needed to accommodate the projected total traffic volumes. These might include adding lanes to existing roadways, adding turn lanes at key intersections and/or at site access points, installing signals and/or other traffic control devices at key intersections, and redesigning traffic signal phasing and timing plans.

The reader is referred to *Travel Estimation Techniques for Urban Planning* (TRB, 1998) for additional information concerning simplified traffic impact analyses.

Example 7

The staff of a local traffic engineering department has estimated that large shopping centers generate an average of 49.64 vehicle trips per day per 1000 sq. ft. of retail floor area. This estimate is based on a sample of 146 shopping center sites. The sample standard deviation is ±22.26 trips. The likelihood that a particular shopping center will generate more than 86 trips per 1000 sq. ft is most nearly:

(a) .01

(b) .05

(c) .10

(d) .25

(e) .50

Solution

From Eq. (66) the 95% confidence interval is

$$\mathrm{CL} = \bar{x} \pm Z\left(s/\sqrt{n}\right) = 49.64 \pm 1.96\left(22.26/\sqrt{146}\right) = 49.64 \pm 3.61.$$

Therefore, there is a 95% chance that the true mean trip rate for such shopping centers is between 46.0 and 53.3 trips.

Equation (66) can be used to assess the likelihood that a shopping center will generate more traffic than 86 trips per 1000 sq. ft (note that this is a "one-tail" situation).

$$x = \bar{x} + Zs = 49.64 + 1.7(22.26) = 86.0$$
$$z = 1.63.$$

In this case it can be concluded that there is only a 1 in 20 chance that a shopping center will generate more than about 86 trips per 1000 sq. ft. The correct answer is (b).

HIGHWAY ROUTE SURVEYING

Highway surveys involve measuring and computing horizontal and vertical angles, elevations, and horizontal distances. The results of these surveys are used to prepare detailed plan and profile base maps of proposed roadways. In addition, the elevations determined in the survey serve as the basis for calculation of

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Concentric Circular Curves

The location and full lengths of inner and outer concentric circular arcs relative to the roadway centerline are often required for reference during roadway construction. These indicate the locations of property lines, curb lines, and/or grading reference points.

Referring to Fig. 10 for the appropriate notation, Hickerson (1964) provides the following basic relationships concerning outer and inner concentric curves:

$$L = R\Delta \tag{98}$$

$$L_o = R_o\Delta \tag{99}$$

or

$$L_o = L + w\Delta \tag{100}$$

or

$$L_o = L + a \tag{101}$$

$$L_i = R_i\Delta \tag{102}$$

or

$$L_i = L - w\Delta \tag{103}$$

or

$$L_i = L - a \quad \text{(Note that } L_o + L_i = 2L\text{, and } L_o - L_i = 2a\text{)} \tag{104}$$

where

Δ = total central angle
L, R = length and radius of the centerline curve
L_o, R_o = corresponding elements of outside curve
L_i, R_i = corresponding elements of inside curve
C = arc (or chord) subtended by θ
C_o, C_i = outside and inside arcs subtended by θ
w = radial distance from center to outer or inner curve
$a = w\Delta = 0.017453w\Delta$ (Δ in degrees)

