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3. Banking or Superelevation
4. Sight and Stopping Distances
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6. Vertical Curve Lengths
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1. Definitions and Acronyms

   AASHTO

   HCM

   CBD
2. Vehicle Dynamics

Newton's second law of motion

\[ F_n = ma \]

\[ m = \frac{w}{g} \]

Net tractive force

\[ F_n = F_w - F_f \]

Net force

\[ F_n = \frac{(550)(\text{horsepower})}{V_{\text{ft/sec}}} \]
Relationship between position, velocity, and acceleration for linear motion as a function of time:

\[ a = \frac{dv}{dt} = \frac{d^2s}{dt^2} \]

\[ v = \frac{ds}{dt} = a \int dt \]

\[ s = \int v \, dt = \int \int a \, dt^2 \]
Example - 4000 pound car at 80 mph skids 580 ft before stopping

a) Time to stop?

Initial velocity?

\[ v_o = (80) \left( \frac{5280}{3600} \right) = 117.3 \text{ ft/sec} \]

\( s = 580 \text{ ft and final velocity is } v = 0 \)

\[ t = \frac{2s}{v_o - v} = \frac{(2)(580)}{(117.3 - 0)} = 9.89 \text{ sec} \]
b) Acceleration?

\[ a = \frac{v - v_0}{t} = \frac{0 - 117.3}{9.89 \text{ sec}} = -11.9 \text{ ft/sec}^2 \]

c) Retarding force?

\[ F_f = ma = \left( \frac{4000 \text{ lb car}}{32.2 \text{ ft/s}^2} \right) (11.9) = 1480 \text{ lbf} \]

d) Coefficient of friction?

\[ u = \frac{F_f}{N} = \frac{1480}{4000} = 0.37 \]
SITUATION 101

Two cars are traveling 60 mph in the same lane in the same direction, 200 ft. apart on a level roadway. The leading car hits a disabled milk truck and comes to an instantaneous stop. The driver of the trailing car, whose perception and reaction time is 0.5 sec, locks the brakes and skids into the lead car. The coefficient of friction between the roadway and skidding tires is 0.6.
(a) How far does the trailing car skid after the brakes lock?

Car speed in common units:

\[ v_0 = \frac{(60 \text{ mi/hr}) \cdot \frac{(5280 \text{ ft})}{\text{mi}}}{(3600 \text{ sec/hr})} = 88 \text{ ft/sec} \]

Distance trailing car has traveled by the time its driver has perceived the danger and applied the brakes:

\[ s_1 = v_0 t = (88 \text{ ft/sec})(0.5 \text{ sec}) = 44 \text{ ft} \]

Total skidding distance is:

\[ s_{skid} = 120 \text{ ft} - s_1 = 120 \text{ ft} - 44 \text{ ft} = 76 \text{ ft} \]
(b) What is the speed of the trailing car at the moment it collides with the stationary lead car?

Frictional force on the trailing car depends on the coefficient of friction and the normal force, which is equal to the car's weight. It is negative because it opposes the existing motion:

\[ F_{\text{friction}} = -f \quad N = -f \quad W = -0.6mg \]

Use Newton's Second Law to find the deceleration of the trailing car during the skid:

\[ a = \frac{F_{\text{friction}}}{m} = 0.6 \frac{mg}{m} = (0.6)(32.2 \text{ft/sec}^2) = 19.32 \text{ft/sec}^2 \]

Use the uniform acceleration theory to compute the velocity of the car at the moment of impact:
\[ v = \sqrt{v_0^2 + 2as} = \sqrt{(88 \text{ ft/sec})^2 + (2)(-19.32 \text{ ft/sec}^2)(76 \text{ ft})} = 69.3 \text{ ft/sec} \]

(c) If the trailing car is able to skid past (not into) the stationary lead car, what length skid marks would it leave?

Uniform acceleration theory predicts skid distance as:

\[ s = \frac{v^2 - v_0^2}{2a} = \frac{(0)^2 - (88 \text{ ft/sec})^2}{(2)(19.32 \text{ ft/sec}^2)} = 200 \text{ ft} \]
3. Banking or Superelevation

\[ e + f = \frac{V^2}{15R} \]

e = rate of superelevation (ft/ft) \approx 0.08 \text{ to } 0.12
f = side friction factor
V = velocity (mph)
R = radius of curve (ft)
4. **Sight and Stopping Distances**

Sight distance - length of roadway the driver can see

Stopping sight distance - distance allowing the driver travelling at maximum speed to stop before coming upon an observed object

Driver eye height assumed to be 3.5 feet above roadway surface and object height assumed to be 6 inches

Stopping sight distance equation

\[
S = (1.47)(t_p)(V) + \frac{(V^2)}{(30)(f/G)}
\]

- **S** = stopping sight distance (ft)
- **t_p** = braking reaction-perception time (sec)
- **V** = velocity (mph)
- **f** = coefficient of friction
- **e** = rate of superelevation (ft/ft)
- **G** = gradient (ft/ft)
5. Circular Curve Lengths
Horizontal Curve Equations

\[ D = \text{Degree of Curve, Arc Definition} \]
\[ 1^\circ = \text{1-Degree of Curve} \]
\[ 2^\circ = \text{2-Degree of Curve} \]
\[ \text{P.C.} = \text{Point of Curve (also called B.C.)} \]
\[ \text{P.T.} = \text{Point of Tangent (also called E.C.)} \]
\[ \text{P.I.} = \text{Point of Intersection} \]
\[ I = \text{Intersection of Angle (also called } \Delta \text{); Angle between two tangents.} \]
\[ L = \text{Length of Curve, from P.C. to P.T.} \]
\[ T = \text{Tangent Distance} \]
\[ E = \text{External Distance} \]
\[ R = \text{Radius} \]
\[ \text{L.C.} = \text{Length of Long Chord} \]
\[ M = \text{Length of Middle Ordinate} \]
\[ c = \text{Length of Sub-Chord} \]
\[ d = \text{Angle of Sub-Chord} \]
Horizontal Curve Equations

\[ R = \frac{L.C.}{2 \sin (I/2)}; \quad T = R \tan (I/2) = \frac{L.C.}{2 \cos (I/2)} \]

\[ \frac{L.C.}{2} = R \sin (I/2); \quad D 1^\circ = R = 5,729.58; \quad D 2^\circ = \frac{5,729.58}{2} \]

\[ M = R [1 - \cos (I/2)] = R - R \cos (I/2); \quad D = \frac{5,729.58}{R} \]

\[ \frac{E + R}{R} = \sec (I/2); \quad \frac{R - M}{R} = \cos (I/2) \]

\[ c = 2R \sin (d/2); \quad d = \frac{c}{2R} \]

\[ L.C. = 2R \sin (I/2); \quad E = R [\sec (I/2) - 1] = R \sec (I/2) - R \]
Example:

