قسم
الهندسة المدنية
المرحلة الرابعة

Transportation Engineering

Lectures

2017 – 2018

هندسة المواصلات

P-1

Lec.
Dr. Ahmed Mancy

5500

كلية المنصور الجامعة

Relationships among units and important values

1 ft = 0.3 m 100 ft = 30 m

1 mile (mi) = 1609 m = 1.6 km 1 mile = 5280 ft

****

Distance = ut u: speed t: time

Distance (m) = 0.278 ut u: speed in km/hr t: time in seconds

0.278 = 1000/3600 = 1000 m / 3600 second (to absorb the difference between speed in (km/hr) and time in (second) and distance in (meters)

Distance (m) = 1.47 ut u: speed in mi/hr t: time in seconds

1.47 = 5280/3600 = 5280 ft / 3600 second (to absorb the difference between speed in (mi/hr) and time in (second)and distance in (feet)

= =

$g = 9.81\ m/sec^2 = 32.2\ ft/sec^2$

254 = 2 x 9.81 /(0.278 x 0.278)

D_b(m), u (km/hr), f(unit-less), G(unit-less)

$$D_b = \frac{u^2}{254(f \pm G)}$$

30 = 2 x 32.2 /(1.47 x 1.47)

D_b (ft), u (mi/hr), f(unit-less), G(unit-less)

$$D_b = \frac{u^2}{30(f \pm G)}$$

****

127 = 9.81 /(0.278 x 0.278)

Db (m), u (km/hr), f_s(unit-less), e (unit-less)

$$R = \frac{u^2}{127(e + f_s)}$$

15 = 32.2 /(1.47 x 1.47)

Db (ft), u (mi/hr), f_s (unit-less), e (unit-less)

$$R = \frac{u^2}{15(e + f_s)}$$


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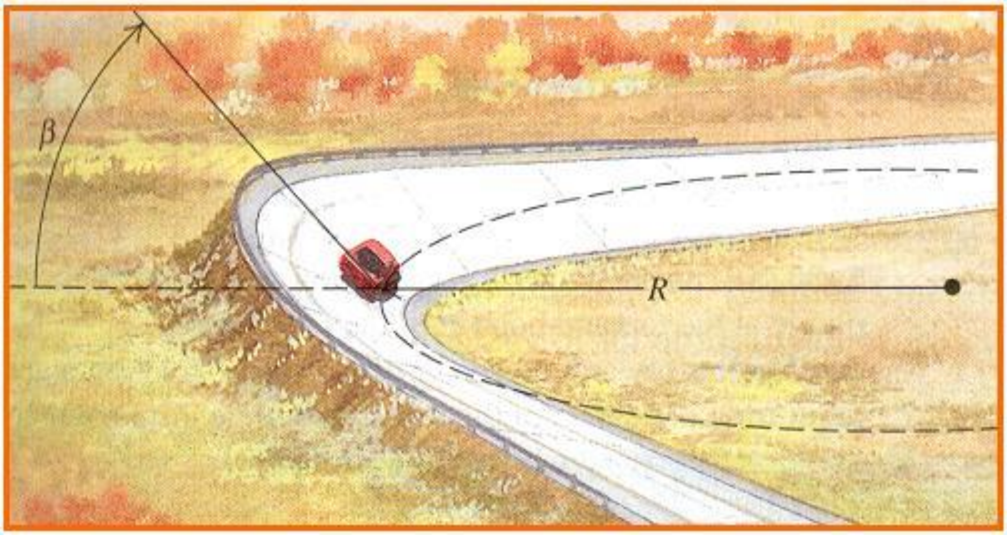
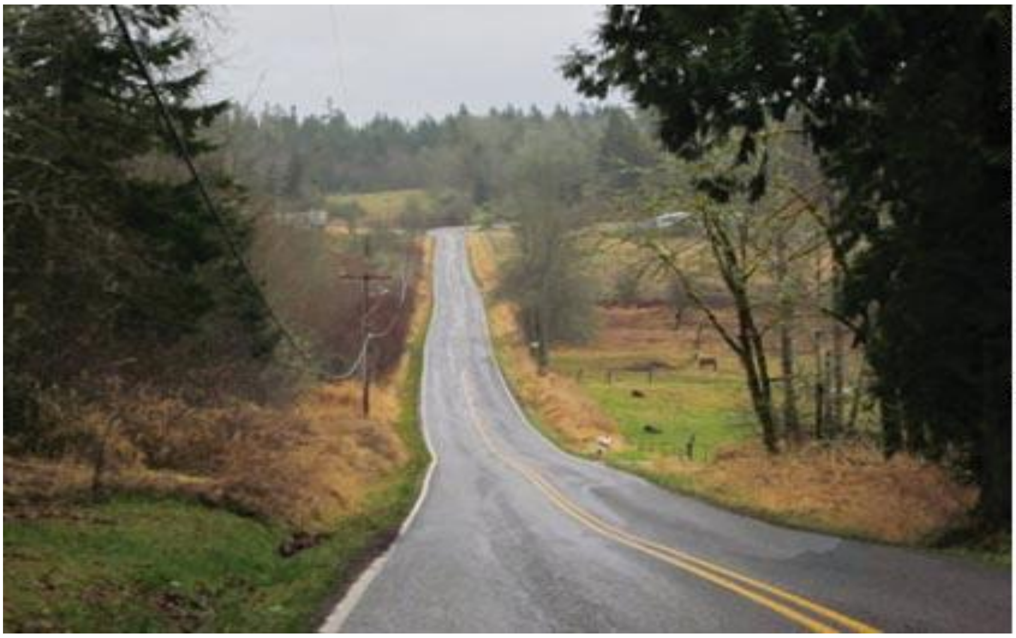

β
R


BOUNDARY


60
ROADWORK
60 KMPH
SPEED
ZONE
60
ROAD
WORK



Thurston County Public Works, WA


3m
centre line of
the inside lane

Relationships among: Distance, Velocity, and Acceleration

$d = \{(U_f + U_i)\, t\}/2$

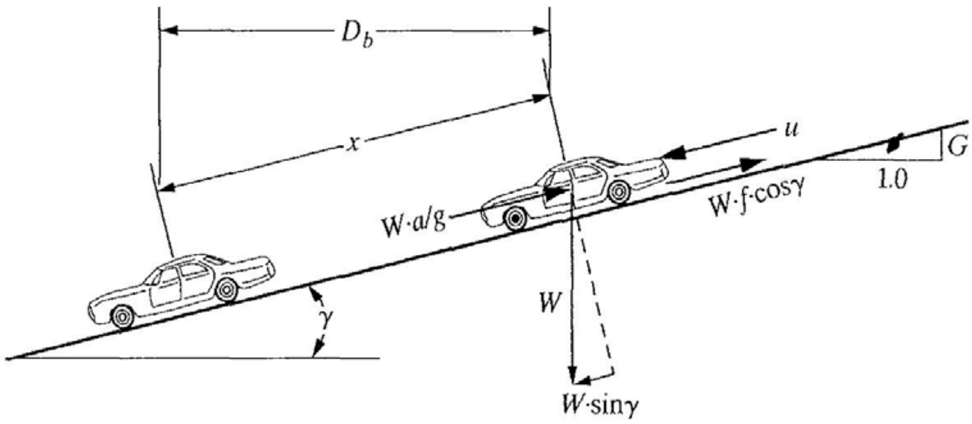
$t = 2d/(U_f + U_i)$

$a = (U_f - U_i)/t$ **(at deceleration)**

$a = (U_f - U_i) / (2d/(U_f + U_i))$

$a = \{(U_f)^2 - (U_i)^2\}/2d$

at $U_f = 0 \rightarrow a = -(U_i)^2/2d$



W = weight of vehicle
f = coefficient of friction
g = acceleration of gravity
a = vehicle acceleration
u = speed when brakes applied
D_b = braking distance
γ = angle of incline
$G = \tan\gamma$ (% grade/100)
x = distance traveled by the vehicle along the road during braking

Figure 3.7 Forces Acting on a Vehicle Braking on a Downgrade

always made with respect to the horizontal plane. Since the braking distance is an input in the design of highway curves, the horizontal component of the distance x is used.

$$\text{Frictional force on the vehicle} = Wf\cos\gamma$$

The force acting on the vehicle due to deceleration is Wa/g, where g is acceleration due to gravity. The component of the weight of the vehicle is $W\sin\gamma$. Substituting into $\Sigma f = ma$, we obtain

$$W\sin\gamma - Wf\cos\gamma = \frac{Wa}{g} \tag{3.18}$$

The deceleration that brings the vehicle to a stationary position can be found in terms of the initial velocity u as $a = -u^2/2x$ (assuming uniform deceleration), where x is the distance traveled in the plane of the grade during braking. Eq. 3.16 can then be written as

$$W\sin\gamma - Wf\cos\gamma = -\frac{Wu^2}{2gx} \tag{3.19}$$

However, $D_b = x\cos\gamma$, and we therefore obtain

$$\frac{Wu^2}{2gD_b}\cos\gamma = Wf\cos\gamma - W\sin\gamma$$

giving

$$\frac{u^2}{2gD_b} = f - \tan\gamma$$

$$D_b = \frac{u^2}{2g(f - \tan\gamma)} \tag{3.20}$$

Note, however, that $\tan\gamma$ is the grade G of the incline (that is, percent of grade/100), as shown in Figure 3.7.

Eq. 3.20 can therefore be written as

$$D_b = \frac{u^2}{2g(f - G)} \tag{3.21}$$

If g is taken as 9.81 m/sec^2 and u is expressed in km/h, Eq. 3.19 becomes

$$D_b = \frac{u^2}{254(f - G)} \tag{3.22}$$

and D_b is given in m.

A similar equation could be developed for a vehicle traveling uphill, in which case the following equation is obtained.

$$D_b = \frac{u^2}{254(f + G)} \tag{3.23}$$

A general equation for the braking distance can therefore be written as

$$D_b = \frac{u^2}{254(f \pm G)} \tag{3.24}$$

$f = \frac{a}{g}$ a = 3.41 m/sec^2

$$D_b = \frac{u^2}{254\left(\frac{a}{g} \pm G\right)} \tag{3.25}$$

Similarly, it can be shown that the horizontal distance traveled in reducing the speed of a vehicle from u_1 to u_2 in km/h during a braking maneuver is given by

$$D_b = \frac{u_1^2 - u_2^2}{254\left(1\frac{a}{g} \pm G\right)} \tag{3.26}$$

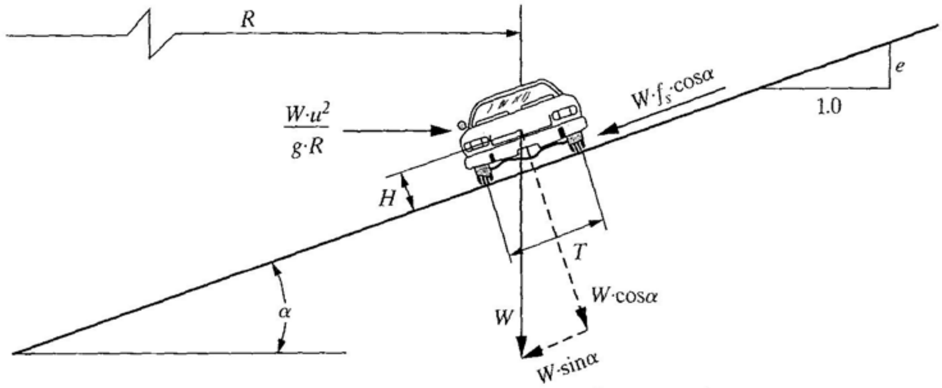
1/3.6

m

$$S(\cancel{ft}) = 0.278ut + \frac{u^2}{254\left(1\frac{a}{g} \pm G\right)} \tag{3.27}$$

t= perception-reaction time = 2.5 sec

Minimum Radius of circular curve



a_c : accelaration of curvilinear motion

W = weight of vehicle
f_s = coefficient of side friction
g = acceleration of gravity
u = speed when brakes applied
R = radius of curve
α = angle of incline
e = tan α (rate of superelevation)
T = track width
H = height of center of gravity

Centrifugal Force:

$$F_c = \frac{W a_c}{g}$$

$$a_c = \frac{u^2}{R}$$

Figure 3.8 Forces Acting on a Vehicle Traveling on a Horizontal Curve Section of a Road

When the vehicle is in equilibrium with respect to the incline (that is, the vehicle moves forward but neither up nor down the incline), we may equate the three relevant forces and obtain

$$\frac{Wu^2}{gR}\cos\alpha = W\sin\alpha + Wf_s\cos\alpha \tag{3.31}$$

where f_s = coefficient of side friction and $(u^2/g) = R(\tan\alpha + f_s)$. This gives

$$R = \frac{u^2}{g(\tan\alpha + f_s)} \tag{3.32}$$

Tan α, the tangent of the angle of inclination of the roadway, is known as the *rate of superelevation e*. Eq. 3.32 can therefore be written as

$$R = \frac{u^2}{g(e + f_s)} \tag{3.33}$$

Again, if g is taken as 9.81 m/sec^2 and u is measured in km/h, the minimum radius R is given in m as

$$R = \frac{u^2}{127(e + f_s)} \tag{3.34}$$

Eq. 3.34 shows that to reduce R for a given velocity, either e or f_s or both should be increased.

Coefficient of side friction	
Design Speed (km/h)	Coefficient of side friction, f_s
48	0.20
64	0.16
80	0.14
96	0.12
112	0.1

Example 3.9 Minimum Radius of a Highway Horizontal Curve

An existing horizontal curve on a highway has a radius of 142 m, which restricts the posted speed limit on this section of the road to only 61.5% of the design speed of the highway. If the curve is to be improved so that its posted speed will be the design speed of the highway, determine the minimum radius of the new curve. Assume that the rate of superelevation is 0.08 for both the existing curve and the new curve to be designed.

Solution:

- Use Eq. 3.34 to find the posted speed limit on the existing curve. Since the posted speed limit is not known, assume f_s is 0.16.

$$R = \frac{u^2}{127(e + f_s)}$$

$$142 = \frac{u^2}{127(0.08 + 0.16)}$$

$$u = 65.79 \text{ km/h}$$

- The posted speed limit is 64 km/h, as speed limits are usually posted at intervals of 8 km/h.
- Check assumed f_s for 64 km/h = 0.16.
- Determine the design speed of the highway.

$$\frac{64}{0.615} = 104.07 \text{ km/h}$$

- Design speed = 104 km/h.
- Find the radius of the new curve by using Eq. 3.34 with the value of f_s for 104 km/h from Table 3.3 (f_s = 0.11, interpolating between 96 km/h and 104 km/h).

$$R = \frac{104^2}{127(0.08 + f_s)}$$

$$= \frac{(104)^2}{127(0.08 + 0.11)} = 448.24 \text{ m}$$

Example 3.9 Minimum Radius of a Highway Horizontal Curve

An existing horizontal curve on a highway has a radius of 465 ft, which restricts the posted speed limit on this section of the road to only 61.5% of the design speed of the highway. If the curve is to be improved so that its posted speed will be the design speed of the highway, determine the minimum radius of the new curve. Assume that the rate of superelevation is 0.08 for both the existing curve and the new curve to be designed.

Solution:

- Use Eq. 3.34 to find the posted speed limit on the existing curve. Since the posted speed limit is not known, assume f_s is 0.16.

$$R = \frac{u^2}{15(e + f_s)}$$

$$465 = \frac{u^2}{15(0.08 + 0.16)}$$

$$u = 40.91 \text{ mi/h}$$

- The posted speed limit is 40 mi/h, as speed limits are usually posted at intervals of 5 mi/h.
- Check assumed f_s for 40 mi/h = 0.16.
- Determine the design speed of the highway.

$$\frac{40}{0.615} = 65.04 \text{ mi/h}$$

- Design speed = 65 mi/h.
- Find the radius of the new curve by using Eq. 3.34 with the value of f_s for 65 mi/h from Table 3.3 (f_s = 0.11, interpolating between 60 mi/h and 65 mi/h).

$$R = \frac{65^2}{15(0.08 + f_s)}$$

$$= \frac{(65)^2}{15(0.08 + 0.11)} = 1482.45 \text{ ft}$$

DECISION SIGHT DISTANCE

Decision sight distance reaction time = 3 - 14.5 s

Complex location → driver reaction > 2.5 s

Example locations:

1. exit or entrance gore
2. lane drop
3. freeway left-side entrance or exit
4. railroad/highway grade crossing
5. approach to detour or lane closure
6. toll plaza
7. intersection location where unusual or unexpected maneuvers are required.

Table 3.5

Columns A and B were developed using:

$$\mathbf{SSD = 0.278\ ut + D_b}$$

Columns C, D, and E were developed using Equation : $\mathbf{DSD = 0.278\ ut}$

where: DSD = decision sight distance, m

Table 3.5 Decision Sight Distances for Different Design Speeds and Avoidance Maneuvers

U.S. Customary					
	Decision Sight Distance (ft)				
	Avoidance Maneuver				
Design Speed (mi/h)	*A*	*B*	*C*	*D*	*E*
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445
75	875	1545	1180	1365	1545
80	970	1685	1260	1455	1650

Note: Brake reaction distance predicted on a time of 2.5 s; deceleration rate of 11.2 ft/s^2 used to determine calculated sight distance.

Avoidance Maneuver *A:* Stop on rural road—t = 3.0 s

Avoidance Maneuver *B:* Stop on urban road—t = 9.1 s

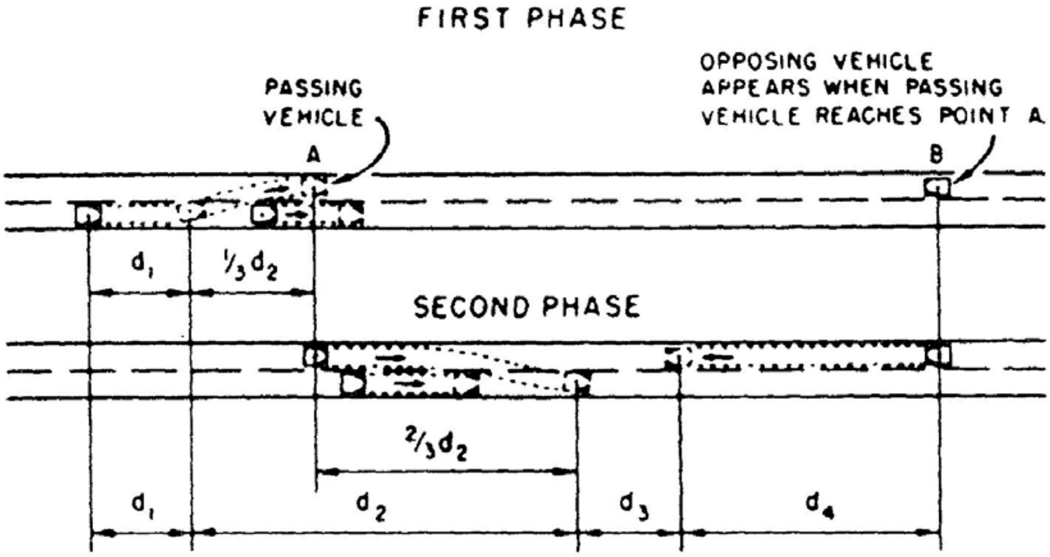
Avoidance Maneuver *C:* Speed/path/direction change on rural road—t varies between 10.2 and 11.2 s

Avoidance Maneuver *D:* Speed/path/direction change on suburban road—t varies between 12.1 and 12.9 s

Avoidance Maneuver *E:* Speed/path/direction change on urban road—t varies between 14.0 and 14.5 s

Passing sight distance (PSD)

PSD = $d_1 + d_2 + d_3 + d_4$



where

d_1 = distance traversed during perception-reaction time and during initial acceleration to the point where the passing vehicle just enters the left lane

d_2 = distance traveled during the time the passing vehicle is traveling in the left lane

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

d_4 = distance moved by the opposing vehicle during two thirds of the time the passing vehicle is in the left lane (usually taken to be 2/3 d_2)

The distance d_1 is obtained from the expression

$$d_1 = 0.278 t_1 \left(u - m + \frac{a t_1}{2} \right) \qquad (3.36)$$

where

t_1 = time for initial maneuver (sec)
a = average acceleration rate (km/h/sec)
u = average speed of passing vehicle (km/h)
m = difference in speeds of passing and impeder vehicles

The distance d_2 is obtained from

$$d_2 = 0.278\, u t_2$$

where

t_2 = time passing vehicle is traveling in left lane (sec)
u = average speed of passing vehicle (km/h)

The clearance distance d_3 between the passing vehicle and the opposing vehicle at the completion of the passing maneuver has been found to vary between 30 m and 90 m.