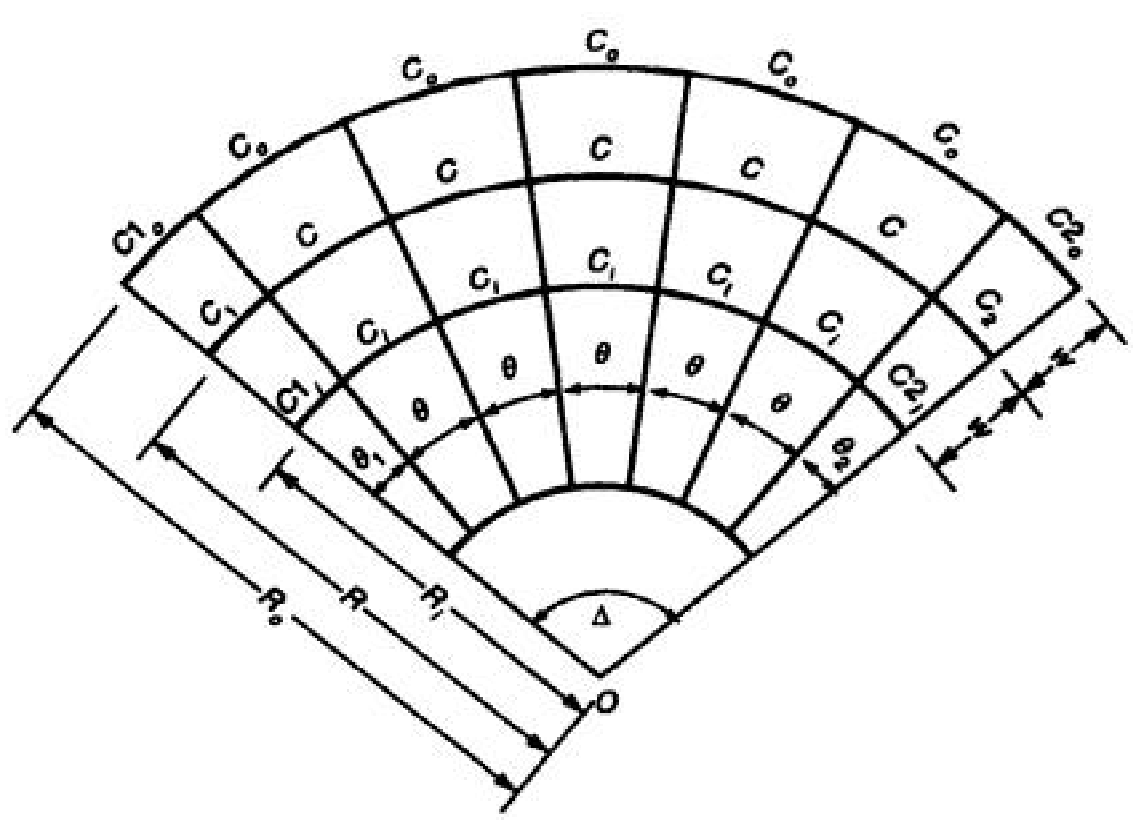

Figure 10 Concentric circular curves

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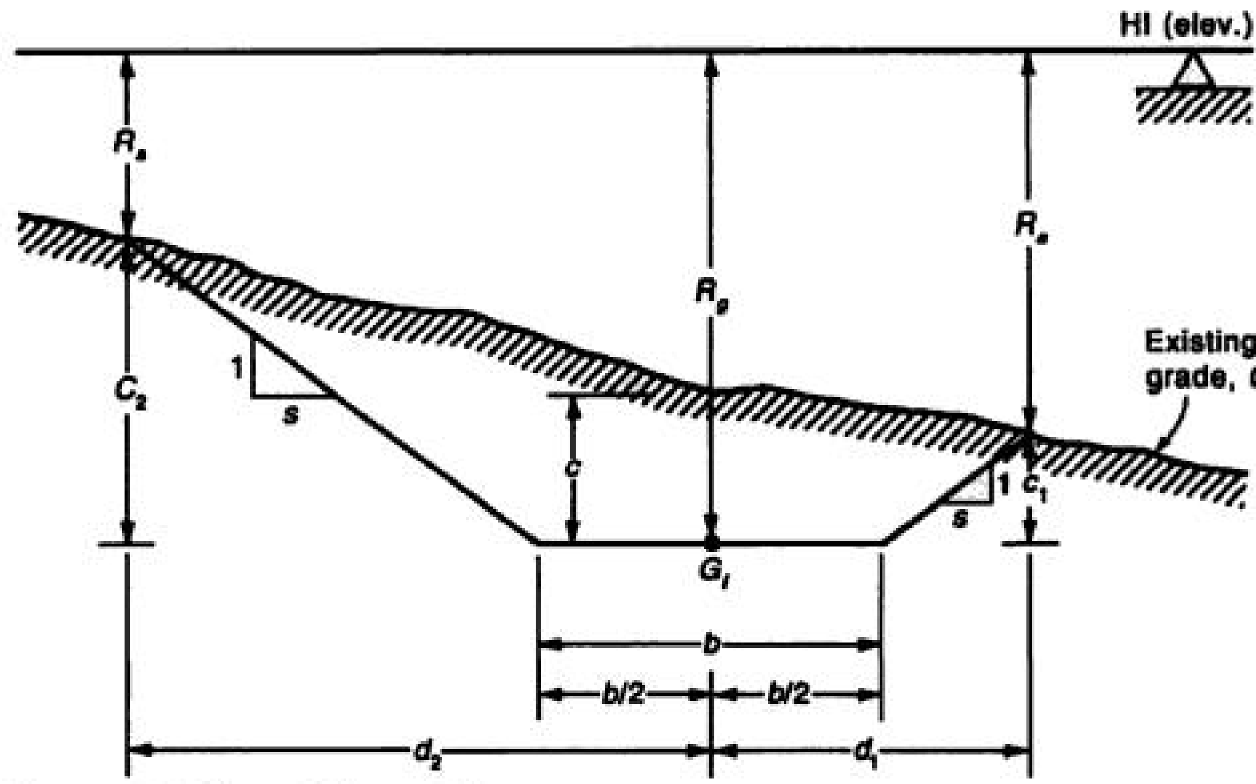