Given:
- \( l = 35^\circ \)
- \( R = 1000' \)
- P.I. Station = 50+00

Calculate:
- \( (1/2)l = \frac{35}{2} = 17.5^\circ \)
- \( D = \frac{5729.58}{1000} = 5.7296^\circ \)
- \( T = 1000 \tan(17.7) = 315.30' \)
- \( L = 100 \left( \frac{35}{5.7296} \right) = 610.86' \)
- P.C. Station = (50+00) - (3+15.30) = 46+84.7
- P.T. Station = (46+84.7) + (6+10.86) = 52+95.56

\[
\begin{align*}
1st s &= (47+00) - (46+84.7) = 15.3' \quad \text{ahead of P.C.} \\
1st d &= (0.3)(15.3)(5.7296) = 26.298' = 0^\circ-26'-19.8'' \\
1st c &= 2(1000)(\sin 0.4383) = 15.3'
\end{align*}
\]
full station $s = 100'$
full station $d = (0.3)(100)(5.7296) = 171.88' = 2^\circ-51'-53''$
full station $c = (2)(1000)(\sin 2.7376) = 99.96'$

last $s = (52+95.56) - (52+00) = 95.56'$
last $d = (0.3)(95.56)(5.7296) = 164.522' = 2^\circ-44'-16''$
last $c = (2)(1000)(\sin 2.7376) = 95.52'$
**Field Notes:**

<table>
<thead>
<tr>
<th>Station</th>
<th>Chord</th>
<th>$\delta$</th>
<th>Defl. Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC = 46+84.70</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>47+00</td>
<td>15.30'</td>
<td>0°26'19&quot;</td>
<td>0°26'18&quot;</td>
</tr>
<tr>
<td>48+00</td>
<td>99.86'</td>
<td>2°51'53&quot;</td>
<td>3°18'12&quot;</td>
</tr>
<tr>
<td>49+00</td>
<td>99.86'</td>
<td>2°51'53&quot;</td>
<td>6°10'05&quot;</td>
</tr>
<tr>
<td>50+00</td>
<td>99.86'</td>
<td>2°51'53&quot;</td>
<td>9°01'58&quot;</td>
</tr>
<tr>
<td>51+00</td>
<td>99.86'</td>
<td>2°51'53&quot;</td>
<td>11°53'51&quot;</td>
</tr>
<tr>
<td>52+00</td>
<td>99.86'</td>
<td>2°51'53&quot;</td>
<td>14°45'44&quot;</td>
</tr>
<tr>
<td>PT = 52+95.56</td>
<td>95.52'</td>
<td>2°44'16&quot;</td>
<td>17°30'00&quot; = $\frac{1}{2}I$</td>
</tr>
</tbody>
</table>

**Figure 25-5. Curve layout by deflection angles.**
6. Vertical Curve Lengths

Length of Crest Vertical curves

a. Stopping sight distance
b. Rider comfort \( L = \frac{AV^2}{46.5} \)

Length of Sag Vertical curves

a. Headlight beam distance
b. Rider comfort
c. Drainage
d. General Appearance
Stopping Sight Distance

A. For $S<L$

\[ L = \frac{AS^2}{100(\sqrt{2H_1} + \sqrt{2H_2})^2} \]

B. For $S>L$

\[ L = 2S - \frac{200(\sqrt{H_1} + \sqrt{H_2})^2}{A} \]

where

$L =$ length of vertical curve  
$A =$ algebraic difference in grades (%)  
$S =$ sight distance (braking or passing)  
$H_1 =$ height of drivers’eyes above the roadway surface $= 3.5 \text{ ft}$  
$H_2 =$ height of object above the roadway surface $= 0.5 \text{ ft}$
Vertical Curve Formulas

\[ L = \text{Length of Curve (horizontal)} \]
\[ g_2 = \text{Grade of Forward Tangent} \]
\[ PVC = \text{Point of Vertical Curvature} \]
\[ a = \text{Parabola Constant} \]
\[ PVI = \text{Point of Vertical Intersection} \]
\[ y = \text{Tangent Offset} \]
\[ PVT = \text{Point of Vertical Tangency} \]
\[ E = \text{Tangent Offset at PVI} \]
\[ g_1 = \text{Grade of Back Tangent} \]
\[ r = \text{Rate of Change of Grade} \]
\[ x = \text{Horizontal Distance from PVC to Point on Curve} \]
\[ x_m = \text{Horizontal Distance to Min/Max Elevation on Curve} = -\frac{g_1}{2a} \]

\[ y = ax^2; \quad a = \frac{g_2 - g_1}{2L} \]
\[ E = a\left(\frac{L}{2}\right)^2; \quad r = \frac{g_2 - g_1}{L} \]

\[ \text{Tangent Elevation} = Y_{PVC} + g_1x \quad \text{and} \quad = Y_{PVI} + g_2(x - L/2) \]
\[ \text{Grade Elevation} = Y_{PVC} + g_1x + ax^2 \]
Example: \( L = 800' = 8 \) stations
G1 = +2%
G2 = -3%
Sta. VPC = 800+00
Elev (vpc) = 1000'
A = (-3%) - (+2%) = -5%

<table>
<thead>
<tr>
<th>Station</th>
<th>x</th>
<th>G1x</th>
<th>( \frac{(Ax^2)}{(2L)} )</th>
<th>Elev of x (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>800+00</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1000.00</td>
</tr>
<tr>
<td>801+00</td>
<td>1</td>
<td>2</td>
<td>-0.313</td>
<td>1001.69</td>
</tr>
<tr>
<td>802+00</td>
<td>2</td>
<td>4</td>
<td>-1.25</td>
<td>1002.75</td>
</tr>
<tr>
<td>803+00</td>
<td>3</td>
<td>6</td>
<td>-2.8125</td>
<td>1003.19</td>
</tr>
<tr>
<td>804+00</td>
<td>4</td>
<td>8</td>
<td>-5</td>
<td>1003.00</td>
</tr>
<tr>
<td>805+00</td>
<td>5</td>
<td>10</td>
<td>-7.813</td>
<td>1002.19</td>
</tr>
<tr>
<td>806+00</td>
<td>6</td>
<td>12</td>
<td>-11.25</td>
<td>1000.75</td>
</tr>
<tr>
<td>807+00</td>
<td>7</td>
<td>14</td>
<td>-15.313</td>
<td>998.69</td>
</tr>
<tr>
<td>808+00</td>
<td>8</td>
<td>16</td>
<td>-20</td>
<td>996.00</td>
</tr>
</tbody>
</table>

Location of high point on curve: \( x' = \frac{(8)(2)/[2-(-3)]}{2} = 3.2 \) stations
Station of high point: \( (800+00) + (3+20) = 803+20 \)
Elevation of high point: \( 1000' + (2)(3.2) + (-5)(3.2^2)/(2)(8) = 1003.2' \)
A 400-ft-long, equal-tangent vertical sag curve connects two tangents: a -2.5% downward (left-to-right) tangent and a +2% upward (left-to-right) tangent. The tangents intersect (PVI) at station 425+25 at an elevation of 945.31 ft. The width of the paved surface is 43 ft. The pavement has an average positive slope of 1.6% (0.015 ft/ft) from centerline (crown) of pavement to gutter.
**REQUIREMENT**

(a) What is the station of the beginning of the vertical curve (BVC)?

**SOLUTION**

(a) \[ x_{BVC} = x_{PVI} - \frac{LC}{2} = 426.25 \text{ sta} - \frac{4 \text{ sta}}{2} = 424.25 \text{ (sta 24+25)} \]

(b) What is the elevation of the BVC?