Table 3.6 shows these components calculated for different speeds. It should be made clear that values given in Table 3.6 are for design purposes and cannot be used for marking passing and no-passing zones on completed highways. The values used

Table 3.6 Components of Safe Passing Sight Distance on Two-Lane Highways

	Speed Range in mi/h (Average Passing Speed in mi/h)			
Component	*30–40 (34.9)*	*40–50 (43.8)*	*50–60 (52.6)*	*60–70 (62.0)*
Initial maneuver:				
a = average acceleration (mi/h/sec)[a]	1.40	1.43	1.47	1.50
t_1 = time (sec)[a]	3.6	4.0	4.3	4.5
d_1 = distance traveled (ft)	145	216	289	366
Occupation of left lane:				
t_2 = time (sec)[a]	9.3	10.0	10.7	11.3
d_2 = distance traveled (ft)	477	643	827	1030
Clearance length:				
d_3 = distance traveled (ft)[a]	100	180	250	300
Opposing vehicle:				
d_4 = distance traveled (ft)	318	429	552	687
Total distance, $d_1 + d_2 + d_3 + d_4$ (ft)	1040	1468	1918	2383

Functional Classification of highways

Urban: Population >= 5000 person. Urbanized >= 50000 person

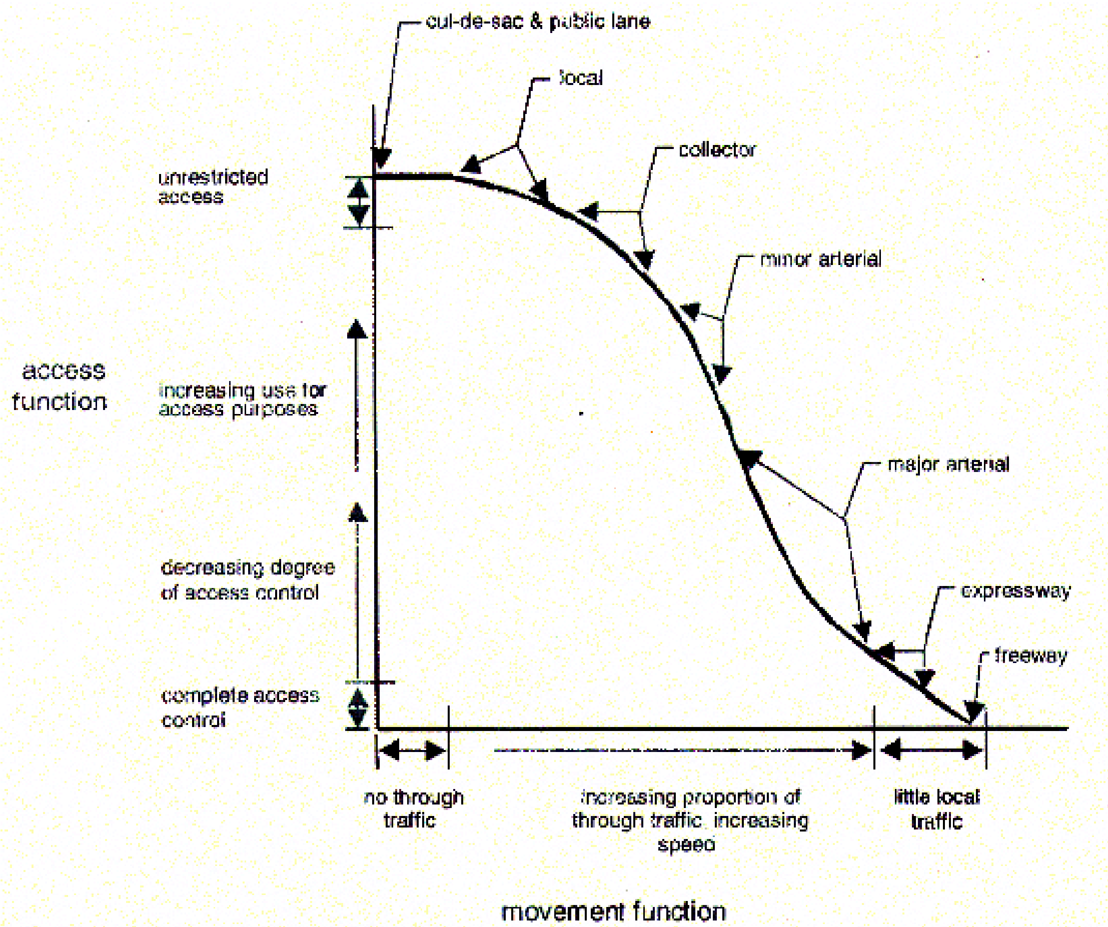
Rural: Population < 5000 person

Urban Highways:

1. Arterials
   - Principle
     - ❖ Freeways (full control of access)
     - ❖ Expressways (partial control of access)
     - ❖ Others
   - Minor
2. Collectors
3. Local

Rural Highways:

1. Arterials
   - Principle
     - ❖ Freeways (full control of access)
     - ❖ Others
   - Minor
2. Collectors
   - Major
   - Minor
3. Local

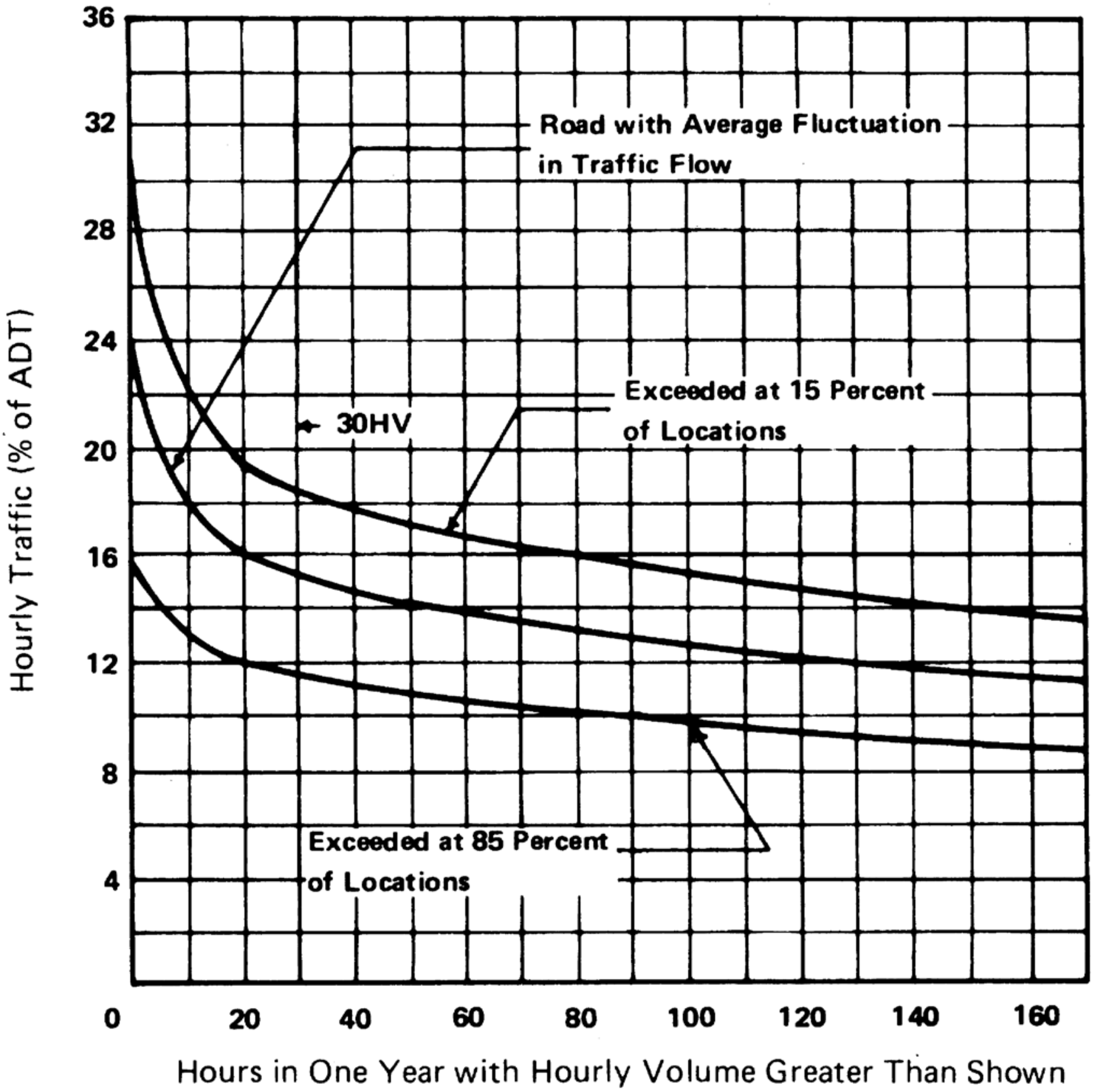


Schematic relationship between movement function access functions of streets

Design Hourly Volume (DHV)

- Projected hourly volume that is used for design
- Percentage of the expected ADT on the highway
- Figure shows the relationship between the highest hourly volumes and ADT.

DHV: Design Hourly Volume			
Highway Type	Normal Fluctuation	Unusual Fluctuation	Remarks
Rural	30th-highest hourly volume 12-18% of ADT (15% Average)	50 percent of the volume that occurs for only a few peak hours during the design year	❖ Lower level of service than that which normally exists on rural roads with normal fluctuations ❖ This may result in some congestion during the peak hour, but the capacity of the highway normally will not be exceeded.
Urban	30th-highest hourly volume 8-12% of ADT	average of the highest afternoon peak hour volume for each week in the year	



Relationship between Peak Hour and Annual Average Daily Traffic on Rural Highways

Table 15.1 Minimum Design Speeds for Rural Collector Roads

	Design Speed (mi/h) for Specified Design Volume (veh/day)		
Type of Terrain	*0 to 400*	*400 to 2000*	*Over 2000*
Level	40	50	60
Rolling	30	40	50
Mountainous	20	30	40

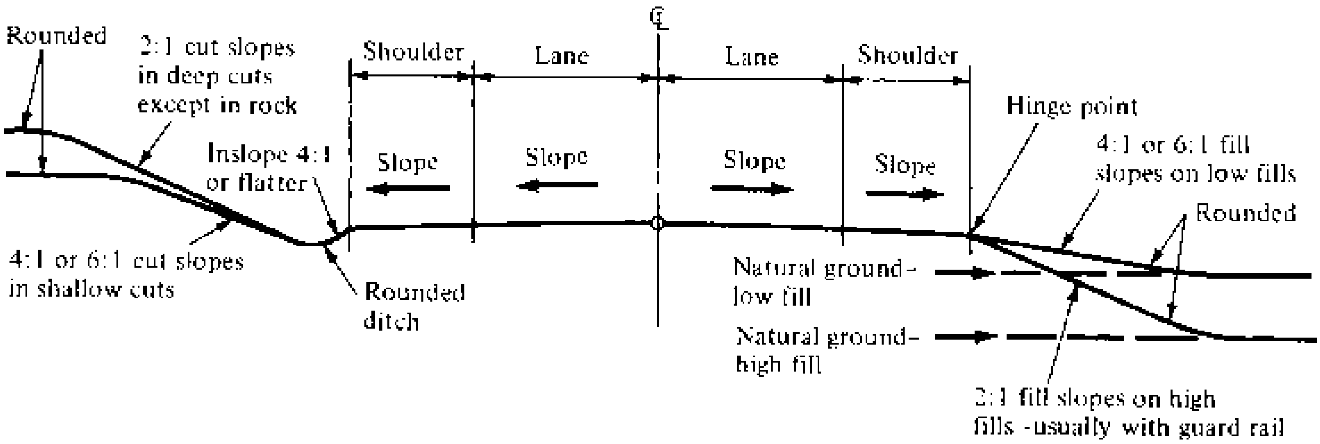
Note: Where practical, design speeds higher than those shown should be considered.
SOURCE: Adapted from *A Policy on Geometric Design of Highways and Streets,* American Association of State Highway and Transportation Officials, Washington, D.C., 2004. Used with permission.

Table 15.2 Minimum Design Speeds for Various Functional Classifications

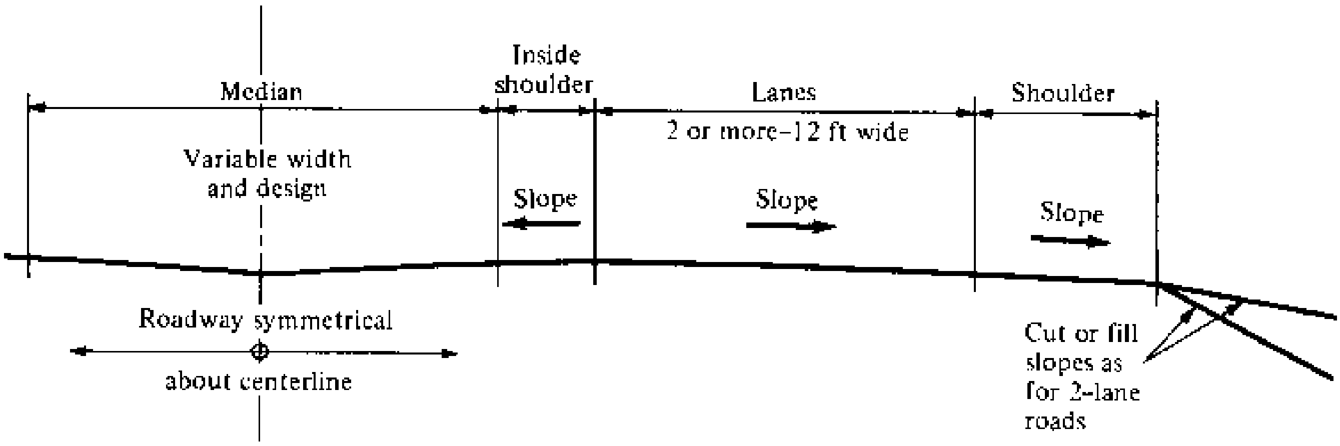
		Speed (mi/h)					
Class		*20*	*30*	*40*	*50*	*60*	*70*
Rural principal arterial	Min 50 mi/h for freeways			x	x	x	x
Rural minor arterial				x	x	x	x
Rural Collector Road	DHV over 400			x	x	x	
	DHV 20-400			x	x	x	
	DHV 100-200		x	x	x		
	Current ADT over 400		x	x	x		
	Current ADT under 400	x	x	x			
Rural Local Road	DHV over 400		x	x	x		
	DHV 200-400		x	x	x		
	DHV 100-200		x	x	x		
	Current ADT over 400		x	x	x		
	Current ADT 250-400	x	x	x			
	Current ADT 50-250	x	x				
	Current ADT under 50	x	x				
Urban principal arterial	Minimum 50 mi/h for freeways		x	x	x	x	x
Urban minor arterial			x	x	x	x	
Urban collector street			x	x	x		
Urban local street		x	x				

SOURCE: *Road Design Manual,* Virginia Department of Transportation, Richmond, VA. (See www.virginiadot.org/business/locdes/rdmanual-index.asp for most current version.)

Cross-Section Elements



Typical Cross Section for Two-Lane Highways



Typical Cross Section for Multilane Highways (half section)

Width of Travel Lanes:

2.7 to 3.6 m

- ✓ 3.6m: High initial cost/low maintenance cost
- ☒ 3m: Low initial cost/High maintenance cost

3-3.3 m: two- lane, two-way rural roads

3.3 m: low-speed

3 m: urban areas: Low traffic volume **and** there are extreme right-of-way constraints

Shoulders:

Functions:

- Necessary stop
- Laterally *support* the pavement structure

Width:

- Width: 0.6 m on minor roads to 3.6 m on major arterials
- Minimum = 0.6 m.
- Preferable= 1.8-2.4 m.

Recommended slopes:

- 2 to 6 % for bituminous and concrete-surfaced shoulders
- 4 to 6 % for gravel or crushed-rock shoulders

Medians:

A median is the section of a divided highway that separates the lanes in opposing directions.

- ✓ The width of a median is the distance between the edges of the inside lanes, <u>including</u> the median <u>shoulders</u>.
- ✓ Width= 1.2 to 24 m or more

Functions:

- ⊕ Recovery area for out-of-control vehicles
- ⊕ Separating opposing traffic
- ⊕ Stopping areas during emergencies
- ⊕ Storage areas for left-turning and U-turning vehicles
- ⊕ Refuge for pedestrians
- ⊕ Reducing the effect of headlight glare
- ⊕ Temporary lanes and cross-overs during maintenance operations

Median type:

- Raised
- Flush
- Depressed

Median Barriers

- Prevent an errant vehicle from crossing the portion of a divided highway

Roadside Barriers:

- *Protect* vehicles from obstacles or slopes on the roadside.
- *Shield* pedestrians and property from the traffic stream.
- Should be provided when the *slope* of an embankment is *high* or when traveling under an *overhead bridge*.

Curbs and Gutters:

Curbs:

- Portland cement concrete or bituminous concrete
- Mainly on urban highways
- Delineate pavement edges
- Pedestrian walkways
- Control drainage, improve aesthetics, and reduce right of way.
- **Vertical** or *sloping*.

No vertical curbs with:

☒ barriers

☒ design speeds > 64 km/h

Gutters or drainage ditches:

✓ Located on the pavement side of a curb to provide drainage facility for the highway

✓ Sloped to prevent any hazard to traffic

✓ Cross slopes= 5 to 8 %

✓ Width = 0.3 to 2 m

✓ Gutters can be designed as V-type sections or as broad, flat, rounded sections.

Guard Rails:

Guard rails are longitudinal *barriers* placed on the outside of sharp curves and at sections with high fills.

- ✓ Main function: prevent vehicles from leaving the roadway

Install Guard rail when:

Fill height >= 2.4 m

Shoulder slopes > 4:1

- ✓ The *weak* post system provides for the post to collapse on impact, with the rail deflecting and absorbing the energy due to impact.

Sidewalks:

Sidewalks are usually provided on roads in urban areas, but are uncommon in rural areas.

- For *pedestrians*
- *Minimum* clear width of 1.2 m in *residential* areas
- Width *range* (1.2 to 2.4 m) in *commercial* areas

Figure Steel-Backed Timber Guardrail

Figure 15.7 Single-Slope Concrete Median Barrier

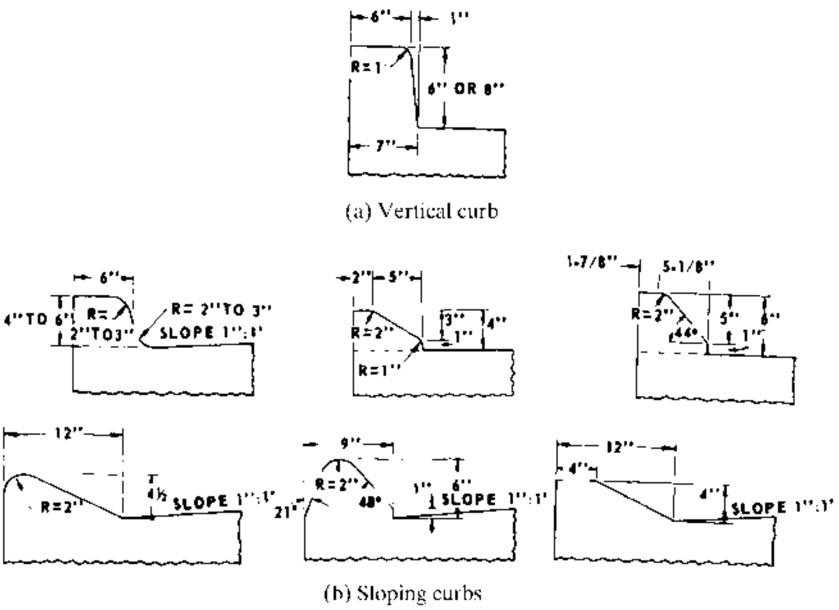


Figure Typical Highway Curbs

Cross Slopes:

- One-way drainage
- Two-way drainage

Shape:

- Plane
- Curved

Curved:

- ✓ A parabola is generally used for curved cross sections
- ✓ highest point of the pavement (called the crown) is slightly rounded
- ✓ cross slope increasing toward the pavement edge

Advantage:

- ✓ Slope increases outward to the pavement edge
- ✓ Enhancing the flow of surface water away from the pavement

Disadvantage:

- ☒ Difficult to construct

Plane

- ✓ Uniform slopes at both sides of the crown.

Recommended rates of cross slopes:

- 1.5 to 2 percent for high- type pavements
- 2 to 6 percent for low-type pavements

Side Slopes:

- Embankments and fills
- Stability for earthworks
- Recovery area for out-of-control vehicles (Safety feature)

Slopes:

3:1 or flatter are generally used for high embankments

Right of Way:

The right of way is the *total* land *area* acquired for the construction of a highway.

- Two- lane urban collector streets 12 and 18 m
- Minimum for two-lane arterials is 25 m
- Undivided four-lane arterials 19 to 33 m
- Divided arterials 36 to 90

Table 15.3 Guide for Earth Slope Design

Height of Cut or Fill (ft)	***Earth Slope, for Type of***		
	Flat or	***Moderately***	***Steep***
0-4	**6:1**	**6:1**	**4:1**
4-10	**4:1**	**4:1**	**2:1***
10-15	**4:1**	**2.50:1**	**1.75:1***
15-20	**2:1***	**2:1***	**1.75:1***
Over 20	**2:1***	**2:1***	**1.75:1***

*Slopes 2:1 or steeper should be subject to a soil stability analysis and should be reviewed for safety.