Figure 14 Slope staking notation

where

d = horizontal distance from G_f to the slope stake
b = width of roadbed
S = slope ratio = ratio of horizontal to vertical (+ for cut, – for fill)
C = vertical distance from G_f to the existing grade

The procedure for determining slope stake locations in the field is a matter of trial and error based on known values of S and C at the center of the roadbed. The basic procedure is (Ives and Kissam, 1952)

1. Establish HI (by means of differential leveling from the nearest benchmark).
2. Compute the grade rod, R_g = HI – finished (profile) grade.
3. Record the cut or fill at the centerline of the roadbed ($R_g - R_a$).
4. Locate the rod at the estimated offset for the slope stake. The estimated offset from the centerline is computed as if the ground were level [offset = (b/2) + $S \times$ centerline cut or fill].
5. Take a rod reading at the estimated distance and find the actual cut or fill ($R_g - R_a$). If the actual and estimated values differ by more than about 30 mm (0.10 ft) have the rod held at a greater or lesser offset than for level ground using the following general rule as a guide: On uphill cuts or downhill fills, move the rod away from the centerline; on uphill fills and downhill cuts, move the rod toward the centerline. Repeat the process until d = (b/2) + SC.
6. Set and label the slope stakes. The stakes should be marked with the station number, the offset from the centerline, the cut or fill, and the slope ratio.

Earthwork

One of the major objectives in evaluating alternative route locations is to minimize the amount of cut and fill. To determine the amount of earthwork required for a given alignment, cross sections (such as those developed using the slope staking

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Table 2 AASHTO design guidelines for local roads (*Continued*)

Design Elements	Local Rural Roads	Local Urban Roads
Width	Ex. 5-5, p. 388	≥3 m (10 ft) lanes; 3.3 m (11 ft) design; 3.6 m (12 ft) industrial areas With parking: 2.2 m (7 ft) residential, 2.9 m (9 ft) industrial commercial
Median	n/a	≥12 m (40 ft) left turn radius
Curbs	n/a	100–150 mm (4–6 in.) high
Drainage	n/a	≥0.20% grade
Cul-de-sacs	n/a	10 m (30 ft) radius residential, 15 m (50 ft) commercial Ex. 5-8, p. 399
Alleys	n/a	5–6 m (16–20 ft) wide. Ex. 5-9, p. 401
Sidewalks	n/a	1.2 m (4 ft) residential, 2.4 m (8 ft) commercial
Driveways	n/a	1.0 m (3 ft) return radius
Structures		
New	Ex. 5-6, p. 390	Ex. 5-6, p. 390
In-place	Ex. 5-7, p. 390	Ex. 5-7, p. 390
Vertical clearance	4.3 m (14 ft)	4.3 m (14 ft)
Right-of-way	Variable	Sufficient to accommodate ultimate planned roadway
Foreslopes	1:2 max.	n/a
Horizontal clearance	2–3 m (7–10 ft)	≥0.5 m (1.5 ft)
Intersections		
Angle	60–90°	60–90°
Sight distance	Ch. 9, pp. 655–681 (various cases)	Chapter 9, pp. 655–681 (various cases)
Curb radius	Based on design vehicles	7.5 m (25 ft) (residential), 10 m (30 ft) (commercial)
RR crossings	MUTCD* & Chap. 9	MUTCD & Chap. 9
Traffic control	MUTCD	MUTCD
Street lighting	n/a	Ex. 5-11, p. 406
Comments		