(b) The elevation of the BVC is

\[ y_{BVC} = y_{PVI} - \frac{g_1 x_{PVI} - LC}{2} = 945.31 \text{ ft} - \left(-2.5 \frac{\text{ ft}}{\text{ sta}}\right) \left(\frac{4 \text{ sta}}{2}\right) = 930.31 \text{ ft} \]

(c) What is the station of the end of the vertical curve (EVC)?

(c) As in (a),

\[ x_{EVC} = x_{PVI} + \frac{LC}{2} = 428.25 \text{ sta} + \frac{4 \text{ sta}}{2} = 428.25 \text{ (sta 428+25)} \]

(d) What is the elevation of the EVC?

(d) The elevation of the EVC is

\[ y_{EVC} = y_{PVI} + \frac{g_2 x_{EVC}}{2} = 945.31 \text{ ft} + \left(2.0 \frac{\text{ ft}}{\text{ sta}}\right) \left(\frac{4 \text{ sta}}{2}\right) = 949.31 \text{ ft} \]

(e) What is the station of the lowest point on the curve?

(e) The rate of grade change per station is

\[ r = \frac{g_1 - g_2}{LC} = \frac{2 \frac{\text{ ft}}{\text{ sta}} - (-2.5 \frac{\text{ ft}}{\text{ sta}})}{4 \text{ sta}} = 1.125 \text{ ft/sta}^3 \]

The distance from the BVC to the lowest point is...
$z_{BVC\text{-}\text{lowest point}} = -\frac{g_1}{r} = \frac{-(-2.5 \text{ ft})}{1.125 \text{ ft}^2} = 2.22 \text{ sta}

The station of the lowest point is

$z_{\text{lowest point}} = z_{BVC} + z_{BVC\text{-}\text{lowest point}} = 424.25 \text{ sta} + 2.22 \text{ sta}

= \boxed{426.47 \text{ sta}}$

(f) What is the elevation of the lowest point on the curve?

(f) Measuring horizontal distances from the BVC, the equation of the centerline of this curve is

$y = \frac{1}{2} x^2 + g_1 \cdot z + y_{BVC}

= \frac{1.125}{2} x^2 + (-2.5)x + 550.31$

Substituting $x = 2.22 \text{ sta}$ as the distance to the lowest point,

Lowest point = \( \left( \frac{1.125}{2} \right)(2.22)^2 + (-2.5)(2.22) + 550.31 = \boxed{947.53 \text{ ft}} \)

(g) What is the elevation of a gutter drain inlet located at the side of the road at the lowest point on the curve?

(g) The paved surface slopes off from the centerline (crown) to the edge. The elevation of the edge is assumed to correspond to the gutter elevation.

$y_{gutter} = y_{\text{centerline}} - \frac{w}{2} \cdot \frac{w}{2}$

= 947.53 ft $- \left( \frac{0.016 \text{ ft}}{\text{ft}} \right) \left( \frac{48 \text{ ft}}{2} \right)$

= 947.15 ft
7. Speed Parameters

Running speed - distance traveled divided by the running time without delays. (Averaged for all traffic)

Average spot speed - average instantaneous speed of all vehicles at a particular location.

Overall travel speed - distance traveled divided by the running time, including delays. (Averaged for all traffic)

Operating speed - highest overall speed at which a driver can travel under favorable weather conditions while driving in a safe manner.

Design speed - maximum safe speed when conditions are so favorable that the design features of the highway govern.

Average highway speed - weighted average of the design speeds over a section of highway.
8. Design Speeds

Roadway design elements depend on design speed.

Maximum safe maintainable speed under design conditions.

Varies by type of facility, traffic volume, and terrain.
9. Volume Parameters

ADT - average daily traffic

AADT - average annual daily traffic

DHV - design hourly volume (30th highest hourly volume during a year)

K - ratio of DHV to AADT (generally, range from 0.10 to 0.12)

D - (directional factor) the percentage of traffic in the dominant flow direction

DDHV - directional design hourly factor = (D)(DHV) = (D)(K)(AADT)

Design capacity - maximum volume of traffic the roadway can handle

MSF - maximum service flow rate per lane under ideal conditions
    (pcphpl: passenger cars per hour per lane)
10. Passenger Car Equivalents

Accounts for differences in size and performance characteristics of buses, trucks, and recreational vehicles.

<table>
<thead>
<tr>
<th>Terrain</th>
<th>(E_B) (buses)</th>
<th>(E_T) (trucks)</th>
<th>(E_R) (RV's)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>1.5</td>
<td>1.7</td>
<td>1.6</td>
</tr>
<tr>
<td>Rolling</td>
<td>3</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Mountainous</td>
<td>5</td>
<td>8</td>
<td>4</td>
</tr>
</tbody>
</table>

Source: AASHTO PGDHS-1990, Table II-4
11. Capacity and Level of Service

Capacity - maximum flow rate under prevailing conditions

Level of service (LOS)- method for quantifying quality of service on highways, defined by density (passenger cars per mile lane)

LOS varies from A (free flow) through F (stop-and-go)

\[ k = \text{density (passenger cars per mile per lane, pc/mi/ln)} \]

*Figure 9.1 Speed-Flow Relationship for Multilane Facilities under Ideal Conditions*
12. Freeway Capacity

\[ c = \text{maximum capacity under ideal conditions} \]

\[ c_{\text{max}} = \text{MSF per lane for a given level of service} \]

volume to capacity ratio for a given LOS \( l \):

\[ \left( \frac{V}{C} \right)_i = \frac{MSF_i}{c_{\text{max}}} \]
Calculation of Service Flow Rate:

\[ SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_p \]

\[ SF_i = c_{max} \times \left( \frac{V}{C} \right)_i \times N \times f_w \times f_{HV} \times f_p \]

SF\(_i\) = service flow rate (passenger cars per hour)
N = number of lanes
f\(_w\) = width adjustment factor
f\(_{HV}\) = heavy vehicle factor
f\(_p\) = population adjustment factor
13. Speed, Flow, and Density

density (D) - number of vehicles per mile per lane

jam density ($D_j$) - density when all vehicles are at a standstill

Speed for a given density is given by:

$$S = S_r \left(1 - \frac{D}{D_j}\right)$$

$S_r =$ free-flow speed

headway - elapsed time, $t$, between the front bumper of one vehicle and the front bumper of the following vehicle passing a given point in the roadway, (sec)

spacing - distance between common points (e.g., the front bumper) on successive vehicles, (ft/veh)
NOTE: FLOW RATE $V_1$ OCCURS UNDER TWO DIFFERENT FLOW CONDITIONS, ILLUSTRATED AS A AND B.
Figure 16.5 Freeway Speed-Flow Relationships Under Ideal Conditions
(v/c ratio is based on 2000 pcp/hpl and is valid only for 60- and 70-mph speeds.)