Maximum Highway Grades (See Table)

Table 15.4 Recommended Maximum Grades

Rural Collectors[a]									
	Design Speed (mi/h)								
Type of Terrain	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*
	Grades (%)								
Level	7	7	7	7	7	7	6	6	5
Rolling	10	10	9	9	8	8	7	7	6
Mountainous	12	11	10	10	10	10	9	9	8

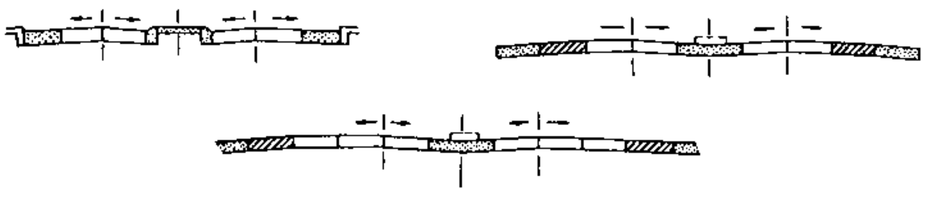
Urban Collectors[a]									
	Design Speed (mi/h)								
Type of Terrain	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*
	Grades (%)								
Level	9	9	9	9	9	8	7	7	6
Rolling	12	12	11	10	10	9	8	8	7
Mountainous	14	13	12	12	12	11	10	10	9

Rural Arterials									
	Design Speed (mi/h)								
Type of Terrain	*40*	*45*	*50*	*55*	*60*	*65*	*70*	*75*	*80*
	Grades (%)								
Level	5	5	4	4	3	3	3	3	3
Rolling	6	6	5	5	4	4	4	4	4
Mountainous	8	7	7	6	6	5	5	5	5

Rural and Urban Freeways[b]							
	Design Speed (mi/h)						
Type of Terrain	*50*	*55*	*60*	*65*	*70*	*75*	*80*
	Grades (%)						
Level	4	4	3	3	3	3	3
Rolling	5	5	4	4	4	4	4
Mountainous	6	6	6	5	5	–	–

Urban Arterials							
	Design Speed (mi/h)						
Types of Terrain	*30*	*35*	*40*	*45*	*50*	*55*	*60*
	Grades (%)						
Level	8	7	7	6	6	5	5
Rolling	9	8	8	7	7	6	6
Mountainous	11	10	10	9	9	8	8

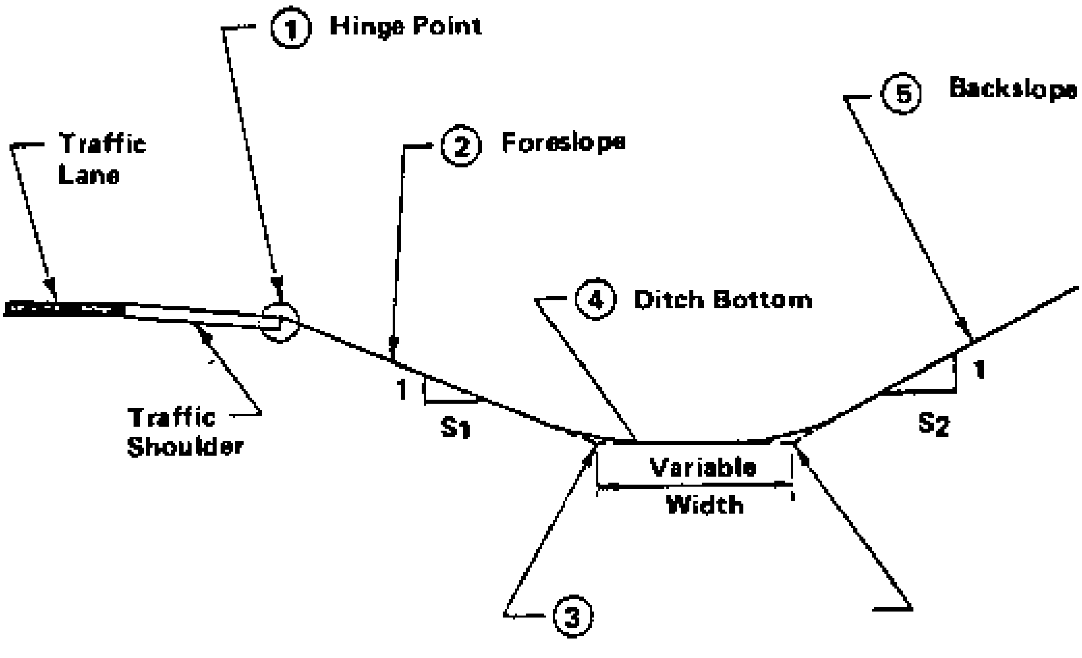
[a]Maximum grades shown for rural and urban conditions of short lengths (less than 500 ft) and on one-way downgrades may be up to 2% steeper.
[b]Grades that are 1% steeper than the value shown may be used for extreme cases in urban areas where development precludes the use of flatter grades and for one-way downgrades, except in mountainous terrain.
SOURCE: Adapted from A *Policy on Geometric Design of Highways and Streets,* American Association of State Highway and Transportation Officials, Washington, D.C., 2004. Used with permission.



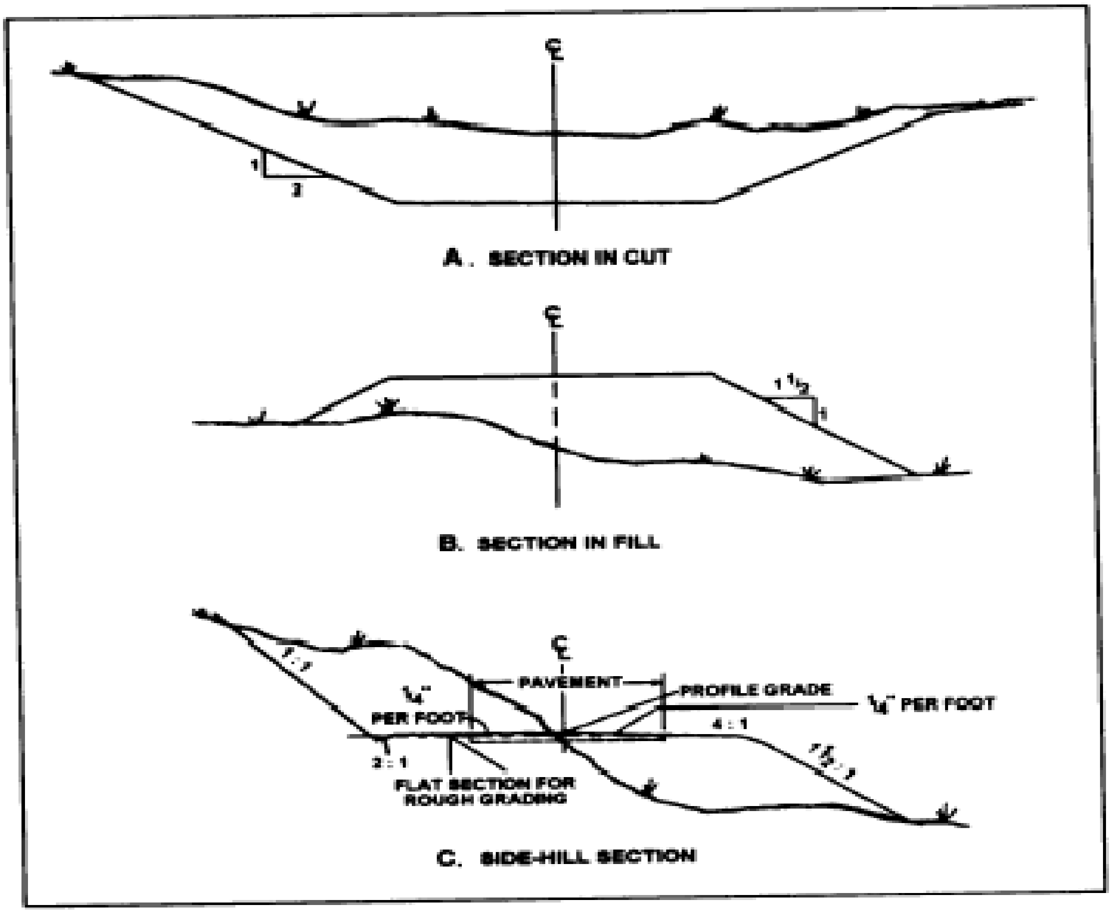
(a) Each pavement slopes two ways.

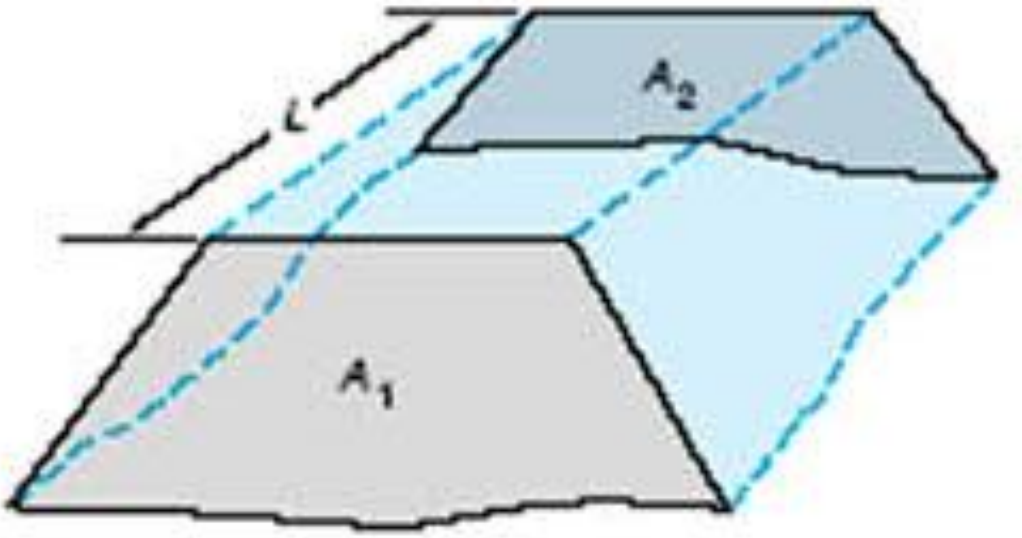
(b) Each pavement slopes one way.

Figure 15.9 Basic Cross Slope Arrangements for Divided Highways



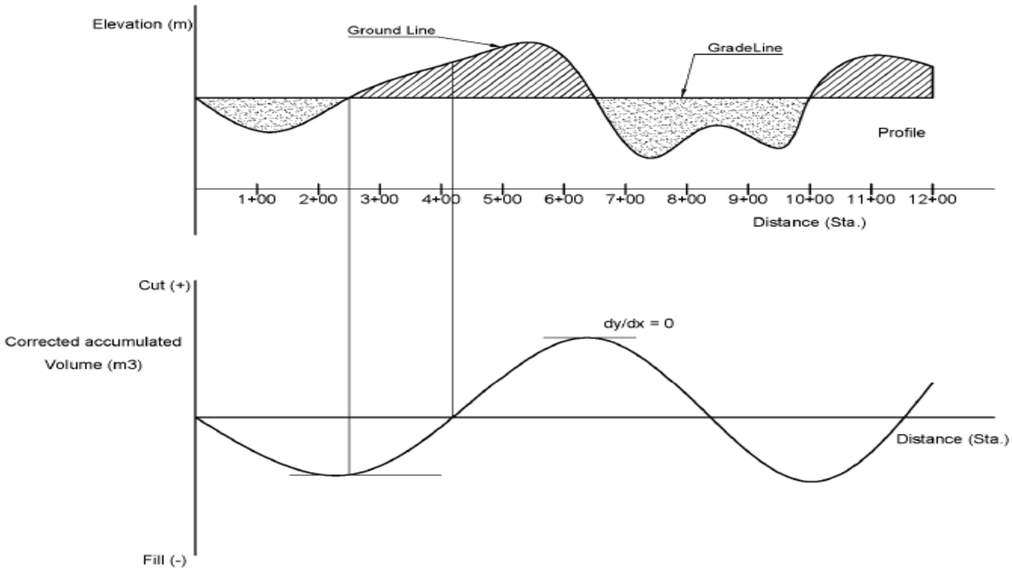
Volume calculations





V = L/2 (A1+A2)

Mass-haul diagram:
Continuous curve showing the relationship between the accumulated algebraic sums of corrected earthwork volume and distance for the purpose of minimizing the cost of excavating hauling & damping the materials (Soil).



- Rising → Cut
- Falling → Fill
- Steep slop → High cut or fill
- Zero slop → Change from cut to fill or vice versa.
- Zero value → Balance between cut and fill

Haul = Volume (m^3) * Distance (station)

Haul distance: The distance of moving the masses of soil from one place to another, in the process of earthwork.

Free haul distance (FHD) : The distance within which there is a fixed price for excavating, hauling, and damping the materials regardless of the distance moved.
Free haul charge (FHC) = $/$m^3$ (ID/m^3)

Over haul distance (OHD):The distance beyond (FHD) for which there is an additional price for each
Over haul charge = $/$m^3$. Station (ID/m^3 .Station)

Borrow: material brought from regions located away from right of way (pits approved by engineer according to the specifications).
Waste: Excess of excavated material used to flatten slopes.

max.OHD. = Borrow charge/ OH Charge

Limit of economical haul distance (LEHD) : The distance beyond which it is more economical to waste and borrow rather than to pay for the cost of over hauling.

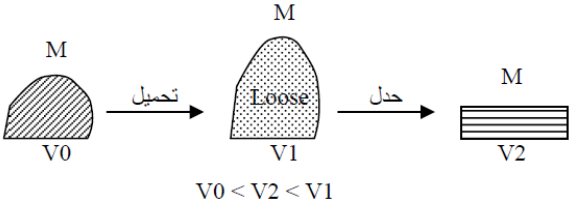
LEHD = FHD + max. OHD

Waste Cost (WC) = FHC ***[Usually]***

Correction:

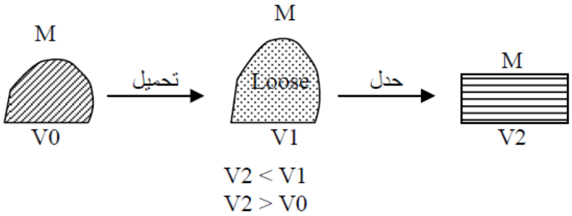
* Sandy, Silty clay

Shrinkage: 5 – 15 % ≈ 10%



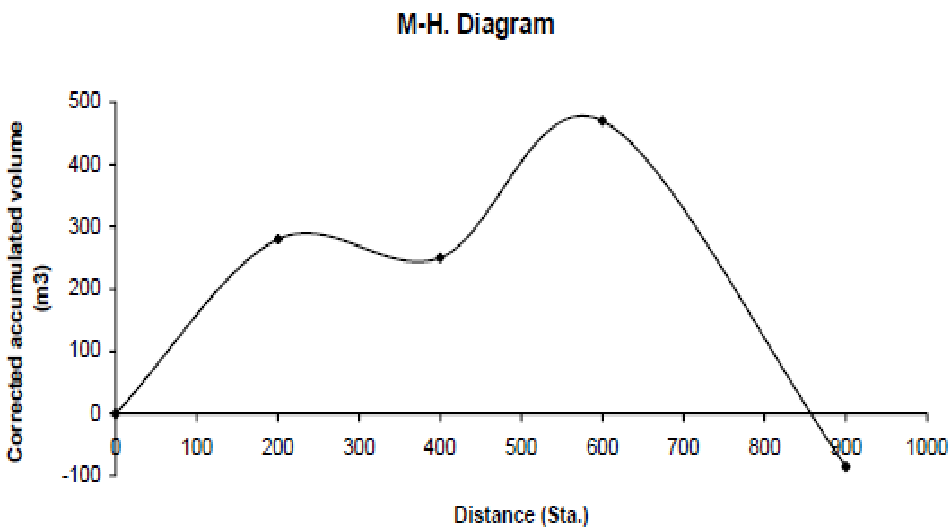
* Lime stone, Sand stone

Bulking: 25 – 35 % ≈ 30%



Ex.:

Sta.	End Area (m^2)		Cut + (m^3)	(-) Shrinkage 10% (m^3)	Corrected Cut + (m^3)	Fill – (m^3)	Balance Vol. (m^3) (Cut-Fill)	Accu. Vol. (m^3)
	Cut	Fill						
0+00	4.0	2.6						0
			0.5*(4+2)*200 =600	600*0.1 =60	600-60 =540	0.5*(2.6+0)*200 =260	+280	
2+00	2	0						+280
			0.5*(2+1)*200 =300	300*0.1 =30	300-30 =270	0.5*(0+3)*200 =300	-30	
4+00	1.0	3						+250
			0.5*(1+7)*200 =800	800*0.1 =80	800-80 =720	0.5*(3+2)*200 =500	+220	
6+00	7	2						+470
			0.5*(7+0)*300 =1050	1050*0.1 =105	1050-105 =945	0.5*(2+8)*300 =1500	-555	
9+00	0	8						-85



Example

Using the following data, Find the total cost

FHD = 400 m (4 station)

FHC = 10 $/m^3$

OHC =7.5 $/m3.Sta

Borrow Cost = 15 $/m3

Station	Accumulated Volume
0	0
1	1100
2	2400
3	3900
4	5400
5	5475
6	4045
7	2395
8	965
9	-135
10	-55
11	1345

Station	Acum. Vol.
0	0
1	1100
2	2400
3	3900
4	5400
5	5475
6	4045
7	2395
8	965
9	-135
10	-55
11	1345

FHD = 400 m (4 station)

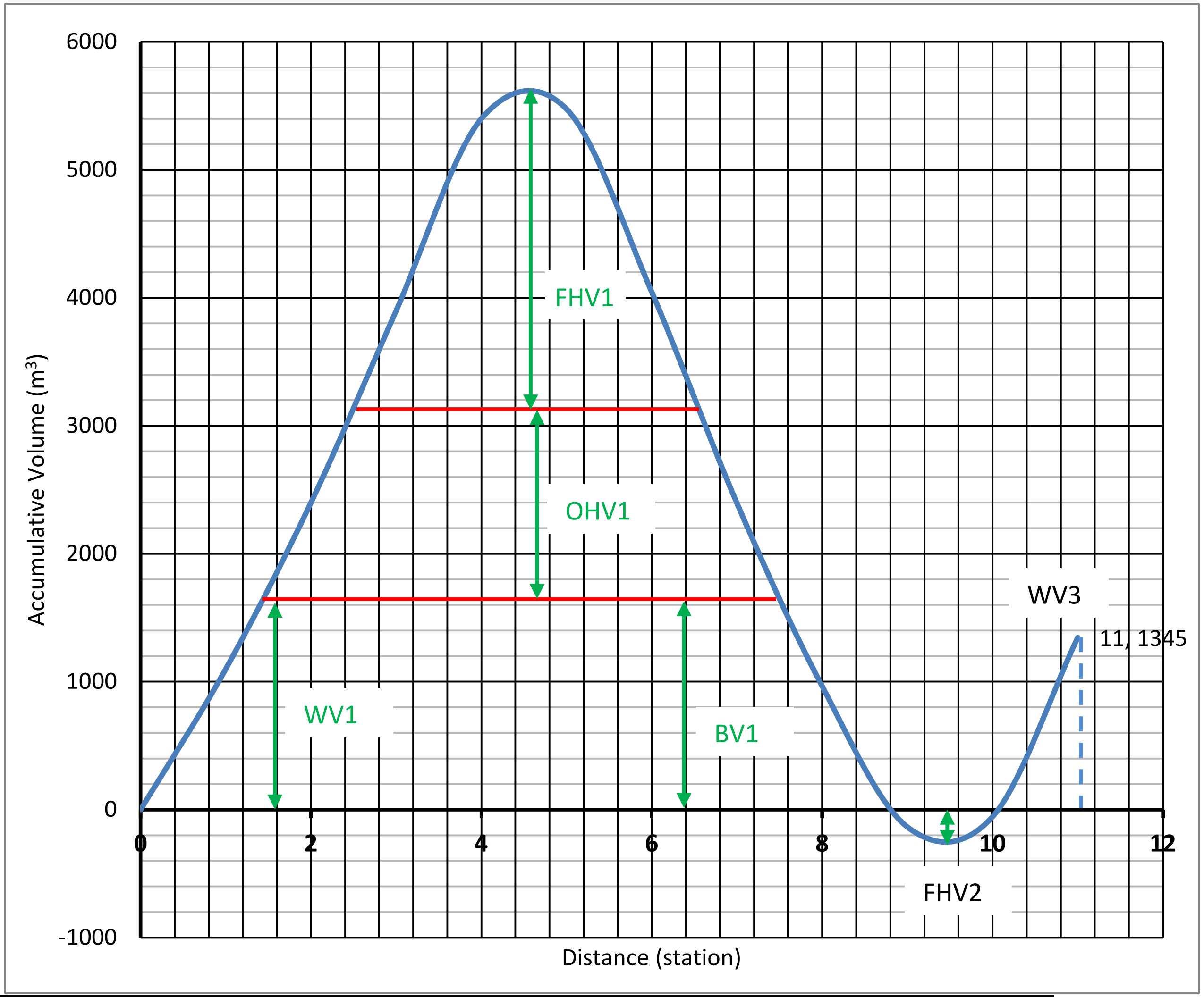
FHC = 10 $\$/m^3$

OHC =7.5 $\$/m^3.Sta$

Borrow Cost = 15 $\$/m^3$

LEHD=BC/OHC+FHC

LEHD=(15/7.5)+4 =6 sta.



From Diagram: (Volumes):

FHV1 = 2450 m3

OHV1 = 1400 m3

av.OHD + FHD = 5 sta.

av.OHD = 1 sta.

BV1 = 1750 m3

WV1 = 1750 m3

FHV2 = 225 m3

WV3 = 1345 m3

Cost :

Loop 1:

FHC1 = 10 (2450) = 24500 $

OHC1 = 10(1400) + 7.5 (1400) * 1 = 24500 $

BC1 = 15 (1750) = 26250 $

WC1 = 10 (1750) = 17500 $

Loop2:

FHC2 = 10 (225) = 2250 $

Loop3

WC3 = 10 (1345) = 13450$

Total Cost = 24500+24500+26250+17500+2250+13450 = 108450 $

Principles of Highway Location

Principle for locating highways is roadway elements

Roadway elements:

- Curvature
- Grade
- Others

Must provide:

- Easy flow of traffic at the design capacity,
- Safety
- Minimum disruption to historic and archeological sites and to other land-use activities.

The highway location process involves four phases:

1- **Office study of existing**

Information:

- Engineering reports
- Maps
- Aerial photographs
- Charts

Data:

- Engineering: topography, geology, climate, and traffic volumes.
- Social and demographic: land use and zoning patterns.
- Environmental
- Economic

2-Reconnaissance Survey

Identify several feasible routes

Factors

- Terrain and soil conditions.
- Serviceability of route to industrial and population areas.
- Crossing of other transportation facilities, such as rivers, railroads, and other highways.
- Directness of route.