See "Highway System Characteristics and Design Controls" for details in this chapter concerning functional classification, sight distances, and horizontal and vertical alignment design controls.
See Table 7 in this section for guidelines for at-grade intersections.

**Manual on Uniform Traffic Control Devices.*

Collector Roads

Table 3 AASHTO design guidelines for collector roads

Design Elements	Rural Collectors	Urban Collectors
Traffic volumes	20-year design volume	10–20-year design volume
Design speed	70–80 km/hr (45–50 mph) Ex. 6-1, p. 426	≥50 km/hr (30 mph)
Sight distance		
Stopping	Ex. 6-2, p. 426	Ex. 6-2, p. 426
Passing	Ex. 6-3, p. 427	Ex. 6-3, p. 427
Grades	Ex. 6-4, p. 427	Ex. 6-8, p. 436
Crown	High-type pavements: 1.5–2% Low-type pavement: 3–6%	1.5–3.0%
Superelevation	≤0.12; ≤0.8 if snow/ice present	≤0.06
Number of lanes	2	2 + shoulders (use capacity analysi)
Width	Ex. 6-5, p. 429	Residential 3.0–3.6 m (10–12 ft) Industrial 3.6 m (12 ft) Parking: 2.1–3.3 m (7–11 ft)

(*Continued*)

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Table 7 AASHTO design guidelines for at-grade intersections *(Continued)*

Design Elements	At-Grade Intersections
Driveways	Should not be within functional boundary of intersection.
Railroad crossings	Two-crossing event cases. See Ex. 9-103 to -105, pp. 738–742 for required sight distances.
Traffic control	MUTCD* and capacity analysis. See "Highway System Characteristics and Design Control" in this chapter.

* MUTCD, Manual on Uniform Traffic Control Devices.

Grade Separations and Interchanges

Table 8 AASHTO design guidelines for grade separations and interchanges

Design Elements	Grade Separations
Types	Ex. 10-3 to 10-5, pp. 760–762
Warrants	See p. 763–764
Lateral clearance	Four-lane roadways: 3.0 m (10 ft) median. Six or more lanes: 6.6 m (22 ft) (3.0 m shoulders + median barrier)
Vertical clearance	4.4 m (14.5 ft) min.
Horizontal distance required for grade separation	6.0–8.4 m (20–28 ft) difference in elevation. Horizontal distance between roadways varies with design speed, gradient, and rise and fall required. See Ex. 10-8, pp. 772–773
	Interchanges
Types	Seven basic types. See Ex. 10-1, p. 748. Most common type is the Diamond interchange. (See Ex. 10-16 to -18, pp. 783–784, for examples.)
Warrants	Six basic warrants (design designation, congestion relief, elimination of hazard, topography, user benefits, traffic volume). See pp. 749–750.
Spacing	Urban areas: 1.5 km (1 mi). Rural areas: 3 km (2 mi)
Ramps	
Design speed	≥20 km/hr (15 mph). See Ex. 10-56, p. 830
Grades	3–8%. See pp. 832–833
Superelevation	See pp. 834–836
Cross slopes	1.5–2.0%
Pavement width	3.6–13.6 m (12–45 ft). See Ex. 10-67, p. 843.
Lateral clearance	See pp. 842–844 for eight design values.
Spacing	120–600 m (400–2000 ft) See Ex. 10-68, p. 848.
Design	See Exhibits pp. 849–866 for typical ramp designs.
Comments	

See "Highway System Characteristics and Design Controls" for details concerning functional classification, sight distances, and horizontal and vertical alignment design controls.