Figure 16.6 Freeway Density-Flow Relationships Under Ideal Conditions
(v/c ratio is based on 2000 pcp/hpl and is valid only for 60- and 70-mph speeds.)
14. Determining Level of Service

Depends on density criteria, BUT criteria varies depending on freeway's design speed

<table>
<thead>
<tr>
<th>Level of service</th>
<th>V/C Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design Speed</td>
</tr>
<tr>
<td></td>
<td>70 mph</td>
</tr>
<tr>
<td>A</td>
<td>0.35</td>
</tr>
<tr>
<td>B</td>
<td>0.54</td>
</tr>
<tr>
<td>C</td>
<td>0.77</td>
</tr>
<tr>
<td>D</td>
<td>0.93</td>
</tr>
<tr>
<td>E</td>
<td>1.00</td>
</tr>
</tbody>
</table>
SITUATION 103

A six-lane freeway with 10-ft lanes has a 70-mph design speed while passing through rolling terrain. The few sections that have grades greater than 3% are less than 1/4 mile long each. The clear distance between overpass abutments and roadside obstacles is 2 ft for both roadside and median edges. The directional weekday peak-hour volume is 2400 vehicles, and an average of 750 vehicles pass during the most congested 15-min peak-hour period. The traffic stream consists of 14% trucks, 8% buses, and 4% recreational vehicles.
REQUIREMENT

Note: Base your answers on the 1985 edition of the
Highway Capacity Manual (HCM).

(a) How many lanes of traffic travel in each direction?
   (A) 1    (B) 2    (C) 3
   (E) 4    (E) 6

SOLUTION

(a) Three lanes of traffic travel in each direction on a
six-lane freeway. \( N = 3 \).  
   \[ \boxed{C} \]

(b) What is the approximate service flow rate?
   (A) 750 vehicles per hour
   (B) 1500 vehicles per hour
   (C) 2400 vehicles per hour
   (D) 3000 vehicles per hour
   (E) 4500 vehicles per hour

(b) The service flow rate, \( SF \), is the flow rate during the
peak 15-minute period expressed in vehicles per hour.

\[ SF = 4V_s = (4)(750 \text{ vph}) = 3000 \text{ vph} \]
   \[ \boxed{D} \]

(c) What is the approximate heavy vehicle correction factor?
   (A) 0.5    (B) 0.6    (C) 0.7
   (D) 0.8    (E) 0.9

(c) The heavy vehicle correction factor, \( f_{HV} \), is found
in two steps. First, the passenger car equivalent, \( E \), for
each truck, bus, and recreational vehicle is found. Since
the terrain has no extended grades greater than \( \frac{3}{4} \) mile,
HCM Table 3-3 can be used. For rolling terrain,

\[
\begin{align*}
\text{trucks:} & \quad E_T = 4.0 \\
\text{buses:} & \quad E_B = 3.0 \\
\text{RV’s:} & \quad E_R = 3.0 \\
\end{align*}
\]

Next, the heavy vehicle correction factor, \( f_{HV} \), is cal-
lculated from HCM Eq. 3-4. The \( P \)'s are the fractions
(not percentages) of each of the respective traffic types.

\[
f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)} \\
= \frac{1}{1 + (0.14)(4 - 1) + (0.08)(3 - 1) + (0.04)(3 - 1)} \\
= 0.602 \\
\]
   \[ \boxed{B} \]

(d) What is the approximate volume-to-capacity ratio?
   (A) 0.71    (B) 0.79    (C) 0.86
   (D) 0.92    (E) 0.98
(d) From HCM Table 3-1 for a 70-mph design speed, the maximum capacity, $c_f$, under ideal conditions is 2000 pcp/hpl (passenger cars per hour per lane).

From HCM Table 3-10, the factor to adjust for the driver population, $f_p$, on weekdays is 1.00.

From HCM Table 3-2, the lane width adjustment factor, $f_w$, for three 10-ft lanes in each direction with 2 ft of clearance on each side of the roadway is 0.85.

The volume-to-capacity ratio, $v/c$, is calculated from HCM Eq. 3-3.

$$v/c = \frac{SF}{c_f \times N \times f_w \times f_{HV} \times f_p}$$

$$= \frac{3000 \text{ vph}}{(2000 \text{ pcp/hpl})(3 \text{ lanes})(0.85)(0.602)(1.0)}$$

$$= 0.977$$

(e) At which level of service is the freeway operating?

(A) A  (B) B  (C) C
(D) D  (E) E

(e) From HCM Table 3-1, the maximum volume-to-capacity ratio for level of service D is 0.53, so the level of service is E.

(E)

(f) What would the volume-to-capacity ratio have to be in order to improve the level of service to the next lower capacity level of service?

(A) 0.35  (B) 0.54  (C) 0.77
(D) 0.93  (E) 1.00

(f) Since the maximum volume-to-capacity ratio for level of service D is 0.53, this is the improved value.

(D)

Questions (g) through (j) assume maximum freeway capacity during the peak hour.

(g) At which level of service is the freeway operating?

(A) A  (B) B  (C) C
(D) D  (E) E

(g) The level of service is always E when the freeway is operating at capacity.

(E)

(h) What is the approximate service flow rate at this level of service?

(A) 2000 vehicles per hour  (B) 3000 vehicles per hour
(C) 3500 vehicles per hour  (D) 4500 vehicles per hour
(E) 6000 vehicles per hour

(h) The service flow rate for level of service E, $SF_E$, is calculated using HCM Eq. 3-3. All factors are the same, except that the volume-to-capacity ratio is 1.0 since the freeway is operating at capacity.

$$SF_E = (v/c)_E \times c_f \times N \times f_w \times f_{HV} \times f_p$$

$$= (1.00)(2000 \text{ pcp/hpl})(3 \text{ lanes})(0.85)(0.602)(1.0)$$

$$= 3070 \text{ vph}$$

(B)
(i) What is the approximate peak-hour factor?
(A) 0.6  (B) 0.7  (C) 0.8  
(D) 0.9  (E) 1.0

(i) Use HCM Eq. 2-1 to calculate the peak-hour factor, PHF.
\[ \text{PHF} = \frac{V}{SF} = \frac{2400 \text{ vph}}{3000 \text{ vph}} = 0.8 \]

\[ \boxed{C} \]

(j) What is the actual peak hour volume?
(A) 2450 vehicles per hour.
(B) 2700 vehicles per hour
(C) 2950 vehicles per hour
(D) 3250 vehicles per hour
(E) 4700 vehicles per hour

(i) The volume during the peak hour is
\[ V_E = \text{PHF} \times SF_E = (0.8)(3070 \text{ vph}) = 2456 \text{ vph} \]

\[ \boxed{A} \]
15. Signalized Intersection Capacity
Depends on many factors --
- width of approach -- number of lanes
- parking conditions
- traffic direction
- environment
- bus and truck traffic
- percentage of turning traffic

Capacity can be calculated by:

\[ c = s \times \frac{g}{C} \]

- \( C \) = Capacity
- \( S \) = Ideal Saturation Flow Rate
- \( g/C \) = ratio of green time to cycle length

where:

\[ s = n \cdot s_0 \cdot N \cdot f_w \cdot f_{HV} \cdot f_g \cdot f_p \cdot f_{bb} \cdot f_a \cdot f_{RT} \cdot f_{LT} \]
16. Standard Truck Loadings