✪ Control points between the two terminals are determined for each feasible route.

3- Preliminary Location Survey

3.1 Economic Evaluation

3.2 Environmental Evaluation

4- Final Location Survey

The final location survey is the detailed layout of the selected route

Determine:

- Final horizontal and vertical alignments
- Final positions of structures and drainage

FACTORS INFLUENCING HIGHWAY DESIGN

• Functional classification

• Design hourly traffic volume and vehicle mix

• Design speed

• Design vehicle

• Cross section of the highway, such as lanes, shoulders, and medians

• Presence of heavy vehicles on steep grades

• Topography of the area that the highway traverses

• Level of service

• Available funds

• Safety

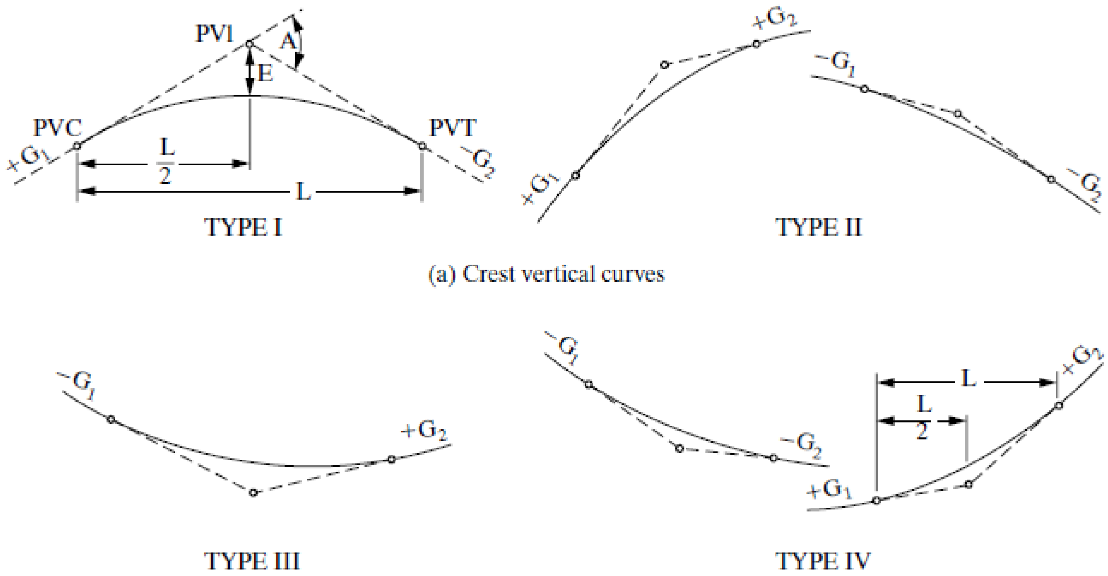
• Social and environmental factors

✪ These factors are often **interrelated**

Vertical Curve

Length of Crest Vertical Curves

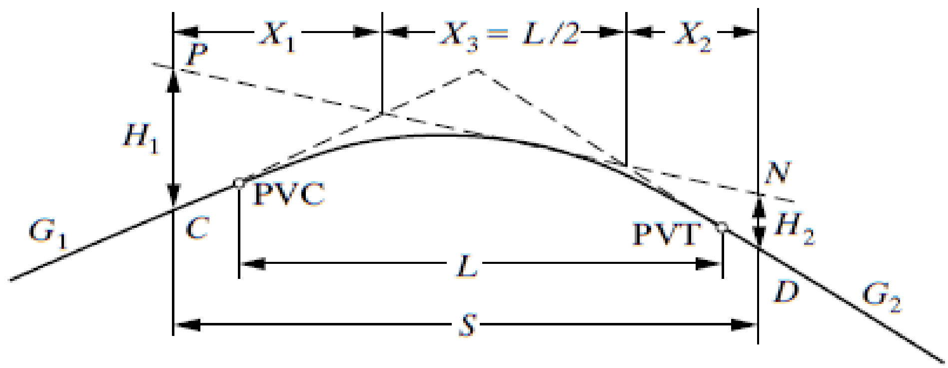
Provision of a minimum stopping sight distance (SSD) is the only criterion used for design of a crest vertical curve. As illustrated in Figures 15.12 and 15.13, there are



(b) Sag vertical curves

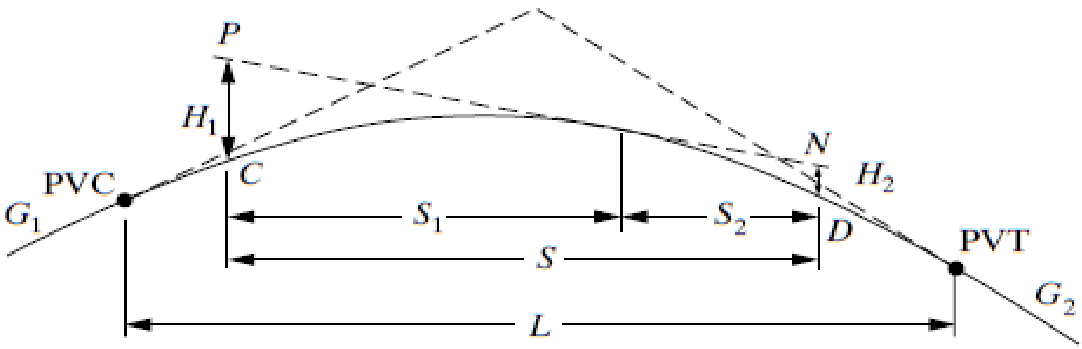
G_1 , G_2 = grades of tangents (%)
A = algebraic difference
L = length of vertical curve
PVC = point of vertical curve
PVI = point of vertical intersection
PVT = point of vertical tangent

$$A = G1 – G$$



L = length of vertical curve (ft)
S = sight distance (ft)
H_1 = height of eye above roadway surface (ft)
H_2 = height of object above roadway surface (ft)
G_1 , G_2 = grades of tangents (%)
PVC = point of vertical curve
PVT = point of vertical tangent

Sight Distance on Crest Vertical Curve ($S > L$)



L = length of vertical curve (ft)
S = sight distance (ft)
H_1 = height of eye above roadway surface (ft)
H_2 = height of object above roadway surface (ft)
G_1 , G_2 = grades of tangents (%)
PVC = point of vertical curve
PVT = point of vertical tangent

Sight Distance on Crest Vertical Curve ($S < L$)

From the properties of the parabola,

$$X_3 = \frac{L}{2}$$

The SSD S is

$$S = X_1 + \frac{L}{2} + X_2 \tag{15.1}$$

X_1 and X_2 can be determined from grades G_1 and G_2 and their algebraic difference A.

The minimum length of the vertical curve for the required sight distance is obtained as

$$L_{\min} = 2S - \frac{200(\sqrt{H_1} + \sqrt{H_2})^2}{A} \text{ (for } S > L) \tag{15.2}$$

$H_1 = 1.1$ m

$H_2 = 0.6$ m

$$L_{min} = 2S - \frac{658}{A} \qquad \text{(for } S > L) \tag{15.3}$$

$$L_{\min} = \frac{AS^2}{200(\sqrt{H_1} + \sqrt{H_2})^2} \text{ (for } S < L) \tag{15.4}$$

$$L_{min} = \frac{AS^2}{658} \qquad \text{for } S < L) \tag{15.5}$$

Example 15.1 Minimum Length of a Crest Vertical Curve

A crest vertical curve is to be designed to join a +3% grade with a −2% grade at a section of a two-lane highway. Determine the minimum length of the curve if the design speed of the highway is 96 km/h, $S < L$, and a perception-reaction time of 2.5 sec. The deceleration rate for braking (a) is 3.41 m/s².

Solution:

- Use the equation derived in Chapter 3 to determine the SSD required for the design conditions. (Since the grade changes constantly on a vertical curve, the worst-case value for G of 3% is used to determine the braking distance.)

$$\begin{aligned} SSD &= 0.278ut + \frac{u^2}{254\left\{\left(\frac{a}{9.81}\right) - G\right\}} \\ &= 0.278 \times 96 \times 2.5 + \frac{96^2}{254\left\{\frac{3.41}{9.81} - 0.03\right\}} \\ &= 66.72 + 114.24 \\ &= 180.96 \text{ m} \end{aligned}$$

- Use Eq. 15.5 to obtain the minimum length of vertical curve:

$$\begin{aligned} L_{min} &= \frac{AS^2}{658} \\ &= \frac{5 \times (180.96)^2}{658} \\ &= 248.83 \text{ m} \end{aligned}$$

Example 15.2 Maximum Safe Speed on a Crest Vertical Curve

An existing vertical curve on a highway joins a +4.4% grade with a −4.4% grade. If the length of the curve is 82 m, what is the maximum safe speed on this curve? What speed should be posted if 8 km/h increments are used?
Assume a = 3.41 m/sec², perception-reaction time = 2.5 sec, and that $S < L$.

Solution:

- Determine the SSD using the length of the curve and Eq. 15.5.

$$\begin{aligned} L_{min} &= \frac{AS^2}{658} \\ 82 &= \frac{8.8 \times S^2}{658} \\ S &= 78.30 \text{ m} \end{aligned}$$

- Determine the maximum safe speed for this sight distance using the equation for SSD from Chapter 3.

$$78.30 = 0.278 \times 2.5u + \frac{u^2}{254\left\{\frac{3.41}{9.81} - 0.044\right\}}$$

u = 55.4 km/h

Speed limit intervals = 8 km/h (…, 24, 32, 40, 48, 56, 64, 72, 80, 88, 96, 104, 112 km/h)

➔ Nearest Speed limit is 48 km/h

Example 15.1 Minimum Length of a Crest Vertical Curve

A crest vertical curve is to be designed to join a +3% grade with a −2% grade at a section of a two-lane highway. Determine the minimum length of the curve if the design speed of the highway is 60 mi/h, $S < L$, and a perception-reaction time of 2.5 sec. The deceleration rate for braking (a) is 11.2 ft/sec².

Solution:

- Use the equation derived in Chapter 3 to determine the SSD required for the design conditions. (Since the grade changes constantly on a vertical curve, the worst-case value for G of 3% is used to determine the braking distance.)

$$\begin{aligned}
\text{SSD} &= 1.47ut + \frac{u^2}{30\left\{\left(\frac{a}{32.2}\right) - G\right\}} \\
&= 1.47 \times 60 \times 2.5 + \frac{60^2}{30\left\{\frac{11.2}{32.2} - 0.03\right\}} \\
&= 220.50 + 377.56 \\
&= 598.1 \text{ ft}
\end{aligned}$$

- Use Eq. 15.5 to obtain the minimum length of vertical curve:

$$\begin{aligned}
L_{\min} &= \frac{AS^2}{2158} \\
&= \frac{5 \times (598.1)^2}{2158} \\
&= 828.8 \text{ ft}
\end{aligned}$$

Example 15.2 Maximum Safe Speed on a Crest Vertical Curve

An existing vertical curve on a highway joins a +4.4% grade with a −4.4% grade. If the length of the curve is 275 ft, what is the maximum safe speed on this curve? What speed should be posted if 5 mph increments are used?
Assume $a = 11.2$ ft/sec^2, perception-reaction time = 2.5 sec, and that $S < L$.

Solution:

- Determine the SSD using the length of the curve and Eq. 15.5.

$$L_{\min} = \frac{AS^2}{2158}$$

$$275 = \frac{8.8 \times S^2}{2158}$$

$$S = 259.69 \text{ ft}$$

- Determine the maximum safe speed for this sight distance using the equation for SSD from Chapter 3.

$$259.69 = 1.47 \times 2.5u + \frac{u^2}{30\left\{\dfrac{11.2}{32.2} - 0.044\right\}}$$

which yields the quadratic equation

$$u^2 + 33.50u - 2367.02 = 0$$

- Solve the quadratic equation to find the u, the maximum safe speed.

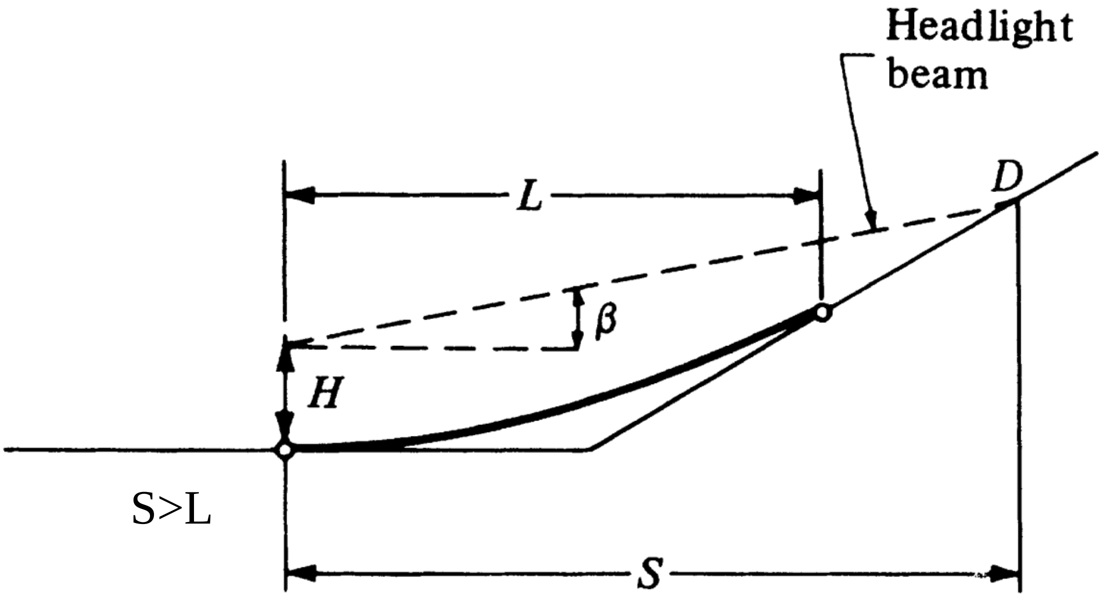
$$u = 34.7 \text{ mi/h}$$

The maximum safe speed for an SSD of 259.69 ft is therefore 34.7 mi/h.

If a speed limit is to be posted to satisfy this condition, a conservative value of 30 mi/h will be used.

Sag

1. SSD Criterion



S>L

$$L_{\min} = 2S - \frac{200(H + S\tan\beta)}{A} \text{(for } S > L\text{)} \qquad (15.6)$$

H = 0.6 m β = 1 degree

$$L_{min} = 2S - \frac{120 + 3.5S}{A} \qquad \text{(for } S > L\text{)} \qquad (15.7)$$

Similarly, for the condition when $S < L$, it can be shown that

$$L_{\min} = \frac{AS^2}{200(H + S\tan\beta)} \text{(for } S < L\text{)} \qquad (15.8)$$

H = 0.6 m β = 1 degree

$$L_{min} = \frac{AS^2}{120 + 3.5S} \qquad \text{(for } S < L\text{)} \qquad (15.9)$$

2. **Comfort Criterion**

$$L_{min} = \frac{Au^2}{395}$$

3. **Appearance Criterion:** $\mathbf{L_{min} = 30A}$

4. **Drainage Criterion :** $\mathbf{L_{max}}$

Example 15.3 Minimum Length of a Sag Vertical Curve

A sag vertical curve is to be designed to join a −5% grade to a +2% grade. If the design speed is 64 km/h, determine the minimum length of the curve that will satisfy all criteria. Assume a = 3.41 m/sec^2 and perception-reaction time = 2.5 s.

Solution:

- Find the stopping sight distance.

$$\text{SSD} = 0.278\,ut + \frac{u^2}{254\left(\frac{3.41}{9.81} - G\right)}$$

$$= 0.278 \times 64 \times 2.5 + \frac{64^2}{254(0.35 - .05)} = 44.48 + 53.75$$

$$= 98.23 \text{ m}$$

- Determine whether $S < L$ or $S > L$ for the headlight sight distance criterion. For $S > L$,

$$L_{\min} = 2S - \frac{(120 + 3.5S)}{A}$$

$$= 2 \times 98.23 - \frac{120 + 3.5 \times 98.23}{7}$$

$$= 130.2 \text{ m}$$

(This condition is not appropriate since ~~324.78~~ ft < ~~430.03~~ ft. Therefore $S \not> L$.)
$130.2^m < 98.23^m$

For $S < L$,

$$L_{\min} = \frac{AS^2}{120 + 3.5S}$$

$$= \frac{7 \times (98.23)^2}{120 + 3.5 \times 98.23}$$

$$= 145.63 \text{ m}$$

This condition is satisfied since 98.23 < 145.63.

- Determine minimum length for the comfort criterion.

$$L_{\min} = \frac{Au^2}{395}$$

$$= \frac{7 \times 64^2}{395} = 72.60 \text{ m}$$

L_{min} = 30A = 30(7) = 210 m

➔ L_{min} which satisfy all criteria is 210 m

Example 15.3 Minimum Length of a Sag Vertical Curve

A sag vertical curve is to be designed to join a −5% grade to a +2% grade. If the design speed is 40 mi/h, determine the minimum length of the curve that will satisfy all criteria. Assume $a = 11.2$ ft/sec^2 and perception-reaction time = 2.5 sec.

Solution:

- Find the stopping sight distance.

$$\text{SSD} = 1.47\,ut + \frac{u^2}{30\left(\frac{11.2}{32.2} - G\right)}$$
$$= 1.47 \times 40 \times 2.5 + \frac{40^2}{30(0.35 - .05)} = 147.0 + 177.78$$
$$= 324.78 \text{ ft}$$

- Determine whether $S < L$ or $S > L$ for the headlight sight distance criterion. For $S > L$,

$$L_{min} = 2S - \frac{(400 + 3.5S)}{A}$$
$$= 2 \times 324.78 - \frac{400 + 3.5 \times 324.78}{7}$$
$$= 430.03 \text{ ft}$$

(This condition is not appropriate since 324.78 < 430.03. Therefore $S \ngtr L$.)
For $S < L$,

$$L_{min} = \frac{AS^2}{400 + 3.5S}$$
$$= \frac{7 \times (324.78)^2}{400 + 3.5 \times 324.78}$$
$$= 480.48 \text{ ft}$$

This condition is satisfied since 324.78 < 480.48.

- Determine minimum length for the comfort criterion.

$$L_{min} = \frac{Au^2}{46.5}$$
$$= \frac{7 \times 40^2}{46.5} = 240.86 \text{ ft}$$

- Determine minimum length for the general appearance criterion.

$$L_{min} = 100\,A$$
$$= 100 \times 7 = 700 \text{ ft}$$

The minimum length to satisfy all criteria is 700 ft.
(*Note*: In order to check the maximum length drainage requirement, it is necessary to use procedures for calculating curve elevations that are discussed later in this chapter.)