Roadside Design

The design of safe and efficient roadways must consider the elements of alignment, grade, traveled way, and the roadside environment. Because accidents involving single vehicles running off the road constitute more than 50% of all fatal accidents, the design of the roadside environment is an essential element in the total engineering of the roadway. The guiding philosophy behind roadside design is the provision of a traversable roadside recovery area or *clear zone*, which should be free of obstacles such as unyielding signs and supports, drainage structures, utility poles, and steep slopes. Studies have indicated that on high-speed highways, a

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Table 11b LOS criteria for basic freeway segments, metric units (*Continued*)

Criterion	A	B	C	D	E
Maximum v/c	0.33	0.51	0.74	0.91	1.00
Maximum service flow rate (pc/hr/ln)	770	1210	1740	2135	2350
FFS = 100 km/hr					
Maximum density (pc/km/ln)	7	11	16	22	28
Minimum speed (km/hr)	100.0	100.0	100.0	93.8	82.1
Maximum v/c	0.30	0.48	0.70	0.90	1.00
Maximum service flow rate (pc/hr/ln)	700	1100	1600	2065	2300
FFS = 90 km/hr					
Maximum density (pc/km/ln)	7	11	16	22	28
Minimum speed (km/hr)	90.0	90.0	90.0	89.1	80.4
Maximum v/c	0.28	0.44	0.64	0.87	1.00
Maximum service flow rate (pc/hr/ln)	630	990	1440	1955	2250

Note: The exact mathematical relationship between density and v/c has not always been maintained at LOS boundaries because of the use of rounded values. Density is the primary determinant of LOS. The speed criterion is the speed at maximum density for a given LOS.

Source: TRB (2000).

ratio for each LOS is shown (these values were determined from speed-flow and density-flow relationships similar to those outlined in "Traffic Flow Characteristics and Design Controls" earlier in this chapter). The capacity of basic freeway sections depends on the free-flow speed, with capacities ranging from 2400 passenger cars per hour per lane (pc/hr/ln) for free-flow speeds of 75 mph and 70 mph, down to 2250 pc/hr/ln for free-flow speeds of 55 mph. All values refer to maximum flow rates which can be accommodated in a peak 15-minute period for the given LOS. For each LOS and free-flow speed, the maximum service flow rate (MSF) is computed as capacity times the v/c ratio.

Capacities and maximum service flow rates in Tables 11a and 11b are expressed in terms of equivalent passenger car flow rates. These are calculated from actual hourly volumes by means of

$$v_p = V/(\mathrm{PHF} \times N \times f_{HV} \times f_p) \quad \textbf{(170)}$$

where

v_p = 15-minute passenger-car-equivalent flow rate (pc/hr/ln)
V = hourly volume
PHF = peak hour factor
N = number of lanes
f_{HV} = heavy-vehicle adjustment factor
f_p = driver population factor

The heavy vehicle factor, in turn, is calculated as

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1)] \quad \textbf{(171)}$$

where P_T and P_R are the proportions of trucks or buses and recreational vehicles in the traffic stream, and E_T and E_R are passenger car equivalents for trucks or buses and recreational vehicles. Values of f_p, E_T, and E_R are tabulated in Chapter 23 of the HCM.

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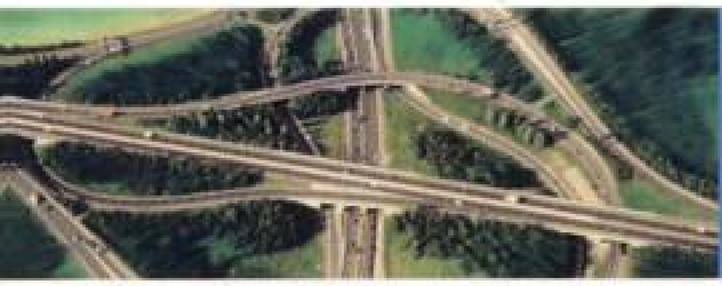

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