Various separation between axles

Types of axles

All loads are axle loads

16 STANDARD TRUCK LOADINGS

Table 16.12 and figure 16.8 illustrate standard truck loads commonly used for design. In cases where the separation between axles varies, the distance that produces the maximum stress in the section should be used.
Except in theoretical stress studies, no attempt is made to account for the number of tires per axle. Although it is true that stresses at shallow depths are caused principally by individual wheels acting singly, stresses at greater depths are maximum midway between wheels. Deep stresses due to dual-wheeled axles are approximately the same as for single-wheeled axles. Therefore, required pavement thickness is determined by the total axle load.\textsuperscript{11}
AASHTO Design Vehicles for Road Geometry

Passenger Car - P

Single Unit Truck - SU

Bus - Bus

Intermediate length semitrailer - WB-40

Large semitrailer - WB-50

Semitrailer/full trailer combination - WB-60
Standard Truck Loads Used for Design

Table 16.12
Standard Truck Loadings
(All loads are axle loads)

<table>
<thead>
<tr>
<th>Load designation</th>
<th>$F_1$</th>
<th>$F_2$</th>
<th>$F_3$</th>
<th>$d_1$</th>
<th>$d_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H20–44</td>
<td>8000</td>
<td>32,000</td>
<td>0</td>
<td>14'</td>
<td></td>
</tr>
<tr>
<td>H15–44</td>
<td>6000</td>
<td>24,000</td>
<td>0</td>
<td>14'</td>
<td></td>
</tr>
<tr>
<td>H10–44</td>
<td>4000</td>
<td>16,000</td>
<td>0</td>
<td>14'</td>
<td></td>
</tr>
<tr>
<td>HS20–44</td>
<td>8000</td>
<td>32,000</td>
<td>32,000</td>
<td>14'</td>
<td>14' to 30'</td>
</tr>
<tr>
<td>HS15–44</td>
<td>6000</td>
<td>24,000</td>
<td>24,000</td>
<td>14'</td>
<td>14' to 30'</td>
</tr>
<tr>
<td>3</td>
<td>16,000</td>
<td>17,000</td>
<td>17,000</td>
<td>15'</td>
<td>4'</td>
</tr>
<tr>
<td>3S2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the purposes of designing road geometry, AASHTO defines design vehicles according to the following categories: passenger car (P), single unit truck (SU), single unit bus (BUS), intermediate length semitrailer (WB-40), large semitrailer (WB-50), and semitrailer/full trailer combination (WB-60). In the case of W3 vehicles, the number represents the wheelbase distance between the front (cab) axle and the last trailer axle. [AASHTO PGDHS–1990, Table II-1]

Classification into axle types also is common. An axle may be single or tandem, and each axle may have single or dual tires. Spread tandem axles, where two axles are separated by more than 96 inches, generally are classified as two single axles.
17. Pavement Types

Rigid pavement - Portland cement concrete

Prestressed concrete pavement

Flexible asphalt concrete

Full-depth asphalt pavement

Deep-lift asphalt pavement
Rigid Pavement

Typical Applications

Advantages and Disadvantages

Reinforced/Unreinforced

AASHTO specifications
AASHTO Specifications

Amount of longitudinal steel

\[ A_s = \frac{F_{lw}}{2f_s} \]

Spacing and sizing for round steel dowels
Flexible Asphalt Concrete

Typical uses

Advantages/Disadvantages
18. Asphalt Binders

Become less dependent on imported oil

30% to 40% sulfur content

Silicone added for workability

Standard equipment

Superior quality
19. Minimum Layer Thicknesses

1.5" - 2" for asphalt surface course

4" for cement-, lime- and asphalt-treated bases and subbases
20. Pavement Design Parameters

Layer strengths

Equivalent axle loadings

Predicting traffic growth
Converting Strength Parameters for Layer Strengths

Soil Support Value - Appendix A

Appendix A: Approximate Correlation between California Bearing Ratio and Subgrade Modulus
Converting Strength Parameters for Layer Strengths

Soil Support Value - Appendix B

Appendix B: Revised Soil Support Correlations
Converting Strength Parameters for Layer Strengths

Soil Support Value - Appendix C

Appendix C: Approximate Correlation between Subgrade Modulus and Soil R-value
Equivalent Axle Loadings

*Method A:* Use actual loadometer data

*Method B:* Convert other equivalent axle loads to 18-kip loads

\[ EAL_{18\ kips} = EAL_{n\ kips} \left( \frac{nkips}{18} \right)^4 \]

*Method C:* Convert EAL data from other durations

\[ EAL_{20\ years} = EAL_{n(years)} \left( \frac{20}{n(years)} \right) \]
Equivalent Axle Loadings (continued)

*Method D:* Approximate equivalent 18 kip axle loads from # of trucks passing/day

\[
EAL_{18\text{kip}, \text{20 years}} = 1380(2 \text{ axle}) + 3680(3 \text{ axle}) + 5880(4 \text{ axle}) + 13,780(5 \text{ axle})
\]

*Method E:* Convert 5000 pound EWL to 18-kip EAL

\[
EAL = \frac{EWL}{11.8}
\]

*Method F:* Convert design index to 20-year EAL

*Method G:* Convert traffic index to 20-year EAL
Predicting Traffic Growth

20-year EAL, constant growth rate of i%

\[ EAL_{20} = EAL_{\text{first year}} \times (F/A, i\%, 20) \]
21. AASHTO Flexible Pavement Design

Step 1: Estimate terminal serviceability, $p_t$

<table>
<thead>
<tr>
<th>Condition</th>
<th>$p_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>very poor</td>
<td>0-1</td>
</tr>
<tr>
<td>poor</td>
<td>1-2</td>
</tr>
<tr>
<td>fair</td>
<td>2-3</td>
</tr>
<tr>
<td>good</td>
<td>3-4</td>
</tr>
<tr>
<td>very good</td>
<td>4-5</td>
</tr>
</tbody>
</table>

Step 2: Equivalent 18-kip single axle loads
<table>
<thead>
<tr>
<th>Step 3: Determine regional factor</th>
<th>( R )</th>
<th>condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.2-1.0</td>
<td>Roadbed frozen to depth of 5&quot; or more</td>
</tr>
<tr>
<td></td>
<td>0.3-1.5</td>
<td>Roadbed dry, summer and fall; no winter freezing</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>Sandy desert</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>Roadbed subject to frost, fairly dry</td>
</tr>
<tr>
<td></td>
<td>4.0-5.0</td>
<td>Roadbed wet, spring break-up thaw, high water table, soil saturated</td>
</tr>
</tbody>
</table>
### Step 4: Soil support value

<table>
<thead>
<tr>
<th>AASHTO Soil Group</th>
<th>CTB, BTB</th>
<th>400 and up</th>
<th>250 and up</th>
<th>300 and up</th>
<th>175-325</th>
<th>200-325</th>
<th>200-300</th>
<th>50-175</th>
<th>50-225</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1-a</td>
<td>sand, clay gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
<tr>
<td>A-1-b</td>
<td>silt, silty clay</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
<tr>
<td>A-2-6, A-2-7</td>
<td>gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
<tr>
<td>A-3</td>
<td>gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
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<tr>
<td>A-4</td>
<td>gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
<tr>
<td>A-5</td>
<td>gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
<tr>
<td>A-6</td>
<td>gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
<tr>
<td>A-7-5, A-7-6</td>
<td>gravel</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td>50-225</td>
<td></td>
</tr>
</tbody>
</table>
Step 5: Determine structural number

Determine the structural number from the terminal serviceability ($p_t$), 20-year, 18-kip equivalent axle loading (EAL), soil support value (S), and regional factor (R) from figure 16.10 or figure 16.11. If the structural number differs from what was used in step 2, repeat from step 2.