Length of curve by K value

L = KA

Table 15.5 Values of *K* for Crest Vertical Curves Based on Stopping Sight Distance

Design Speed (mi/h)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a Calculated	Design
15	80	3.0	3
20	115	6.1	7
25	155	11.1	12
30	200	18.5	19
35	250	29.0	29
40	305	43.1	44
45	360	60.1	61
50	425	83.7	84
55	495	113.5	114
60	570	150.6	151
65	645	192.8	193
70	730	246.9	247
75	820	311.6	312
80	910	383.7	384

[a]Rate of vertical curvature, K, is the length of curve per percent algebraic difference in intersecting grades (A).

$$K = L/A$$

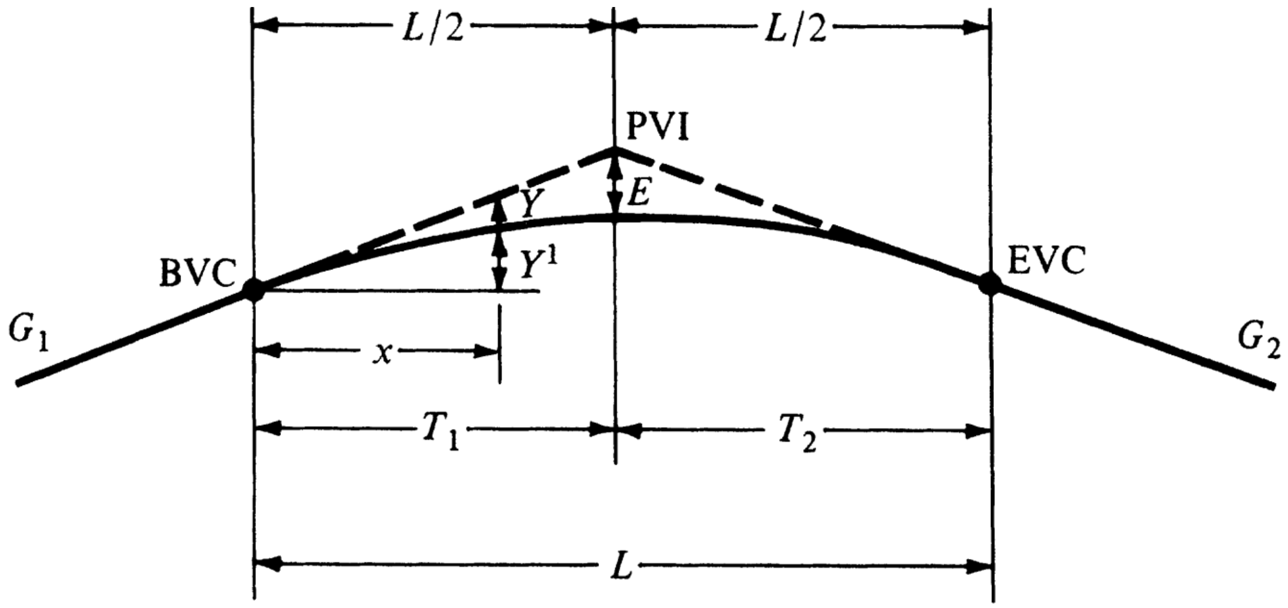
Table 15.6 Values of *K* for Sag Vertical Curves Based on Stopping Sight Distance

Design Speed (mi/h)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a Calculated	Design
15	80	9.4	10
20	115	16.5	17
25	155	25.5	26
30	200	36.4	37
35	250	49.0	49
40	305	63.4	64
45	360	78.1	79
50	425	95.7	96
55	495	114.9	115
60	570	135.7	136
65	645	156.5	157
70	730	180.3	181
75	820	205.6	206
80	910	231.0	231

[a]Rate for vertical curvature, K, is the length of curve (ft) per percent algebraic difference intersecting grades (A).

$$K = L/A$$

Elevation of Crest and Sag Curve



PVI = point of vertical intersection
BVC = beginning of vertical curve (same point as PVC)
EVC = end of vertical curve (same point as PVT)
***E* = external distance**
G_1, G_2 = grades of tangents (%)
L = length of curve
A = algebraic difference of grades, $G_1 - G_2$

Parabola

$\mathbf{Y = ax^2+bx+c}$ **:** $\mathbf{b=0\ ;\ c=0 \rightarrow Y = ax^2}$

$Y'_{max/min} = 2ax$

Totla change in Slope = A (A % → A/100) & x = L →

$$a = \frac{A}{200L}$$

$$Y = \frac{A}{200L}x^2$$

At x = L/2 → $$E = \frac{A}{200L}\left(\frac{L}{2}\right)^2 = \frac{AL}{800}$$

$$Y = (\frac{x}{L/2})^2 E$$

$$
\begin{aligned}
Y^1 &= \frac{G_1 x}{100} - Y \\
&= \frac{G_1 x}{100} - \frac{A}{200L} x^2 \\
&= \frac{G_1 x}{100} - \left(\frac{G_1 - G_2}{200L} \right) x^2
\end{aligned} \tag{15.16}
$$

Differentiating Eq. 15.16 and equating it to 0 will give the value of x at the highest point on the curve:

$$
\frac{dY^1}{dx} = \frac{G_1}{100} - \left(\frac{G_1 - G_2}{100L} \right) x = 0 \tag{15.17}
$$

Therefore:

$$
X_{\text{high}} = \frac{100L}{(G_1 - G_2)} \frac{G_1}{100} = \frac{LG_1}{(G_1 - G_2)} \tag{15.18}
$$

$$
Y^1_{\text{high}} = \frac{LG_1^2}{200(G_1 - G_2)} \tag{15.19}
$$

Example 15.4 Design of Crest Vertical Curve

A crest vertical curve joining a +3 percent and a −4 percent grade is to be designed for 75 mi/h. If the tangents intersect at station (345 + 60.00) at an elevation of 250 ft, determine the stations and elevations of the BVC and EVC. Also, calculate the elevations of intermediate points on the curve at the whole stations. A sketch of the curve is shown in Figure 15.16.

Solution: For a design speed of 75 mi/h, $K = 312$. From Table 15.5,

$$\text{Minimum length} = 312 \times [3 - (-4)] = 2184 \text{ ft}$$

$$\text{Station of BVC} = (345 + 60) - \left(\frac{21 + 84}{2}\right) = 334 + 68$$

$$\text{Station of EVC} = (334 + 68) + (21 + 84) = 356 + 52$$

$$\text{Elevation of BVC} = 250 - \left(0.03 \times \frac{2184}{2}\right) = 217.24 \text{ ft}$$

The remainder of the computation is efficiently done using the format shown in Table 15.7.

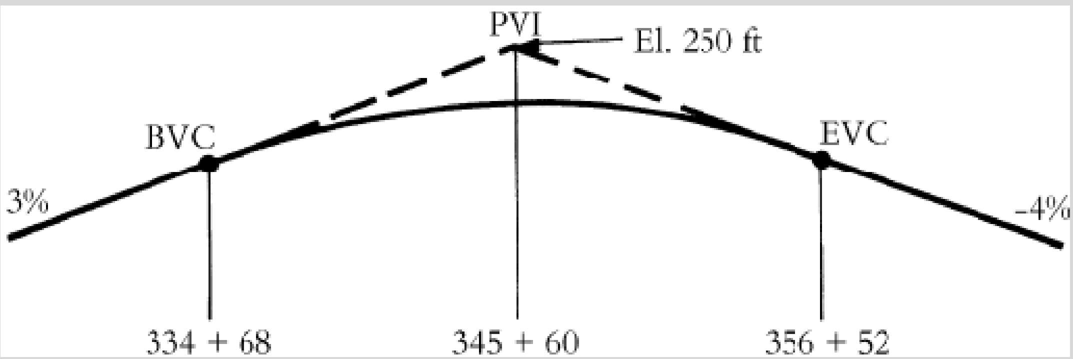


Figure 15.16 Layout of a Vertical Curve for Example 15.4

Table 15.7 Elevation Computations for Example 15.4

Station	*Distance from BVC (x) (ft)*	*Tangent Elevation (ft)*	*Offset* $\left[y = \dfrac{Ax^2}{200L}\right]$ *(ft)*	*Curve Elevation (Tangent Elevation – Offset) (ft)*
BVC 334 + 68	0	217.24	0.01	217.24
BVC 335 + 00	32	$217.24 + \dfrac{32}{100} \times 3 = 218.20$	0.02	218.18
BVC 336 + 00	132	221.20	0.28	220.92
BVC 337 + 00	232	224.20	0.86	223.34
BVC 338 + 00	332	227.20	1.77	225.43
BVC 339 + 00	432	230.20	2.99	227.21
BVC 340 + 00	532	233.20	4.54	228.66
BVC 341 + 00	632	236.20	6.40	229.80
BVC 342 + 00	732	239.20	8.59	230.61
BVC 343 + 00	832	242.20	11.09	231.11
BVC 344 + 00	932	245.20	13.92	231.28
BVC 345 + 00	1032	248.20	17.07	231.13
BVC 346 + 00	1132	251.20	20.54	230.66
BVC 347 + 00	1232	254.20	24.32	229.88
BVC 348 + 00	1332	257.20	28.43	228.77
BVC 349 + 00	1432	260.20	32.86	227.34
BVC 350 + 00	1532	263.20	37.61	225.59
BVC 351 + 00	1632	266.20	42.68	223.52
BVC 352 + 00	1732	269.20	48.07	221.13
BVC 353 + 00	1832	272.20	53.79	218.41
BVC 354 + 00	1932	275.20	59.82	215.38
BVC 355 + 00	2032	278.20	66.17	212.03
BVC 356 + 00	2132	281.20	72.84	208.36
EVC 356 + 52	2184	282.76	76.44	206.32

Example 15.4 Design of Crest Vertical Curve

A crest vertical curve joining a +3 percent and a −4 percent grade is to be designed for 120 km/h. If the tangents intersect at station (345 + 18) at an elevation of 76.2 m determine the stations and elevations of the BVC and EVC. Also, calculate the elevations of intermediate points on the curve at the whole stations. A sketch of the curve is shown in Figure 15.16.

Solution: For a design speed of 120 km/h, $K = 93.5$. From Table 15.5,

$$\text{Minimum length} = 93.5 \times [3 - (-4)] = 654.5 \text{ m}$$

$$\text{Station of BVC} = (345 + 18) - \left(\frac{21 + 25}{2}\right) = 334 + 21$$

$$\text{Station of EVC} = (334 + 21) + (21 + 25) = 356 + 16$$

$$\text{Elevation of BVC} = 76.2 - \left(0.03 \times \frac{664.5}{2}\right) = 66.38$$

The remainder of the computation is efficiently done using the format shown in Table 15.7.

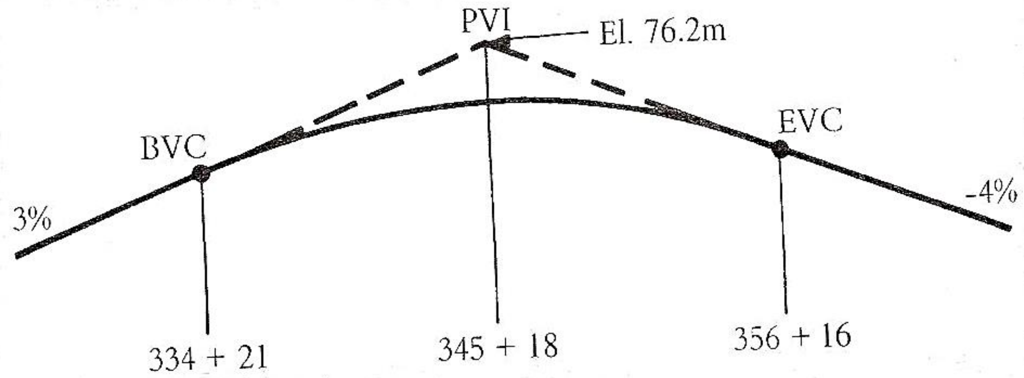


Figure 15.16 Layout of a Vertical Curve for Example 15.4

Table 15.7 Elevation Computations for Example 15.4

Station	*Distance from BVC (x) (m)*	*Tangent Elevation (m)*	*Offset* $\left[y = \frac{Ax^2}{200L}\right]$ *(m)*	*Curve Elevation (Tangent Elevation – Offset) (m)*
BVC 334 + 21	0	66.38	0.00	66.38
BVC 335 + 00	9 ~~10~~ 9	$66.38 + \frac{\cancel{10}\,9}{100} \times 3 = 66.6\cancel{8}5$ 66.68	0.01	66.6~~7~~4
BVC 336 + 00	40	67.58	0.09	67.49
BVC 337 + 00	70	68.48	0.26	68.22
BVC 338 + 00	100	69.38	0.53	68.85
BVC 339 + 00	130	70.28	0.90	69.38
BVC 340 + 00	160	71.18	1.37	69.81
BVC 341 + 00	190	72.08	1.93	70.15
BVC 342 + 00	220	72.98	2.59	70.39
BVC 343 + 00	250	73.88	3.34	70.54
BVC 344 + 00	280	74.78	4.19	70.59
BVC 345 + 00	310	75.68	5.14	70.54
BVC 346 + 00	340	76.58	6.18	70.40
BVC 347 + 00	370	77.48	7.32	70.16
BVC 348 + 00	400	78.38	8.56	69.82
BVC 349 + 00	430	79.28	9.89	69.39
BVC 350 + 00	460	80.18	11.32	68.86
BVC 351 + 00	490	81.08	12.84	69.76
BVC 352 + 00	520	81.98	14.46	67.52
BVC 353 + 00	550	82.88	16.18	66.70
BVC 354 + 00	580	83.78	17.99	65.79
BVC 355 + 00	610	84.68	19.90	64.78
BVC 356 + 00	640	84.58	21.90	63.68
EVC 356 + 16	654.5	85.85	22.91	62.94

Example 15.5 Design of Sag Vertical Curve

A sag vertical curve joins a −3 percent grade and a +3 percent grade. If the PVI of the grades is at station (435 + 50) and has an elevation of 235 ft, determine the station and elevation of the BVC and EVC for a design speed of 70 mi/h. Also compute the elevation on the curve at 100-ft intervals. Figure 15.17 shows a layout of the curve.

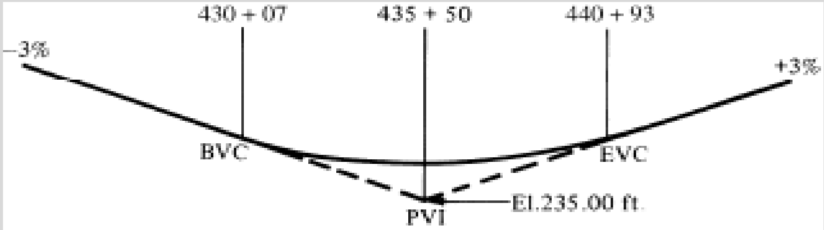


Figure 15.17 Layout for a Sag Vertical Curve for Example 15.5

Solution: For a design speed of 70 mi/h, $K = 181$, using the higher rounded value in Table 15.7.

$$\text{Length of curve} = 181 \times 6 = 1086 \text{ ft}$$
$$\text{Station of BVC} = (435 + 50) - (5 + 43) = 430 + 07$$
$$\text{Station of EVC} = (435 + 50) + (5 + 43) = 440 + 93$$
$$\text{Elevation of BVC} = 235 + 0.03 \times 543 = 251.29$$
$$\text{Elevation of EVC} = 235 + 0.03 \times 543 = 251.29$$

The computation of the elevations is shown in Table 15.8.

Table 15.8 Elevation Computations for Example 15.5

Station	*Distance from BVC (x) (ft)*	*Tangent Elevation (ft)*	*Offset* $\left[y = \frac{Ax^2}{200L}\right]$ *(ft)*	*Curve Elevation (Tangent Elevation + Offset) (ft)*
BVC 430 + 07	0	251.29	0.28	251.29
BCV 431 + 00	93	248.50	0.24	248.74
BCV 432 + 00	193	245.50	1.03	246.53
BCV 433 + 00	293	242.50	2.37	244.87
BCV 434 + 00	393	239.50	4.27	243.77
BVC 435 + 00	493	236.50	6.71	243.21
BCV 436 + 00	593	233.50	9.71	243.21
BCV 437 + 00	693	230.50	13.27	243.77
BCV 438 + 00	793	227.50	17.37	244.87
BCV 439 + 00	893	224.50	22.03	246.53
BCV 440 + 00	993	221.50	27.24	248.74
EVC 440 + 93	1086	218.71	32.58	251.29

Example 15.5 Design of Sag Vertical Curve

A sag vertical curve joins a −3 percent grade and a +3 percent grade. If the PVI of the grades is at station (435 + 15) and has an elevation of 70 m, determine the station and elevation of the BVC and EVC for a design speed of 112 km/h. Also compute the elevation on the curve at 30 m intervals. Figure 15.17 shows a layout of the curve.

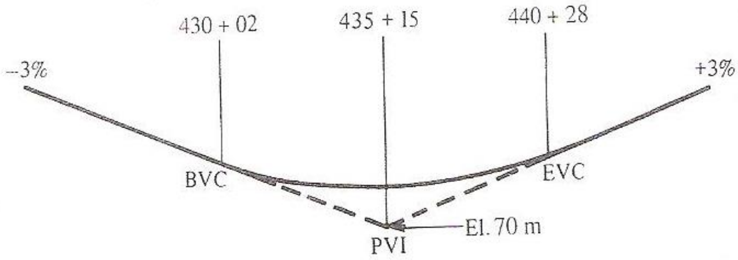


Figure 15.17 Layout for a Sag Vertical Curve for Example 15.5

Solution: For a design speed of 112 km/h, $K = 54.3$, using the higher rounded value in Table 15.6.

$$\text{Length of curve} = 54.3 \times 6 = 325.8$$
$$\text{Station of BVC} = (435 + 15) - (5 + 43) = 430 + 02$$
$$\text{Station of EVC} = (435 + 15) + (5 + 43) = 440 + 28$$
$$\text{Elevation of BVC} = 70 + 0.03 \times 163 = 74.89$$
$$\text{Elevation of EVC} = 70 + 0.03 \times 163 = 74.89$$

The computation of the elevations is shown in Table 15.8.

Table 15.8 Elevation Computations for Example 15.5

Station	*Distance from BVC (x) (m)*	*Tangent Elevation (m)*	*Offset* $\left[y = \frac{Ax^2}{200L}\right]$ *(m)*	*Curve Elevation (Tangent Elevation + Offset) (m)*
BVC 430 + 02	0	74.89	0.00	74.89
BCV 431 + 00	28	74.05	0.08	74.13
BCV 432 + 00	58	73.15	0.32	73.47
BCV 433 + 00	88	72.25	0.73	72.98
BCV 434 + 00	118	71.35	1.19	72.54
BVC 435 + 00	148	70.45	2.02	72.47
BCV 436 + 00	178	69.55	2.92	72.47
BCV 437 + 00	208	68.65	3.99	72.54
BCV 438 + 00	238	67.75	5.23	72.98
BCV 439 + 00	268	66.85	6.62	73.47
BCV 440 + 00	298	65.95	8.18	74.13
EVC 440 + 28	325.8	65.11	9.79	74.90

Horizontal Alignment

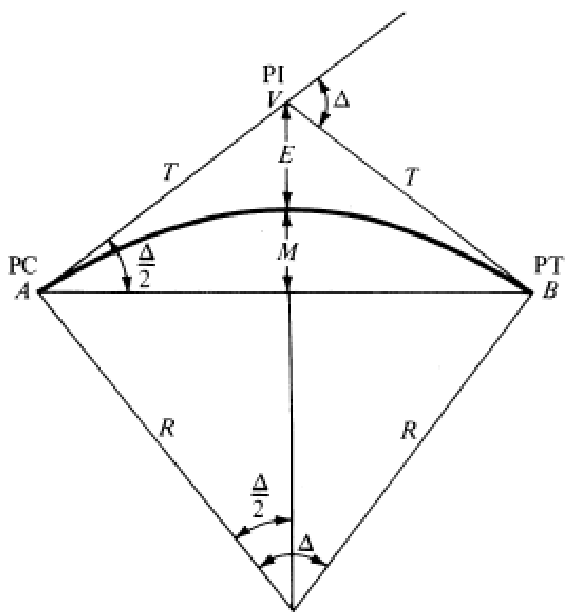
Simple Curves

Figure 15.18 is a layout of a simple horizontal curve. The curve is a segment of a circle with radius R, which is discussed in Chapter 3 for the case when SSD is unobstructed. The relationship was shown to be

$$R = \frac{u^2}{g(e + f_s)}$$

where

R = minimum radius
u = design speed
e = superelevation
f_s = coefficient of side friction



R = radius of circular curve
T = tangent length
Δ = intersection angle
M = middle ordinate
PC = point of curve
PT = point of tangent
PI = point of intersection
E = external distance

Figure 15.18 Layout of a Simple Horizontal Curve

$$\theta = \frac{\pi D_a^\circ}{180}$$

since

$$\frac{\pi R}{180} = \frac{100}{D_a^\circ}$$

Solving for R yields

$$R = \frac{180 \times 100}{\pi D_a^\circ} \qquad R = \frac{5729.6}{D_a^\circ}$$

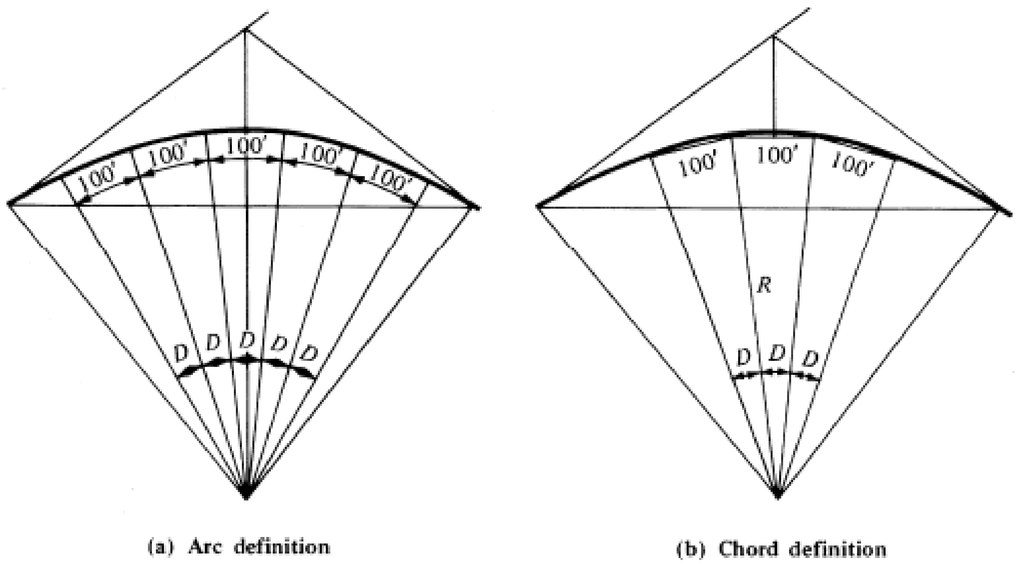


Figure 15.19 Arc and Chord Definitions for a Circular Curve

Attention :**R = 5729.6/D_a^o**

This equation is used in case of Station length = 100 ***units***

Formulas for Simple Curve

Figure 15.18

$$T = R\tan\frac{\Delta}{2} \tag{15.22}$$

The expression for the length C of the chord AB, which is known as the *long chord*, is

$$C = 2R\sin\frac{\Delta}{2} \tag{15.23}$$

The expression for the external distance E, which is the distance from the point of intersection to the curve on a radial line is

$$E = R\sec\frac{\Delta}{2} - R$$

$$E = R\left(\frac{1}{\cos\frac{\Delta}{2}} - 1\right) \tag{15.24}$$

The expression for the middle ordinate M, which is the distance between the midpoint of the long chord and the midpoint of the curve is:

$$M = R - R\cos\frac{\Delta}{2}$$

$$= R\left(1 - \cos\frac{\Delta}{2}\right) \tag{15.25}$$

The expression for the length of the curve L is:

$$L = \frac{R\Delta\pi}{180} \tag{15.26}$$

Field Location of a Simple Horizontal Curve. Simple horizontal curves are usually

The first deflection angle VAp to the first whole station on the curve, which is usually less than a station away from the PC, is equal to $\frac{\delta_1}{2}$ based on the properties of a circle.