* Total equivalent 20 year load = daily equivalent load x 365 x 20 years
Step 6: Determine layer coefficients

(Strength Coefficients)

subbase coefficient, $a_3$
- sandy gravel 0.11
- sand, sandy clay 0.05–0.10
- lime-treated soil 0.11
- lime-treated clay, gravel 0.14–0.18

base coefficient, $a_2$
- sandy gravel 0.07
- crushed stone 0.14
- cement treated base (CTB)
  - $f'_{c,7-day} > 650$ psi 0.23
  - 400–650 0.20
  - < 400 0.15
- bituminous treated base (BTB)
  - coarse 0.34
  - sand 0.30
- lime treated base 0.15–0.30
- soil cement 0.20
- lime/fly ash base 0.25–0.30

surface course coefficient, $a_1$
- plant mix 0.44
- road mix 0.20
- sand asphalt 0.40
Step 7: Write layer thickness equation

\[ t_1 a_1 + t_2 a_2 + t_3 a_3 = SN \]

Example 16.5

It is desired to verify the adequacy of a flexible pavement design for a well-traveled highway in California.
- \( R \) value of basement soil: 10
- subbase: aggregate, thickness 17"
- base: aggregate, thickness 12"
- asphalt concrete: thickness 8"

The surface is expected to carry the following traffic.
- 2-axle trucks: ADT = 935
- 3-axle trucks: ADT = 550
- 4-axle trucks: ADT = 225
- 5-axle trucks: ADT = 1025

Step 1: Assume a terminal serviceability of \( p_t = 2.5 \).

Step 2: Use equation 16.32.

\[ F.AL = [1380](935) + [3680](550) + [5880](225) \]
\[ + (13,780)(1025) \]
\[ = 18.8 \text{ EE5} \]

Step 3: Choose \( R = 1 \) for all of California.

Step 4: From appendix B, the soil support value, \( S \), is approximately 3 for an \( R \) value of 10.

Step 5: From figure 16.11, the required structural number is approximately 6.

Step 6: Use the following layer coefficients: \( a_3 = 0.11, a_2 = 0.14, a_1 = 0.44 \).

Step 7: Calculate the actual structural number from equation 16.35.

\[ (0.44)(8) + (0.14)(12) + (0.11)(17) = 7.67 \]

Since 7.67 is greater than 6, the pavement has adequate strength.
The following parameters dictated the original design of an existing flexible pavement according to AASHTO procedures:

- initial structural number \((SN_0)\): 3.5
- equivalent repetitions: \(1 \times 10^6\)
- reliability: 95%
- overall standard deviation: 0.45
- roadbed soil modulus \((M_R)\): 8 psi
- design serviceability loss: 2.2

After 900,000 repetitions, the effective structural number of the pavement will be 2.6. A flexible plant-mixed overlay will then be applied to extend the life of the road surface, designed for \(2 \times 10^6\) repetitions in the overlay period, with the remaining structural capacity to support \(4 \times 10^6\) total repetitions before the pavement needs to be replaced.
REQUIRED

(a) What criterion is normally used to determine when a pavement needs to be replaced?

(A) The serviceability is reduced to approximately 2.
(B) The level of service drops to D or below.
(C) The maximum safe speed drops to 55 mph.
(D) The ride quality drops to 50% of the original value.
(E) There are more than 3 pavement defects per 100 ft.

412 SOLUTION

Note: This solution is based on the AASHTO Guide for Design of Pavement Structures.

(a) A terminal serviceability of 2 (and sometimes 2.5 for major highways) is the standard criterion point for pavement replacement.

(b) What is a standard load repetition?

(A) passage of an axle with single or dual wheels
(B) passage of an axle carrying 18,000 pounds
(C) passage of one vehicle weighing 5000 pounds
(D) passage of four wheels
(E) passage of one-half of a tandem axle

(b) The equivalent axle load is 18,000 pounds (18 kips).

(c) What is the remaining life ratio of the flexible pavement just before the overlay is applied?

(A) 0.01
(B) 0.1
(C) 0.33
(D) 0.5
(E) 0.9

(c) The remaining life ratio prior to the overlay is

\[ R_{Lx} = 1 - d_x = 1 - \frac{900,000}{1,000,000} = 1 - 0.9 = 0.1 \]

(d) What is the remaining life ratio of the pavement at the end of the overlay period?

(A) 0.01 (B) 0.1 (C) 0.5

(d) The remaining life ratio at the end of the overlay period is

\[ R_{Ly} = 1 - d_y = 1 - \frac{2 \times 10^6}{4 \times 10^6} = 1 - 0.5 = 0.5 \]

(e) What is the remaining life factor?

(A) 0.10 (B) 0.20 (C) 0.50

(e) The remaining life factor is 0.50.
(e) The remaining life factor is found [Guide, Vol. 2, Fig. CC.11] from the two remaining life factors. For $R_{Lx} = 0.1$ and $R_{Ly} = 0.5$, the remaining life factor, $F_{RL}$, is approximately 0.67.

(f) Assuming no change in the reliability, standard deviation, effective roadbed resilient modulus, and design serviceability loss, what total structural number is required to achieve a terminal serviceability of 2.0 at the end of the overlay period?

(A) 2  
(B) 3  
(C) 4  
(D) 5  
(E) 6

(f) Although the method of constructing the pavement is different, the procedure for calculating the structural number is no different for an overlay than for a new pavement. The total structural number, $S_{Ny}$, is found graphically to be approximately 4 [Guide, Vol. 1, Fig. 3.1].

(g) What additional structural number must be provided by the overlay?

(A) 0.6  
(B) 1.4  
(C) 2.3  
(D) 2.9  
(E) 4.0

(g) If the existing structural number is 2.6 and the required structural number is 4, the overlay must provide an additional structural number of

\[
S_{N_{OL}} = S_{Ny} - (F_{RL})(S_{N_{ref}}) = 4 - (0.67)(2.6) = 2.26
\]

Notice that the overlay would have to provide only 4 - 2.6 = 1.4 if the remaining life had not been specified.

(h) Approximately what is the conversion from structural number to inches of pavement thickness?

(A) 0.05 SN/in  
(B) 0.11 SN/in  
(C) 0.23 SN/in  
(D) 0.30 SN/in  
(E) 0.44 SN/in

(h) For a plant-mix surface course, a layer coefficient, $a$, of 0.44 is commonly used.

(i) Using AASHTO design procedures, what overlay thickness is required?

(A) 2.5 in  
(B) 3.5 in  
(C) 5.0 in  
(D) 7.5 in  
(E) 10.0 in

(i) Only one layer of overlay will be used. Therefore, the design criterion is

\[
a_{OL}h_{OL} = S_{N_{OL}}
\]

The overlay layer will be

\[
h_{OL} = \frac{S_{N_{OL}}}{a_{OL}} = \frac{2.26}{0.44} = 5.14 \text{ in}
\]
(j) When the pavement needs to be replaced (has reached its design terminal serviceability), it will

(A) be criticized by most people riding on it.
(B) not support axle loadings greater than 12 kips.
(C) support fewer than 100,000 additional repetitions before the subgrade fails.
(D) have a structural number of 0.
(E) have remaining life ratio of 0.1.

(i) People "know" when a pavement needs to be fixed. They can feel the bumps and see the dislocations. Serviceability is determined subjectively. A terminal serviceability of 2 means that approximately 85% of the people think the pavement should be repaired or replaced.
22. CALTRANS Flexible Pavement Design

Step 1. Determine 18-kip EAL for the surface

Step 2. Calculate traffic index, $T_I$

$$T_I = (9.0)(EAL/10^6)^{0.119}$$

Step 3. Choose base material

Step 4. Calculate total gravel equivalent, $G_E$

$$G_E = 0.0032(T_I)(100\, R)$$

Step 5. Calculate layer thickness

$$t_{layer} = \frac{G_E}{G_f} \text{ (feet)}$$
Step 6. Select subbase material

<table>
<thead>
<tr>
<th>class</th>
<th>% passing # 4 sieve</th>
<th>R-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30–75</td>
<td>60</td>
</tr>
<tr>
<td>2</td>
<td>35–95</td>
<td>50</td>
</tr>
<tr>
<td>3</td>
<td>45–100</td>
<td>40</td>
</tr>
</tbody>
</table>

Step 7. Determine thickness of base

*Step 7:* Determine the thickness of the base. Calculate the total gravel equivalent from the R-value of the subbase and equation 16.37. Subtract the surface layer's gravel equivalent from the total gravel equivalent. Calculate the base's thickness from equation 16.38.

\[
GE = 0.0032(TI)(100 - R) \\
\text{16.37}
\]

Step 8. Determine thickness of subbase

*Step 8:* Determine the thickness of the subbase. Calculate the total gravel equivalent from the R-value of the basement soil and equation 16.37. Subtract the base's and surface layer's gravel equivalents (corresponding to their actual thicknesses) from the total gravel equivalent. Calculate the subbase's thickness from equation 16.38.

\[
t_{\text{layer}} = \frac{GE}{G_f} \text{ (in feet)} \\
\text{16.38}
\]
23. Asphalt Institute Flexible Pavement Design

Step 1: Determine 20-year, 18-kip EAL

Step 2: Convert EAL to traffic class

Step 3: Classify the subgrade soil

<table>
<thead>
<tr>
<th>category</th>
<th>resilient modulus$^{28}$</th>
<th>CBR</th>
<th>R-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>poor</td>
<td>4500 psi</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>medium</td>
<td>12,000</td>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>good-to-excellent</td>
<td>25,000</td>
<td>17</td>
<td>43</td>
</tr>
</tbody>
</table>

Step 4: Choose base and subbase materials

Step 5: Design the pavement

Table 16.22 or 16.23
24. Concrete Pavements

Figure 16.13
25. AASHTO Rigid Pavement Design

Step 1: Select terminal serviceability, $p_t = 2.5$ for highways

Step 2: Determine 20-year, 18-kip EAL

Step 3: Select subbase material

<table>
<thead>
<tr>
<th>type</th>
<th>description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>open graded</td>
</tr>
<tr>
<td>B</td>
<td>dense graded</td>
</tr>
<tr>
<td>C</td>
<td>cement treated (CTB)</td>
</tr>
<tr>
<td>D</td>
<td>lime treated (LTB)</td>
</tr>
<tr>
<td>E</td>
<td>bituminous treated (BTB)</td>
</tr>
<tr>
<td>F</td>
<td>granular</td>
</tr>
</tbody>
</table>

Step 4: Determine modulus of subgrade reaction

Step 5: Determine modulus of elasticity

Step 6: Determine allowable working stress

$$ f_t = \frac{M_R}{F_S} $$
Step 7: Read the slab thickness

Step 8: Check for steel reinforcement
26. Fatigue Strength Method of Design

Step 1. Proposed pavement design

Step 2. Axle loads from loadometer survey

Step 3. Multiply by load safety factor if desirable

Step 4. Modulus of subgrade reaction, $k_s$

Step 5. Modulus of reaction, $k_{sb}$, on subgrade soil and subbase

Step 6. Modulus of reaction on the base
Step 7. Stress induced in the pavement

Step 8. 28-day modulus of rupture

\[ \frac{M_R}{M_C} = \frac{I}{L} \]

Step 9. Divide stress value by modulus of rupture
Step 10. Determine fatigue life

Step 11. Determine fatigue fraction

Step 12. Determine fraction of fatigue strength used
27. Roadway Detailing

Lane widths - 12 feet freeways, 11 feet restricted areas

Crown slope

Portland cement concrete - 2%

Bituminous mix pavement - 2%

Treated earth and gravel - 2.5 to 3%

Unsurfaced, graded - 2.5 to 3.0%

Shoulders - 6 to 10 feet on right and 5 to 8 feet on left

Shoulder slope - 5% away from median
Maximum grade - 3% for freeways, 6% for other roads, and 2% steeper for rugged terrain

Side slopes on adjacent cuts - 2:1 for freeways, and 1.5:1 for other roads

Cut-to-right-of-way clearance - 10 to 50 feet

Divided median width - 30 to 46 feet, 4-foot minimum

Horizontal clearance to piers and walls - 10 to 30 feet

Vertical clearance - 16.5 feet for structures, 18 feet for signs, 18.5 feet for pedestrian overcrossings
28. Pavement Joints

Control (contraction) joints

Construction (contact) joints

Isolation (expansion) joints

Dowel bars

Hinge (warping) joints
29. Grooving Pavements

Increasing skid resistance and reducing hydroplaning

Requires structurally adequate pavement

Should be continuous
30. Geotextiles

Support and filter fabrics to stabilize and retain soil
31. Subgrade Drainage

High groundwater levels

Subgrade soils which become spongy when saturated

Seeping water from underlying strata

Cuts that interrupt natural drainage

Sag curves with low permeability subgrade
31. Frost Damage

Techniques for reducing damage

Constructing stronger pavement sections

Lower the water table by added drains

Layers of coarse sand or waterproof sheets

Removing frost susceptible materials

Rigid foam sheets to insulate and reduce frost penetration
33. Parking Design

Width of stall
8' common
>9' market areas
12' can be traffic lane when necessary

Length of stall
18' minimum

Parking lot capacity
\[ \text{capacity} = \frac{\text{lot area}}{320} \]

Diagonal Parking
45°, 60°, 75°, 90°

Parallel Parking
34. Intersection Signaling

Conditions requiring signaling

High traffic volume
Interuption of traffic
High crosswalk usage
Nearby school crossing
Need to regulate speed
Excessive accident activity
Need to combine two roads

Signal controllers

Fixed-time
On-demand (traffic actuated)
Determining Fixed-Time Cycle Lengths

**Step 1.** Determine highest volume lanes for x-direction and y-direction

**Step 2.** Determine # of car equivalents

\[ E = \# \text{ cars} + (1.5)(\# \text{ buses}) + (1.5) (\# \text{ trucks}) + (1.6) (\# \text{ left turning veh}) \]