The next deflection angle VAq is

$$\frac{\delta_1}{2} + \frac{D}{2}$$

and the next deflection angle VAv is

$$\frac{\delta_1}{2} + \frac{D}{2} + \frac{D}{2} = \frac{\delta_1}{2} + D$$

The next deflection angle VAs is

$$\frac{\delta_1}{2} + \frac{D}{2} + \frac{D}{2} + \frac{D}{2} = \frac{\delta}{2} + \frac{3D}{2}$$

and the last deflection angle VAB is

$$\frac{\delta_1}{2} + \frac{D}{2} + \frac{D}{2} + \frac{D}{2} + \frac{\delta_2}{2} = \frac{\delta_1}{2} + \frac{\delta_2}{2} + \frac{3D}{2} = \frac{\Delta}{2}$$

To set out the horizontal curve, it is necessary to determine δ_1 and δ_2.

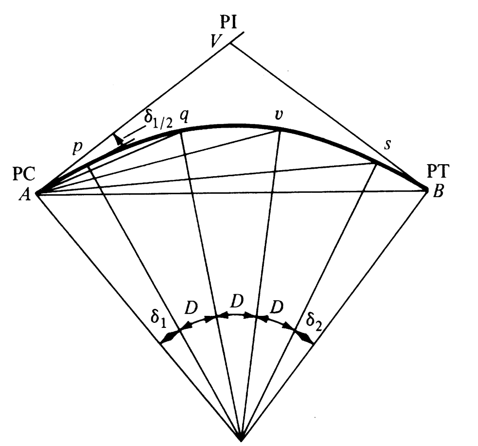


Figure 15.20 Deflection Angles on a Simple Circular Curve

The length of the first arc, l_1, is related to δ_1 as

$$l_1 = \frac{R\pi}{180}\,\delta_1 \tag{15.27}$$

Solving for R provided the following expression.

$$R = \frac{l_1 \times 180}{\delta_1 \pi}$$

Equating R from Eq. 15.27,

$$R = \frac{180L}{\Delta\pi}$$

provides the relationship between the central angle that subtends the length of arc as follows.

$$\frac{l_1}{\delta_1} = \frac{L}{\Delta} = \frac{l_2}{\delta_2}$$

$$\begin{aligned} C_1 &= 2R\sin\frac{\delta_1}{2} \\ C_D &= 2R\sin\frac{D}{2} \\ C_2 &= 2R\sin\frac{\delta_2}{2} \end{aligned} \tag{15.28}$$

where C_1, C_D, and C_2 are the first, intermediate, and last chords, respectively.

To summarize the relationships for deflection angles and chord lengths required to lay out a simple curve, refer to Figure 15.20 and the following formulas:

Chord: $Ap = 2R\sin\frac{\delta_1}{2}$ Deflection angle: $VAp = \frac{\delta_1}{2}$

Chord: $pq = 2R\sin\frac{D_a}{2}$ Deflection angle: $VAq = \frac{\delta_1 + D}{2}$

Chord: $sB = 2R\sin\frac{\delta_2}{2}$ Deflection angle: $VAB = \frac{\delta_1 + D + D + D + \delta_2}{2} = \frac{\Delta}{2}$

Example 15.6 Design of a Simple Horizontal Curve

The intersection angle of a 4° curve is 55°25′, and the PC is located at station 238 + 44.75. Determine the length of the curve, the station of the PT, the deflection angles and the chord lengths for setting out the curve at whole stations from the PC. Figure 15.21 illustrates a layout of the curve.

Solution:

$$\text{Radius of curve} = \frac{5729.6}{D} = \frac{5729.6}{4}$$
$$\approx 1432.4 \text{ ft}$$
$$\text{Length of curve} = \frac{R\Delta\pi}{180} = \frac{1432.4 \times 55.4167\pi}{180}$$
$$= 1385.42 \text{ ft}$$

The station at PT is equal to station (238 + 44.75) + (13 + 85.42) = 252 + 30.17 stations. The distance between the PC and the first station is 239 − (238 + 44.75) = 55.25 ft.

$$\frac{\delta_1}{\Delta} = \frac{l_1}{L}$$
$$\frac{\delta_1}{55.4167} = \frac{55.25}{1385.42}$$

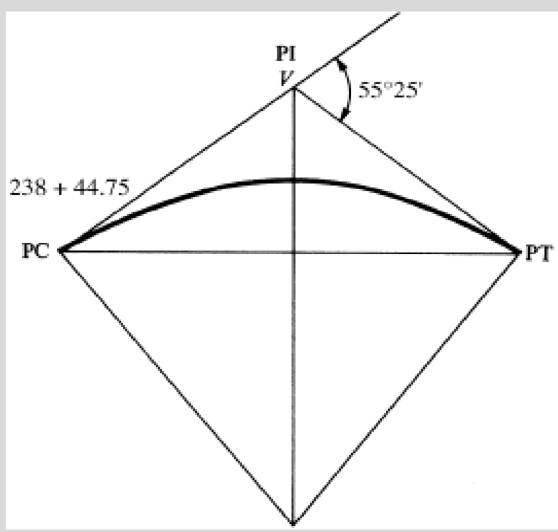


Figure 15.21 Layout of Curve for Example 15.6

Therefore,

$$\delta_1 = 2.210°$$

$$C_1 = 2 \times 1432.4 \sin\left(\frac{2.210}{2}\right) = 55.25 \text{ ft}$$

The first deflection angle to station 239 is $\delta_1/2 = 1.105° = 1°6'18''$.

Similarly,

$$l_2 = (252 + 30.17) - (252) = 30.17 \text{ ft}$$

$$\frac{\delta_2}{2} = \frac{30.17}{1385.42} \times \frac{55.4167}{2} = 0.6034°$$

$$= 36'12''$$

$$C_2 = 2 \times 1432.4 \sin(0.6034°)$$

$$= 30.17 \text{ ft}$$

$$D = 4°$$

$$C_D = 2 \times 1432.42 \sin\left(\frac{4}{2}\right)$$

$$= 99.98 \text{ ft}$$

Note that the deflection angle to PT is half the intersection angle Δ of the tangents. This relationship serves as a check of the computation. Since highway curves are relatively flat, the chord lengths are approximately equal to the arc lengths.

The other deflection angles are computed in Table 15.9.

Table 15.9 Computations of Deflection Angles and Chord Lengths for Example 15.6

Station	*Deflection Angle*	*Chord Length (ft)*
PC 238 + 44.75	0	0
PC 239	1°6′18′	55.25
PC 240	3°6′18′	99.98
PC 241	5°6′18′	99.98
PC 242	7°6′18′	99.98
PC 243	9°6′18′	99.98
PC 244	11°6′18′	99.98
PC 245	13°6′18′	99.98
PC 246	15°6′18′	99.98
PC 247	17°6′18′	99.98
PC 248	19°6′18′	99.98
PC 249	21°6′18′	99.98
PC 250	23°6′18′	99.98
PC 251	25°6′18′	99.98
PC 252	27°6′18′	99.98
PT 252 + 30.17	27°42′30′	30.17

Example 15.6 Design of a Simple Horizontal Curve

The intersection angle of a 4° curve is 55°25′, and the PC is located at station 238 + 13.43. Determine the length of the curve, the station of the PT, the deflection angles and the chord lengths for setting out the curve at whole stations from the PC. Figure 15.21 illustrates a layout of the curve.

Solution:

$$\text{Radius of curve} = \frac{1719}{D} = \frac{1719}{4}$$
$$\approx 429.8 \text{ m}$$

$$\text{Length of curve} = \frac{R\Delta\pi}{180} = \frac{429.8 \times 55.4167\pi}{180}$$
$$= 415.7 \text{ m}$$

The station at PT is equal to station (238 + 13.43) + (13 + 25.63) = 252 + 9.05 stations. The distance between the PC and the first station is 239 − (238 + 13.43) = 16.57 m.

$$\frac{\delta_1}{\Delta} = \frac{l_1}{L}$$

$$\frac{\delta_1}{55.4167} = \frac{16.57}{415.7}$$

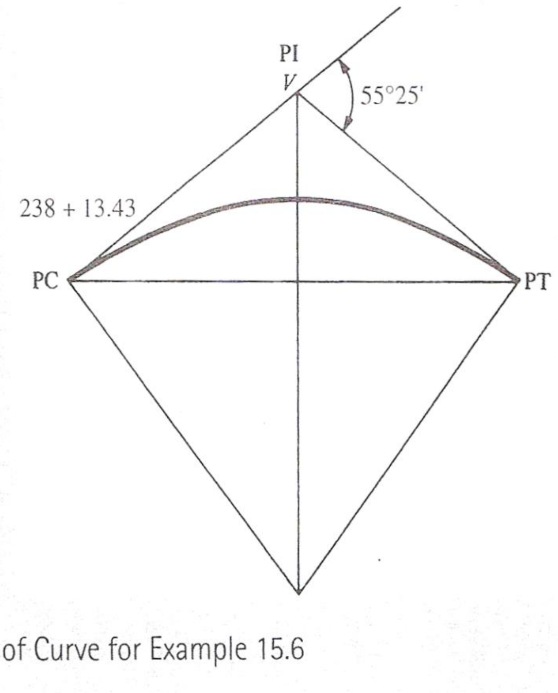


Figure 15.21 Layout of Curve for Example 15.6

Therefore,

$$\delta_1 = 2.210°$$

$$C_1 = 2 \times 429.8 \sin\left(\frac{2.210}{2}\right) = 16.57\text{ m}$$

The first deflection angle to station 239 is $\delta_1/2 = 1.105° = 1°6'18''$.

Similarly,

$$l_2 = (252 + 9.05) - (252) = 9.05\text{ m}$$

$$\frac{\delta_2}{2} = \frac{9.05}{415.7} \times \frac{55.4167}{2} = 0.6034°$$

$$= 36'12''$$

$$C_2 = 2 \times 429.8 \sin(0.6034°)$$

$$= 9.05\text{ m}$$

$$D = 4°$$

$$C_D = 2 \times 429.8 \sin\left(\frac{4}{2}\right)$$

$$= 30\text{ m}$$

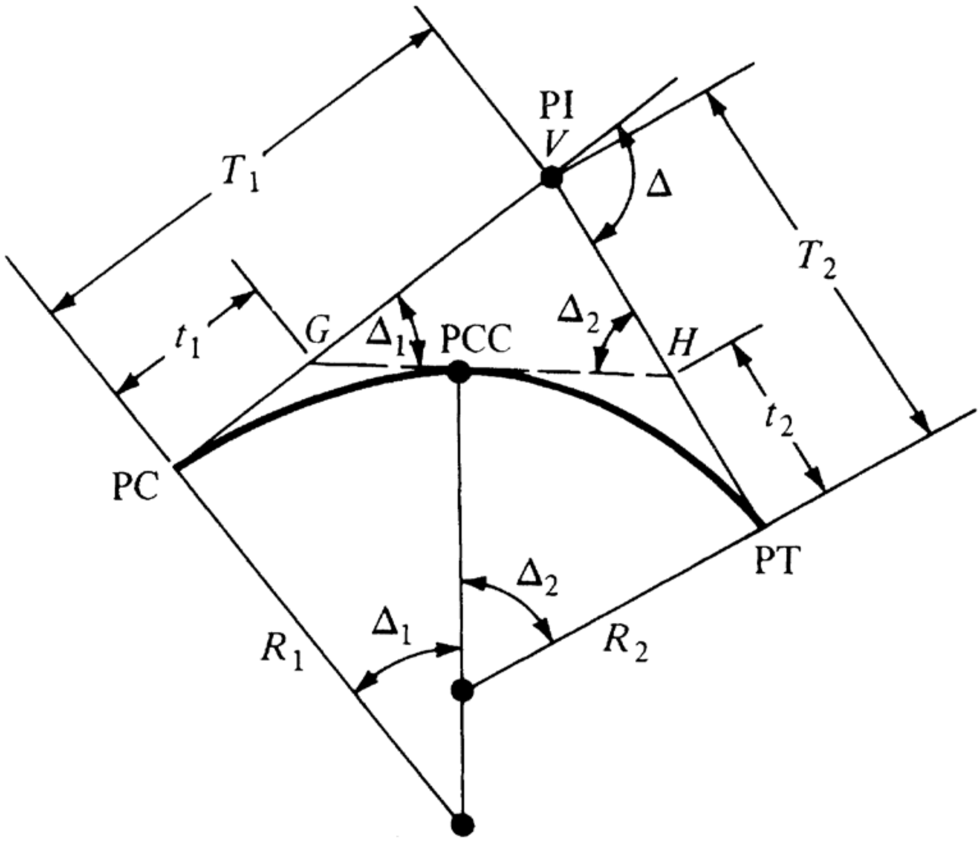
Station	*Deflection Angle*	*Chord Length (m)*
PC 238 + 13.43	0	0
PC 239	1°6'18"	16.57
PC 240	3°6'18"	29.99
PC 241	5°6'18"	29.99
PC 242	7°6'18"	29.99
PC 243	9°6'18"	29.99
PC 244	11°6'18"	29.99
PC 245	13°6'18"	29.99
PC 246	15°6'18"	29.99
PC 247	17°6'18"	29.99
PC 248	19°6'18"	29.99
PC 249	21°6'18"	29.99
PC 250	23°6'18"	29.99
PC 251	25°6'18"	29.99
PC 252	27°6'18"	29.99
PT 252 + 9.05	27°42'30"	9.05

Compound Curves

Used to:

- at-grade intersections,
- ramps of interchanges,
- difficult topographic

☒ To avoid abrupt changes in the alignment, the radii of two consecutive simple curves that form a compound curve should not be widely different ($\leq$ 2:1)



R_1, R_2 = radii of simple curves forming compound curve
Δ_1, Δ_2 = intersection angles of simple curves
Δ = intersection angle of compound curve
t_1, t_2 = tangent lengths of simple curves
T_1, T_2 = tangent lengths of compound curve
PCC = point of compound curve
PI = point of intersection
PC = point of curve
PT = point of tangent

Table 15.10 Lengths of Circular Arc for Different Compound Curve Radii

Minimum Length of Circular Arc (ft)	*Radius (ft)* 100	150	200	250	300	400	500 or more
Acceptable	40	50	60	80	100	120	140
Desirable	60	70	90	120	140	180	200

. In Figure 15.22, R_1 and R_2 are usually known. The following equations can be used to determine the remaining variables.

$$\Delta = \Delta_1 + \Delta_2 \tag{15.29}$$

$$t_1 = R_1 \tan\frac{\Delta_1}{2} \tag{15.30}$$

$$t_2 = R_2 \tan\frac{\Delta_2}{2} \tag{15.31}$$

$$\frac{\overline{VG}}{\sin\Delta_2} = \frac{\overline{VH}}{\sin\Delta_1} = \frac{t_1 + t_2}{\sin(180 - \Delta)} = \frac{t_1 + t_2}{\sin\Delta} \tag{15.32}$$

$$T_1 = \overline{VG} + t_1 \tag{15.33}$$

$$T_2 = \overline{VH} + t_2 \tag{15.34}$$

where

R_1 and R_2 = radii of simple curves forming the compound curve
Δ_1 and Δ_2 = intersection angles of simple curves
t_1 and t_2 = tangent lengths of simple curves
T_1 and T_2 = tangent lengths of compound curves
Δ = intersection angle of compound curve

Example 15.7 Design of a Compound Curve

Figure 15.23 illustrates a compound curve that is to be set out at a highway intersection. If the point of compound curve (PCC) is located at station (565 + 35), determine the deflection angles for setting out the curve.

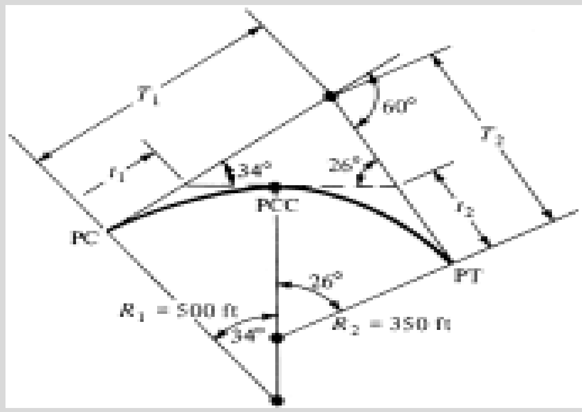


Figure 15.23 Compound Curve for Example 15.7

Solution:

$$t_1 = 500 \tan\frac{34}{2} = 152.87 \text{ ft}$$

$$t_2 = 350 \tan\frac{26}{2} = 80.80 \text{ ft}$$

For length of horizontal curve of 500 ft radius,

$$L = R\Delta_1\frac{\pi}{180} = 500 \times \frac{34\pi}{180}$$
$$= 296.71 \text{ ft}$$

For length of horizontal curve of 350 ft radius,

$$L = R\Delta_2\frac{\pi}{180} = 350 \times \frac{26\pi}{180}$$
$$= 158.82 \text{ ft}$$

Therefore, station of the PC is equal to (565 + 35.00) − (2 + 96.71) = 562 + 38.29.
The station of the PT is equal to (565 + 35.00) + (1 + 58.82) = 566 + 93.82.
For curve of 500 ft radius,

$$\frac{D}{2} = \frac{5729.6}{2 \times 500} = 5°43'47'' \qquad \text{(from Eq. 15.20)}$$

$$l_1 = (563 + 00) - (562 + 38.29) = 61.71 \text{ (ft)}$$

$$\frac{\delta_1}{l_1} = \frac{\Delta}{L}$$

$$\frac{\delta_1}{2} = \frac{61.71 \times 34}{2 \times 296.71} = 3°32'8''$$

$$l_2 = (565 + 35.00) - (565 + 00) = 35 \text{ (ft)}$$

$$\frac{\delta_2}{2} = \frac{35 \times 34}{2 \times 296.71} = 2°0'19''$$

For curve of 350 ft radius,

$$\frac{D}{2} = \frac{5729.6}{2 \times 350} = 8°11'7''$$

$$l_1 = (566 + 00) - (565 + 35.00) = 65 \text{ ft}$$

$$\frac{\delta_1}{2} = \frac{65 \times 26}{2 \times 158.82} = 5°19'16''$$

$$l_2 = (566 + 93.82) - (566 + 00) = 93.82 \text{ ft}$$

$$\frac{\delta_2}{2} = \frac{93.82 \times 26}{2 \times 158.8} = 7°40'44''$$

Computation of the deflection angles are in Table 15.11.

The deflection angles for the 350 ft radius curve are turned from the common tangent with the transit located at PCC. Since each simple curve is relatively flat, calculated lengths of the chords are almost equal to the corresponding arc lengths.

Table 15.11 Computations for Example 15.7

Station	*500 ft Radius Curve Deflection Angle*	*Chord Length (ft)*
PC 562 + 38.29	0	0
563	3°32′8″	61.66′
564	9°15′55″	99.84′
565	14°59′42″	99.84′
PCC 565 + 35.00	17°00′00″	35.00′
Station	*350 ft Radius Curve Deflection Angle*	*Chord Length (ft)*
PCC 565 + 35.00	0	0
566	5°19′16″	64.9′
PT 566 + 93.82	13°00′00″	93.5′

Example 15.7 Design of a Compound Curve

Figure 15.23 illustrates a compound curve that is to be set out at a highway intersection.

If the point of compound curve (PCC) is located at station (565+10.5), determine the deflection angles for setting out the curve.

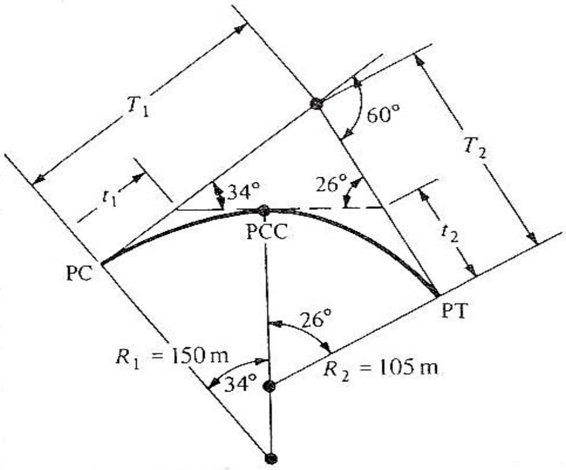


Figure 15.23 Compound Curve for Example 15.7

Solution:

$$t_1 = 150 \tan\frac{34}{2} = 45.86 \text{ m}$$

$$t_2 = 105 \tan\frac{26}{2} = 24.24 \text{ m}$$

For length of horizontal curve of 150 m radius,

$$L = R\Delta_1\frac{\pi}{180} = 150 \times \frac{34\pi}{180}$$
$$= 89.01 \text{ m}$$

For length of horizontal curve of 105 m radius,

$$L = R\Delta_2\frac{\pi}{180} = 105 \times \frac{26\pi}{180}$$
$$= 47.65 \text{ m}$$

Therefore, station of the PC is equal to (565 + 10.50) − (2 + 29.01) = 562 + 11.49.
The station of the PT is equal to (565 + 10.50) + (1 + 17.65) = 566 + 28.15.
For curve of 500 ft radius,

$$\frac{D}{2} = \frac{1719}{2 \times 150} = 5°43'47'' \qquad \text{(from Eq. 15.20)}$$

$$l_1 = (563 + 00) - (562 + 11.49) = 18.51 \text{ m}$$

$$\frac{\delta_1}{l_1} = \frac{\Delta}{L}$$

$$\frac{\delta_1}{2} = \frac{18.51 \times 34}{2 \times 89.01} = 3°32'8''$$

$$l_2 = (565 + 10.50) - (565 + 00) = 10.5\ (\text{m})$$

$$\frac{\delta_2}{2} = \frac{10.5 \times 34}{2 \times 89.01} = 2°0'19''$$

For curve of 105 m radius,

$$\frac{D}{2} = \frac{17.19}{2 \times 105} = 8°11'7''$$

$$l_1 = (566 + 00) - (565 + 10.50) = 19.5\ \text{m}$$

$$\frac{\delta_1}{2} = \frac{19.5 \times 26}{2 \times 47.65} = 5°19'16''$$

$$l_2 = (566 + 28.15) - (566 + 00) = 28.15$$

$$\frac{\delta_2}{2} = \frac{28.15 \times 26}{2 \times 47.65} = 7°40'44''$$

Computation of the deflection angles are in Table 15.11.