**Step 3.** Determine cycle length from table

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<tr>
<th>X-dir</th>
<th>100</th>
<th>200</th>
<th>300</th>
<th>400</th>
<th>500</th>
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<th>800</th>
<th>900</th>
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<tbody>
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</table>
Step 4. Determine proportional split times

\[ \text{Green+Amber time}_{(x \text{ direction})} = \frac{E_x}{(E_x + E_y)} \times \text{Cycle length} \]

X time = \(\frac{300}{(300+800)}\) \times 90 \text{ sec} = 24.5 \text{ sec}
Y time = \(\frac{800}{(300+800)}\) \times 90 \text{ sec} = 65.5 \text{ sec}

Step 5. Specify Amber time
3 to 6 seconds recommended

Step 6. Provide Red-Clearance
1 to 2 seconds recommended

Step 7. Check pedestrian requirements
5 seconds + 4 \text{ ft/sec} \times \text{width of intersection (ft)}
Determining On-Demand Timing

Initial period
\[
\text{# of cars in initial period} = \frac{\text{distance between line and detector}}{20}
\]

Vehicle period
\[
\text{time between when vehicle crosses the detector until it reaches the intersection}
\]

Maximum period
\[
\text{max delay opposing traffic can tolerate}
\]
\[
60 \text{ sec typical for main street}
\]
\[
30 - 40 \text{ sec for side street}
\]

Yellow period
\[
\text{time required to perceive, brake, and stop the vehicle}
\]

Green period
\[
\text{smaller of (initial + vehicle periods) or maximum period}
\]
Time-space diagrams

Step 1. Choose a scale -- 1" = 100' or 1' = 200' typical

Step 2. Draw the main and intersecting streets to scale

Step 3. Assume/obtain average travel speed along the main street

Step 4. Assume cycle length
   \(2 \times \text{travel time between average intersection spacing}\)

Step 5. Check cycle length against heaviest flow intersection
   cycle length table

Step 6. Calculate split times
   based on equivalent volumes

Step 7. Graphically determine offsets due to travel time between
   intersections
35. Interchange Design
   Diamond interchanges
   Cloverleaf interchanges
Directional interchanges
36. Pedestrian Levels of Service

Primary criterion -- Space

Pedestrian Levels of Service in Walkways and Queuing Areas

<table>
<thead>
<tr>
<th>LOS</th>
<th>walkways space (ft²/ped)</th>
<th>flow rate (ped/min-ft)</th>
<th>queuing areas area (sq ft/person)</th>
<th>queuing areas spacing (ft)</th>
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</thead>
<tbody>
<tr>
<td>A</td>
<td>≥130</td>
<td>≤2</td>
<td>≥13</td>
<td>≥4</td>
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<td>B</td>
<td>≥40</td>
<td>≤7</td>
<td>10-13</td>
<td>3.5-4.0</td>
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<td>C</td>
<td>≥24</td>
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<td>7-10</td>
<td>3.0-3.5</td>
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<td>2.0-3.0</td>
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<td>E</td>
<td>≥6</td>
<td>≤25</td>
<td>2-3</td>
<td>≤2</td>
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<tr>
<td>F</td>
<td>≥6</td>
<td>--</td>
<td>≤2</td>
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37. Economic Justification

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<td>Fatality</td>
<td>330,000</td>
<td>287,000</td>
<td>1,000,000</td>
<td>3,500,000</td>
<td>2 to 5,000,000</td>
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<tr>
<td>Non-fatal Injury</td>
<td>3,400</td>
<td>81</td>
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<td></td>
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<tr>
<td>Property Damage</td>
<td>480</td>
<td>520</td>
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</tbody>
</table>
38. Queueing Models

*Can be used to predict:*
- length of waiting time
- average time customer will spend in queue
- probability that a given number of customers will be in the queue

**Nomenclature**

- $L$: expected system length (includes service)
- $L_q$: expected queue length
- $p\{n\}$: probability of $n$ customers in the system
- $s$: number of parallel servers
- $W$: expected time in the system (includes service)
- $W_q$: expected time in the queue
- $\lambda$: mean arrival rate
QUEUING MODELS

General Relationships

\[ L = \lambda \times W \]

\[ L_q = \lambda \times W_q \]

\[ W = W_q + \frac{1}{\mu} \]

\[ \lambda < \mu \times s \]

average service time = \( 1 / \mu \)

average time between arrivals = \( 1 / \lambda \)
QUEUING MODELS

The M/M/1 System

assumptions

There is only one server (s=1)

The calling population is infinite

The service times are exponentially distributed with mean μ

\[ p(t > h) = e^{-\mu h} \]

The arrival rate is distributed as Poisson with mean λ

\[ p(x) = \frac{e^{-\lambda} \lambda^x}{x!} \]
QUEUING MODELS

The M/M/1 System -- Relationships

\[ p(0) = 1 - \rho \]

\[ L = \frac{\lambda}{\mu - \lambda} = L_q + \rho \]

\[ W = \frac{1}{\mu - \lambda} = W_q + \frac{1}{\mu} = \frac{L}{\lambda} \]

\[ p(n) = p(0) \cdot \rho^n \]
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Assumed Speed for Condition (mph)</th>
<th>Brake Reaction Time (sec)</th>
<th>Distance (ft)</th>
<th>Coefficient of Friction $f$</th>
<th>Braking Distance on Level (ft)</th>
<th>Stopping Sight Distance Computed (ft)</th>
<th>Rounded for Design (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>20-20</td>
<td>2.5</td>
<td>73.3-73.3</td>
<td>0.40</td>
<td>33.3-33.3</td>
<td>106.7-106.7</td>
<td>125-125</td>
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<td>25</td>
<td>24-25</td>
<td>2.5</td>
<td>88.0-91.7</td>
<td>0.38</td>
<td>50.5-54.8</td>
<td>138.5-146.5</td>
<td>150-150</td>
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<td>2.5</td>
<td>102.7-110.0</td>
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<td>74.7-85.7</td>
<td>177.3-195.7</td>
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</tr>
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<td>117.3-128.3</td>
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<td>100.4-120.1</td>
<td>217.7-248.4</td>
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<tr>
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<td>36-40</td>
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<td>0.32</td>
<td>135.0-166.7</td>
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<td>212.7-256.7</td>
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<td>400.5-583.3</td>
<td>613.1-840.0</td>
<td>625-850</td>
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</tbody>
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Table III-1. Stopping sight distance (wet pavements).
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<tr>
<th>Design Speed (mph)</th>
<th>Maximum e</th>
<th>Maximum f</th>
<th>Total (e + f)</th>
<th>Maximum Degree of Curve</th>
<th>Rounded Maximum Degree of Curve</th>
<th>Radius (ft)</th>
</tr>
</thead>
<tbody>
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NOTE: In recognition of safety considerations, use of e_max - 0.04 should be limited to urban conditions.
*Calculated using rounded maximum degree of curve.

Table III-6. Maximum degree of curve and minimum radius determined for limiting values of e and f, rural highways and high-speed urban streets.
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