The deflection angles for the 105 m radius curve are turned from the common tangent with the transit located at PCC. Since each simple curve is relatively flat, calculated lengths of the chords are almost equal to the corresponding arc lengths.

Table 15.11 Computations for Example 15.7

Station	*150 m Radius Curve Deflection Angle*	*Chord Length (m)*
PC 562 + 11.49	0	0 m
563	3°32′8″	18.51 m
564	9°15′55″	29.95 m
565	14°59′42″	29.95 m
PCC 565 + 10.5	17°00′00″	10.5 m
Station	*105 m Radius Curve Deflection Angle*	*Chord Length (m)*
PCC 565 + 10.5	0	0 m
566	5°19′16″	19.5 m
PT 566 + 28.15	13°00′00″	28.15 m

Reverse Curves

- ☒ Seldom recommended because sudden changes to the alignment result in difficulty to keep lanes
- ✓ When it is necessary → a preferable design separate by a sufficient length of tangent/ spiral

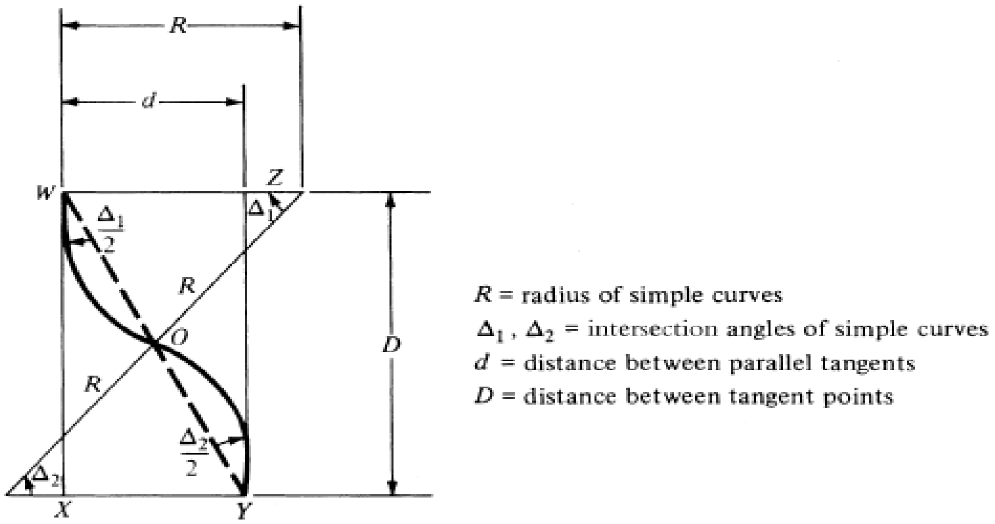


Figure 15.24 Geometry of a Reverse Curve with Parallel Tangents

If d and D are known, the following variables can be determined.

$$\Delta = \Delta_1 = \Delta_2$$

$$\text{Angle } OWX = \frac{\Delta_1}{2} = \frac{\Delta_2}{2}$$

$$\text{Angle } OYZ = \frac{\Delta_1}{2} = \frac{\Delta_2}{2}$$

Therefore, WOY is a straight line, and hence,

$$\tan\frac{\Delta}{2} = \frac{d}{D}$$

$$d = R - R\cos\Delta_1 + R - R\cos\Delta_2$$

$$= 2R(1 - \cos\Delta)$$

$$R = \frac{d}{2(1 - \cos\Delta)}$$

If d and R are known,

$$\cos\Delta = 1 - \frac{d}{2R}$$

$$D = d\cos\frac{\Delta}{2}$$

Transition Curves (Spiral)

Placed between:

- tangents and circular curves
- between two adjacent circular curves having substantially different radii

Used to provide:

- Vehicle path that gradually increases or decreases the radial force as the vehicle enters or leaves a circular curve
- More safety and control
- Means of superelevation before circular curve
- Improve appearance

Length of Spiral Curves

❖ Tangent/circle: Curvature ($0 – D_a$)

❖ Circle/Circle: Curvature (D_{a1} - D_{a2})

$$L_{s,\ min} = \frac{0.0215u^3}{RC}$$

Use Larger Value

$$L_{s,\ min} = \sqrt{24(P_{min})R}$$

$$L_{s,min} = \frac{3.15u^3}{RC} \quad (15.37)$$

$$L_{s,min} = \sqrt{24(p_{min})R} \quad (15.38)$$

L_s = minimum length of curve (ft)
u = speed (mi/h)
R = radius of curve (ft)
C = rate of increase of radial acceleration (ft/sec²/sec). Values range from 1 to 3
P_{min} = minimum lateral offset between the tangent and the circular curve (0.66 ft)

where

L_s = minimum length of curve (m)
u = speed km/h
R = radius of curve (m)
C = rate of increase of radial acceleration (m/sec^2/sec) Values range from **0.3 – 1 m/sec²/sec**
P_{min} = minimum lateral offset between the tangent and the circular curve (0.2 m)

Table 4.1 *Desirable Length of Spiral Curve Transition*

SI units		customary U.S. units	
design speed (kph)	spiral length (m)	design speed (mph)	spiral length (ft)
20	11	15	44
30	17	20	59
40	22	25	74
50	28	30	88
60	33	35	103
70	39	40	117
80	44	45	132
90	50	50	147
100	56	55	161
110	61	60	176
120	67	65	191
130	72	70	205
		75	220
		80	235

Figure 4.13 *Detail Elements of a Transition Spiral*

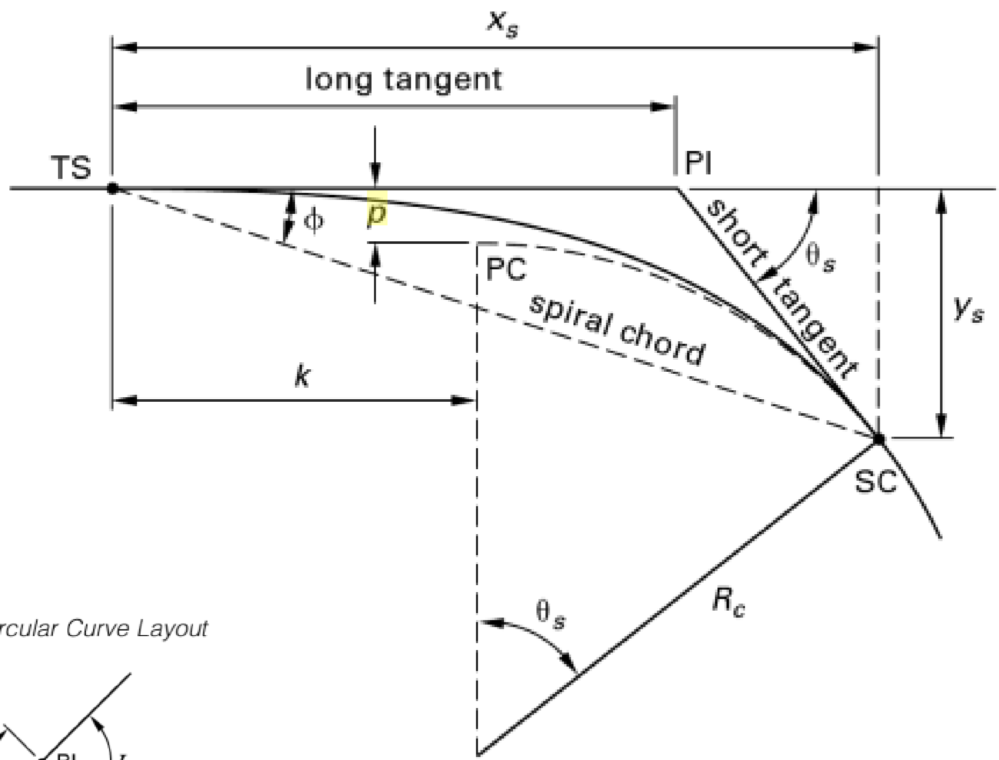
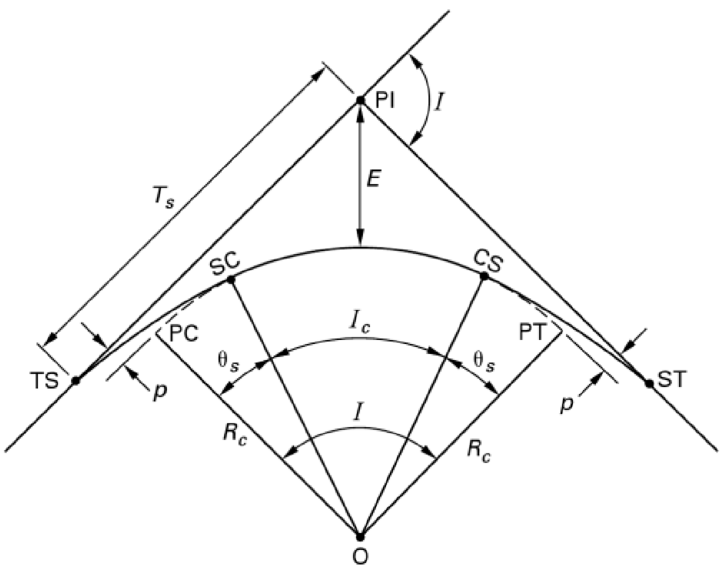


Figure 4.12 *Fully Spiraled Circular Curve Layout*



Length of Superelevation Runoff when Spiral Curves Are Not Used

- ☒ Sometimes no Spiral / Tangent-to-curve transition
- ✓ Superelevation transition length is comprised of superelevation runoff and tangent runout

$$L_r = \frac{(wn_1)e_d}{\Delta}(b_w)$$

L_r = minimum length of superelevation runoff

Δ = maximum relative gradient (%) (0.78% @ 24 km/h to 0.35% @ 128 km/h)

n_1 = number of lanes rotated

b_w = adjustment factor for number of lanes rotated (1_1.00, 2_0.75, 3_0.67)

w = width of one traffic lane (m) (typically 3.6 m)

e_d = design superelevation rate (%)

Superelevation **runoff** is defined as the distance over which the pavement cross slope on the outside lane changes from zero (flat) to full superelevation of the curve (e).

- ✓ Runoff length is divided between the tangent and the curved section
- ☒ Avoids placing the runoff either entirely on the tangent or the curve

Length of *tangent* **runout** consists of the length of roadway needed to accomplish a change on the outside-lane cross slope from normal (i.e., 2 percent) to zero, or vice versa.

Runoff + Runout = Distance over which transition from normal crown to full superelevation

$$L_t = \frac{e_{NC}}{e_d} L_r$$

where

L_t = minimum length of tangent runout (m)
e_{NC} = normal cross slope rate (%)
e_d = design superelevation rate (%)
L_r = minimum length of superelevation runoff (m)

Attainment of Superelevation. It is essential that the change from a crowned cross section to a superelevated one be achieved without causing any discomfort to motorists or creating unsafe conditions. One of **four** methods can be used to achieve this change on **undivided** highways.

1. A crowned pavement is rotated about the profile of the centerline.

2. A crowned pavement is rotated about the profile of the inside edge.

3. A crowned pavement is rotated about the profile of the outside edge.

4. A straight cross-slope pavement is rotated about the profile of the outside edge.

Table 15.12 Superelevation Runoff L_r (ft) for Horizontal Curves

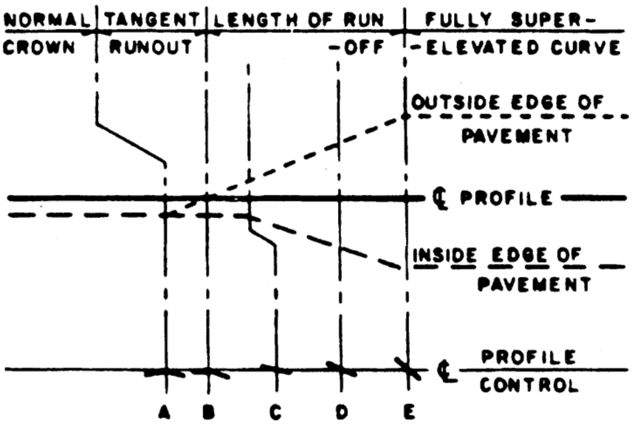
e (%)	*Design Speed (mph)*													
	20		*30*		*40*		*50*		*60*		*70*		*80*	
	1	2	1	2	1	2	1	2	1	2	1	2	1	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2.0	32	49	36	55	41	62	48	72	53	80	60	90	69	103
3.0	49	73	55	82	62	93	72	108	80	120	90	135	103	154
4.0	65	97	73	109	83	124	96	144	107	160	120	180	137	206
5.0	81	122	91	136	103	155	120	180	133	200	150	225	171	257
6.0	97	146	109	164	124	186	144	216	160	240	180	270	207	309
7.0	114	170	127	191	145	217	168	252	187	280	210	315	240	360
8.0	130	195	145	218	166	248	192	288	213	320	240	360	274	411
9.0	146	219	164	245	186	279	216	324	240	360	270	405	309	463
10.0	162	243	182	273	207	310	240	360	267	400	300	450	343	514
11.0	178	268	200	300	228	341	264	396	293	440	330	495	377	566
12.0	195	292	218	327	248	372	288	432	320	480	360	540	411	617

Note: (1) Two-lane – 12 ft 2% cross slope
(2) Multilane – 12 ft each direction rotated separately

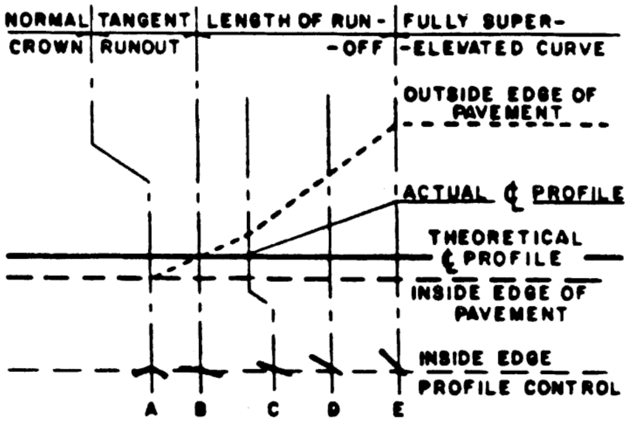
Table 15.13 Tangent Runout Length for Spiral Curve Transition Design

	Tangent Runout Length (ft)				
	Superelevation Rate (%)				
Design Speed (mi/h)	*2*	*4*	*6*	*8*	*10*
15	44	—	—	—	—
20	59	30	—	—	—
25	74	37	25	—	—
30	88	44	29	—	—
35	103	52	34	26	—
40	117	59	39	29	—
45	132	66	44	33	—
50	147	74	49	37	—
55	161	81	54	40	—
60	176	88	59	44	—
65	191	96	64	48	38
70	205	103	68	51	41
75	220	110	73	55	44
80	235	118	78	59	47

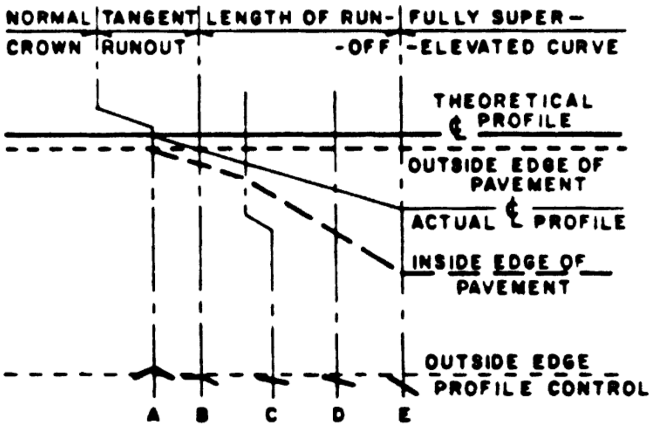
Note: (1) Values for $e = 2\%$ represent the desirable lengths of the spiral curve transition.
(2) Values shown for tangent runout should also be used as the minimum length of the spiral transition curve.



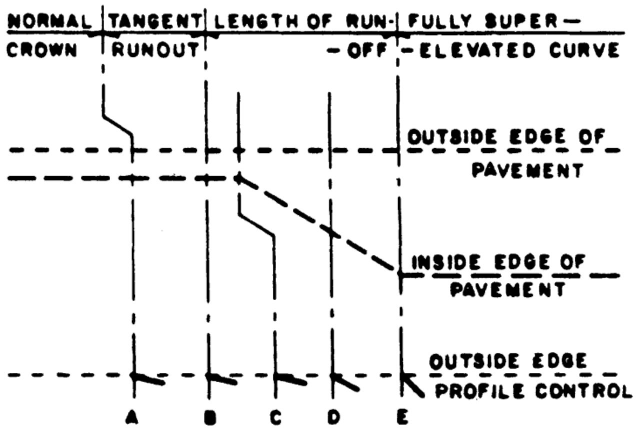
(a) Crowned pavement revolved about centerline



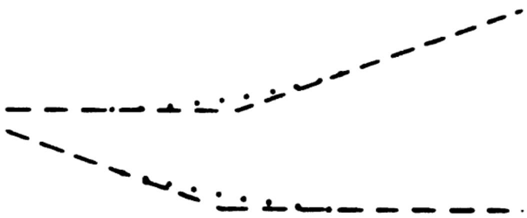
(b) Crowned pavement revolved about inside edge



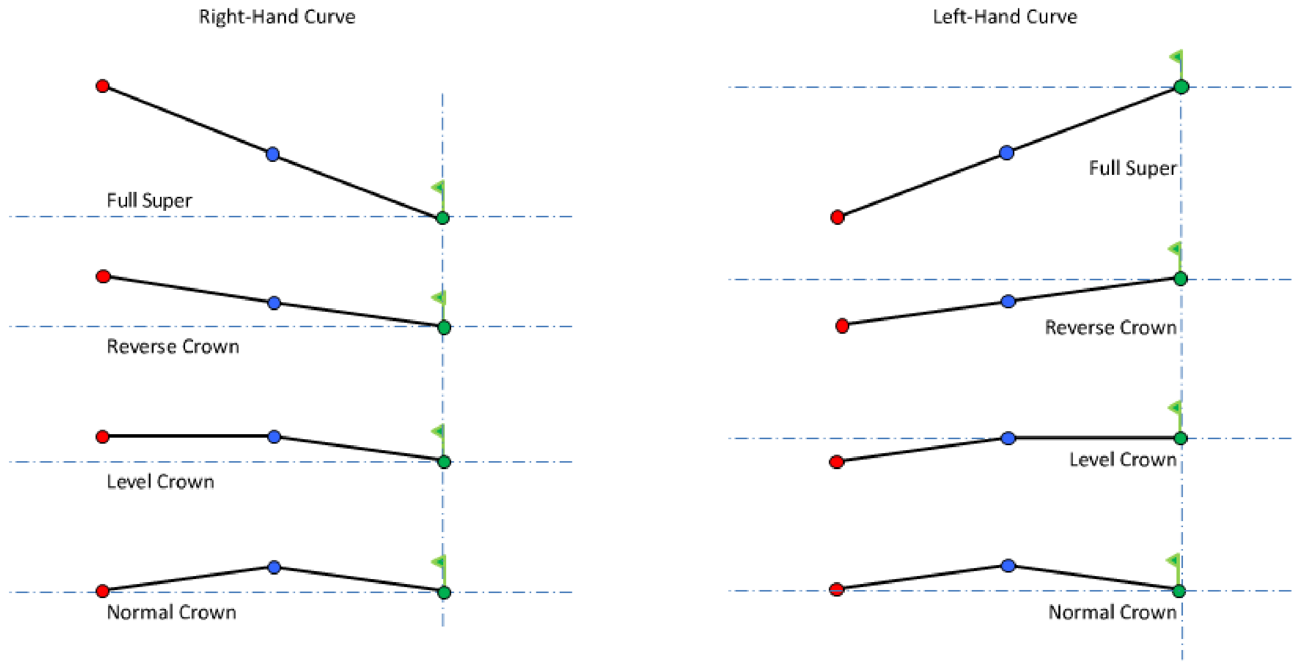
(c) Crowned pavement revolved about outside edge



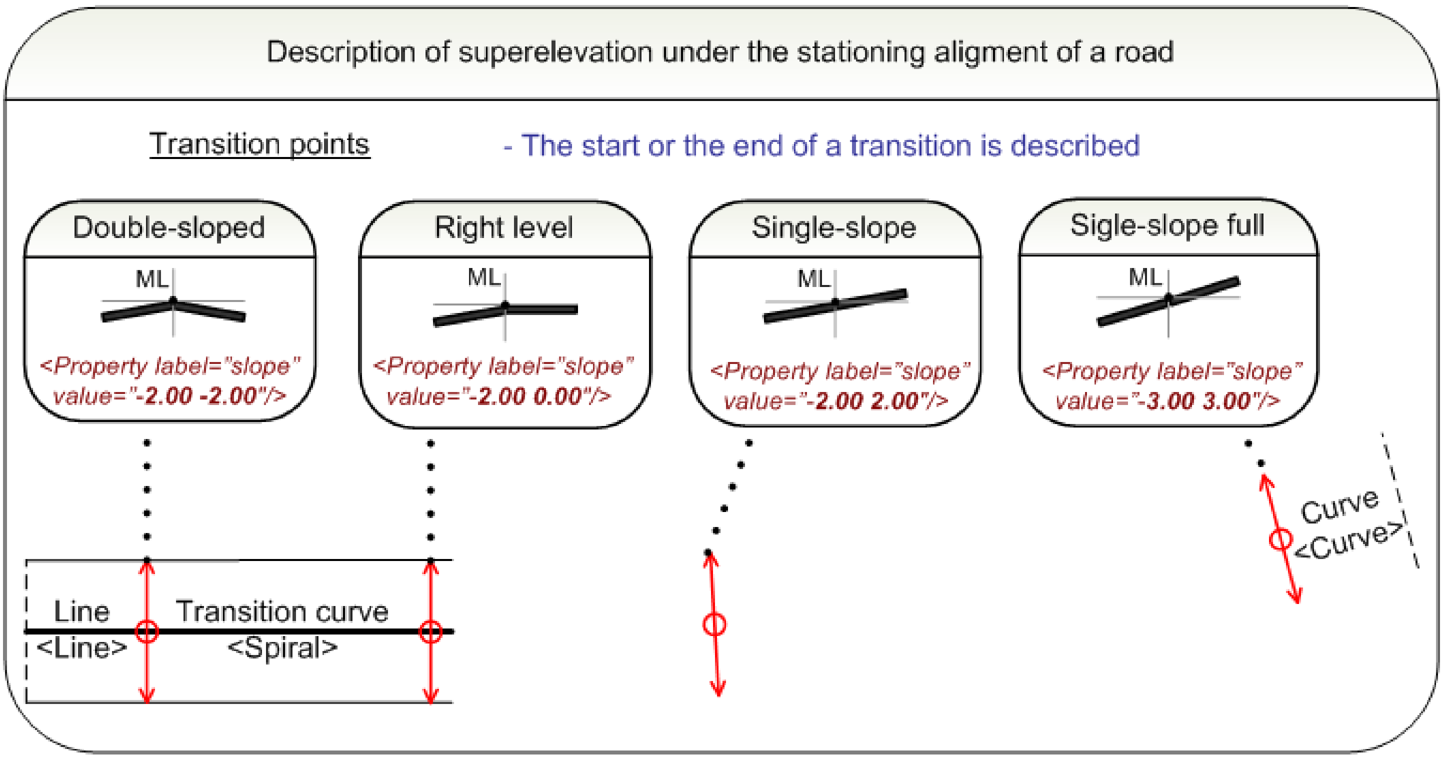
(d) Straight cross slope pavement revolved about outside edge



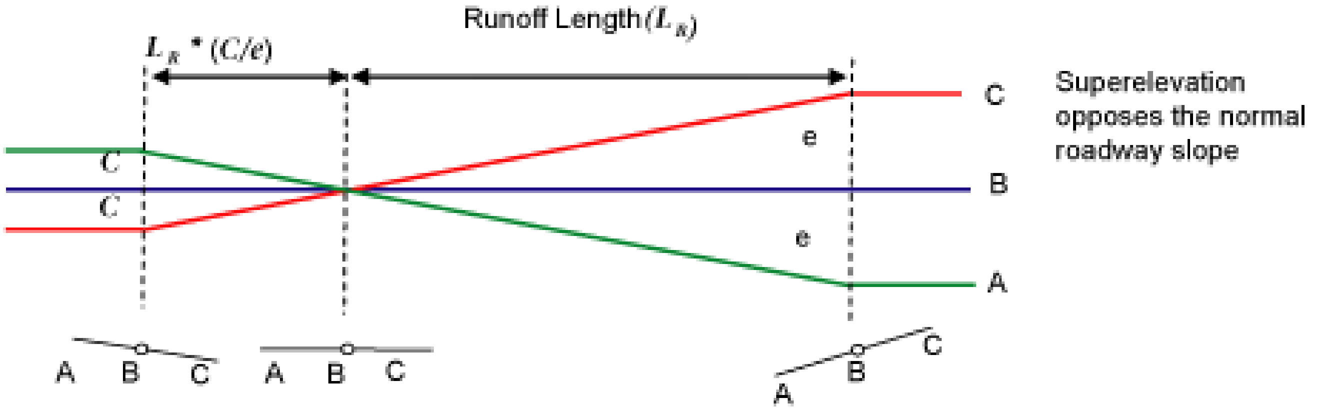
(e) Angular breaks appropriately rounded (dotted lines)

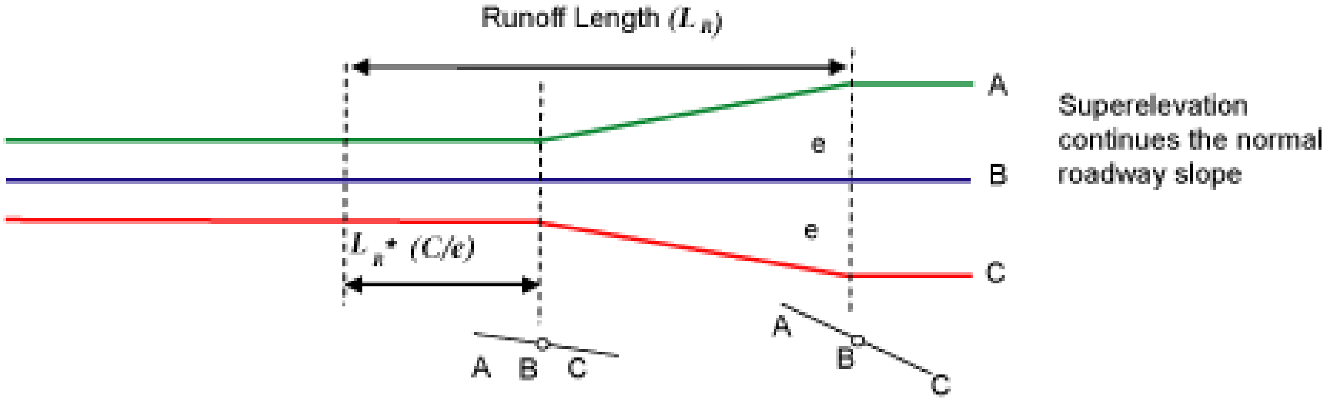


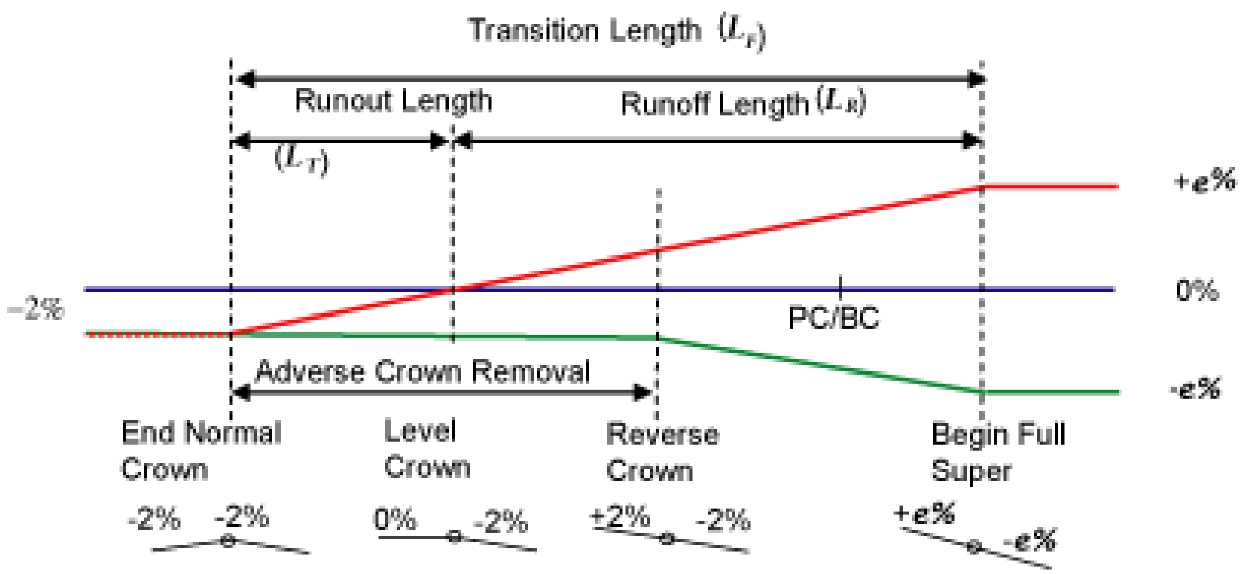
Right-Hand Curve
Full Super
Reverse Crown
Level Crown
Normal Crown
Left-Hand Curve
Full Super
Reverse Crown
Level Crown
Normal Crown



Description of superelevation under the stationing aligment of a road
Transition points
- The start or the end of a transition is described
Double-sloped
ML
<Property label="slope" value="-2.00 -2.00"/>
Right level
ML
<Property label="slope" value="-2.00 0.00"/>
Single-slope
ML
<Property label="slope" value="-2.00 2.00"/>
Sigle-slope full
ML
<Property label="slope" value="-3.00 3.00"/>
Line
<Line>
Transition curve
<Spiral>
Curve
<Curve>


Runoff Length (L_R)
L_R * (C/e)
Superelevation opposes the normal roadway slope
C
B
A
e
e
A B C
A B C
A B C


Runoff Length (L_R)
A
B
C
e
e
Superelevation continues the normal roadway slope
L_R* (C/e)
A B C
A B C


Transition Length (L_T)
Runout Length (L_T)
Runoff Length (L_R)
+e%
0%
-e%
-2%
PC/BC
Adverse Crown Removal
End Normal Crown
Level Crown
Reverse Crown
Begin Full Super
-2% -2%
0% -2%
+2% -2%
+e% -e%

Curve Radii Based on Stopping Sight Distance

$$R = \frac{u^2}{127(e + f_s)}$$

$$\frac{2R\theta\pi}{180} = S$$

and

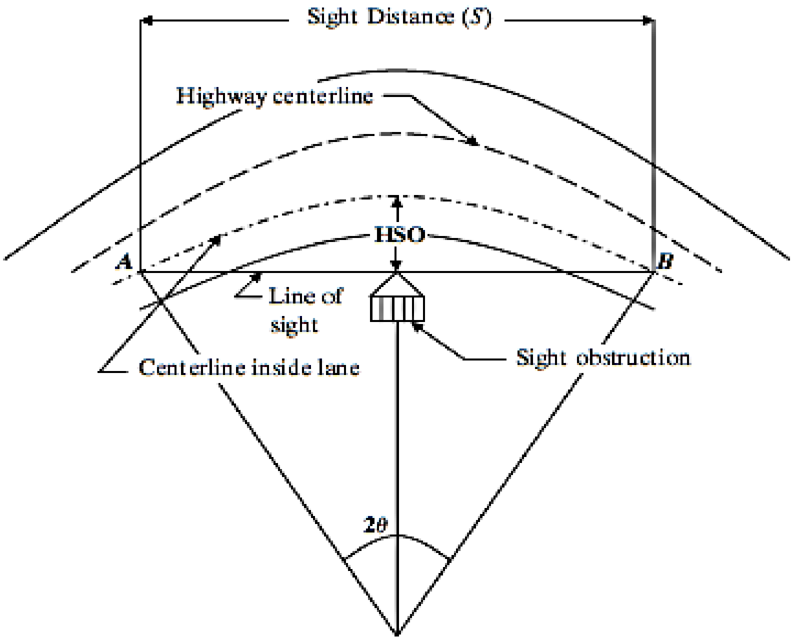
$$\theta = \frac{180(S)}{2\pi R} = \frac{28.65}{R}(S) \tag{15.41}$$

where

R = radius of horizontal curve
S = stopping sight distance
θ = one half central angle

From Figure 15.26(a), we can write

$$\frac{R - m}{R} = \cos\,\theta \tag{15.42}$$



Equating the cosines of θ from Eqs. 15.41 and 15.42 yields

$$\cos\frac{28.65}{R}(S) = \frac{R - m}{R}$$

where m = the Horizontal Sightline Offset, HSO (ft). Solving for m produces the following relationship:

$$m = R\left(1 - \cos\frac{28.65}{R}S\right) \tag{15.43}$$

Eq. 15.43 can be used to determine m, R, or S, depending on which two of the three variables are known.

Example 15.8 Location of Object Near a Horizontal Curve

A horizontal curve with a radius of 240 m connects the tangents of a two-lane highway that has a posted speed limit of 56 km/h If the highway curve is not super-elevated, e = 0, determine the horizontal sightline offset (HSO) that a large bill-board can be placed from the centerline of the inside lane of the curve, without reducing the required SSD. Perception-reaction time is 2.5 sec, and $f = 0.35$.

Solution:

- Determine the required SSD.

$$\mathbf{SSD = 0.278\ ut + u^2/254(f \pm G)}$$

$$0.278\,(56)(2.5) + \frac{(56)^2}{254(0.35)} = 74.2\ \text{m}$$

- Determine m using Eq. 15.39.

$$m = 240\left[1 - \cos\left(\frac{28.65}{240}(74.2)\right)\right] = 74.2(1 - 0.988)\ \text{m}$$

$$= 2.86\ \text{m}$$

Example 15.8 Location of Object Near a Horizontal Curve

A horizontal curve with a radius of 800 ft connects the tangents of a two-lane highway that has a posted speed limit of 35 mi/h. If the highway curve is not superelevated, $e = 0$, determine the horizontal sightline offset (HSO) that a large billboard can be placed from the centerline of the inside lane of the curve, without reducing the required SSD. Perception-reaction time is 2.5 sec, and $f = 0.35$.

Solution:

- Determine the required SSD.

$$\text{SSD} = 1.47\ \text{ut} + \text{u}^2/30\ (\text{f} \pm \text{G})$$

$$(1.47 \times 35 \times 25) + \frac{(35)^2}{30(0.35)} = 245.29\ \text{ft}$$

- Determine m using Eq. 15.39.

$$m = 800\left[1 - \cos\left(\frac{28.65}{800}(245.29)\right)\right] = 800(1 - 0.988)\ \text{ft}$$

$$= 9.6\ \text{ft}$$

Check solution using Fig. 15.26(b).
For $R = 800$ and $V = 35$ mph from Figure 15.26(b) m is estimated to be 9.5 ft.

Extra Widening on Curve

1. Mechanical widening

at a horizontal curve as shown in figure 15.5. Let R_1 is the radius of the outer track line of the rear wheel, R_2 is the radius of the outer track line of the front wheel l is the distance between the front and rear wheel, n is the number of lanes, then the mechanical widening W_m (refer figure 15:1) is derived below:

$$\begin{aligned} R_2^2 &= R_1^2 + l^2 \\ &= (R_2 - W_m)^2 + l^2 \\ &= R_2^2 - 2R_2W_m + W_m^2 + l^2 \\ 2R_2W_m - W_m^2 &= l^2 \end{aligned}$$

Therefore the widening needed for a single lane road is:

$$W_m = \frac{l^2}{2R_2 - W_m} \tag{15.2}$$

If the road has n lanes, the extra widening should be provided on each lane. Therefore, the extra widening of a road with n lanes is given by,

$$W_m = \frac{nl^2}{2R_2 - W_m} \tag{15.3}$$

Please note that for large radius, $R_2 \approx R$, which is the mean radius of the curve,then W_m is given by:

$$W_m = \frac{nl^2}{2R} \tag{15.4}$$

2. Psychological Widening:

$$W_{ps} = \frac{u}{9.5\sqrt{R}}$$

u= design speed km/h

R= curve radius

$$W_{total} = W_m + W_{ps}$$

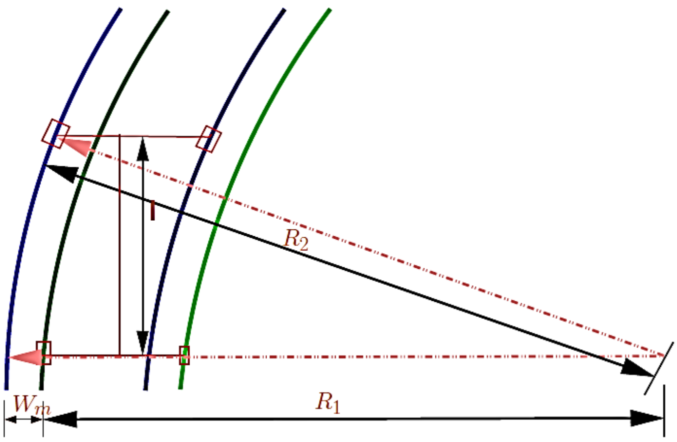
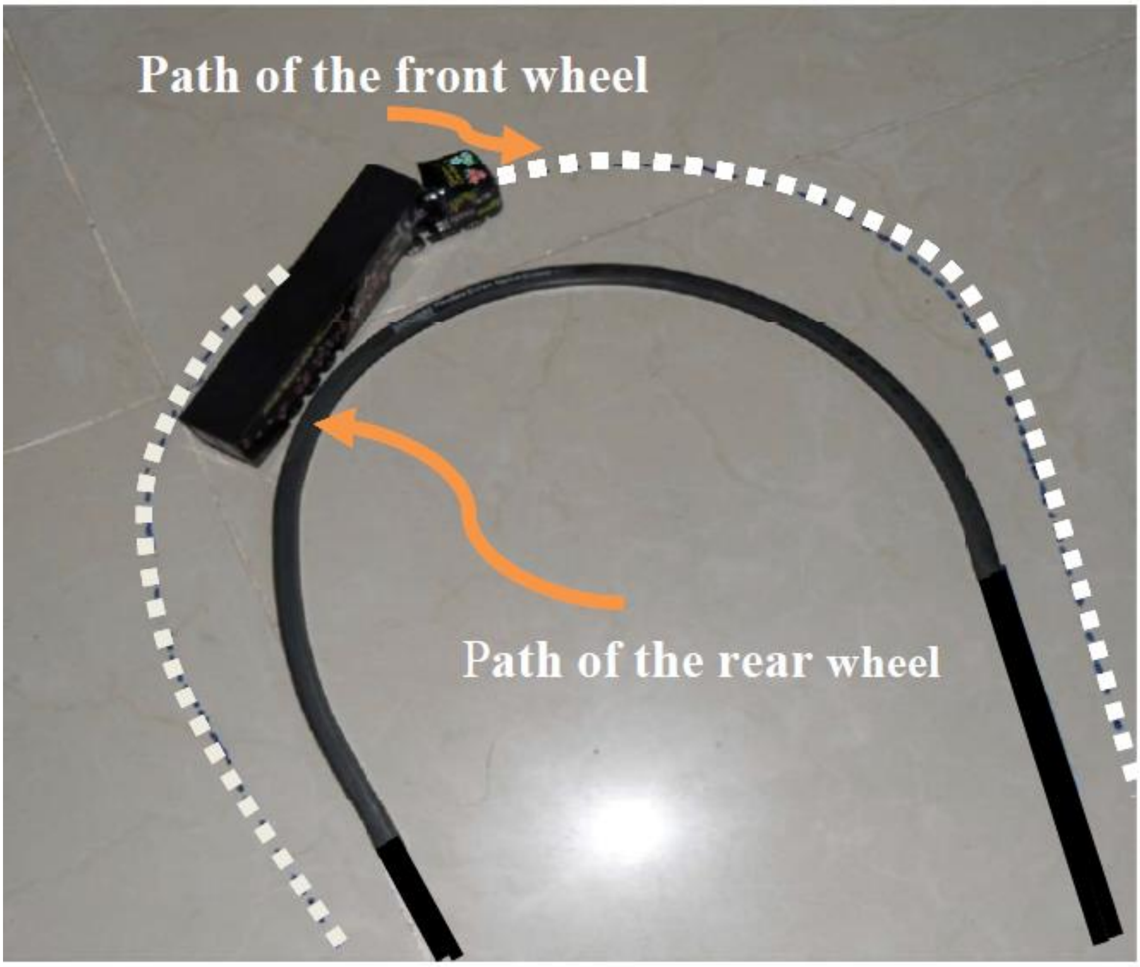


Figure 15:1: Extra-widening at a horizontal curve



Example: find the widening necessary for a horizontal curve having R=100m, wheel base =6.1m, pavement width = 7m, V= 70 km/hr.

$$W = \text{total widening} = W_{ps} + W_m = \frac{V}{9.5\sqrt{R}} + \frac{nl^2}{2R}$$

$$= \frac{70}{9.5\sqrt{100}} + \frac{2(6.1)^2}{2X100} = 0.74 + 0.37 = 1.11\text{m}$$

If the radius R = 50m , the widening will be = 1.8m (more widening is required)

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Transportation Engineering

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2017 – 2018

هندسة المواصلات

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BITUMINOUS MATERIALS (Asphalts or Tars)

Hydrocarbons :

- Natural deposits
- Product of the distillation of crude petroleum. The bituminous materials used in highway construction are either.

- Bitumen has strong adhesive properties
- Colors: ranging from dark brown to black
- Consistency: from liquid to solid; (liquids, semisolids, and solids)

❖ The solid form is usually hard and brittle at *normal* temperatures but will flow when subjected to long, continuous loading

❖ The liquid form is obtained from the semisolid or solid forms by heating, dissolving in solvents, or breaking the material into minute particles and dispersing them in water with an emulsifier to form an asphalt emulsion

Petroleum Asphalt (See the figure)

BITUMINOUS BINDERS:

- Asphalt cement
- Asphalt cutbacks
- Emulsified asphalt

❖ Blown asphalt and road tars are also other types of bituminous material that now are not used commonly in *highway* construction.

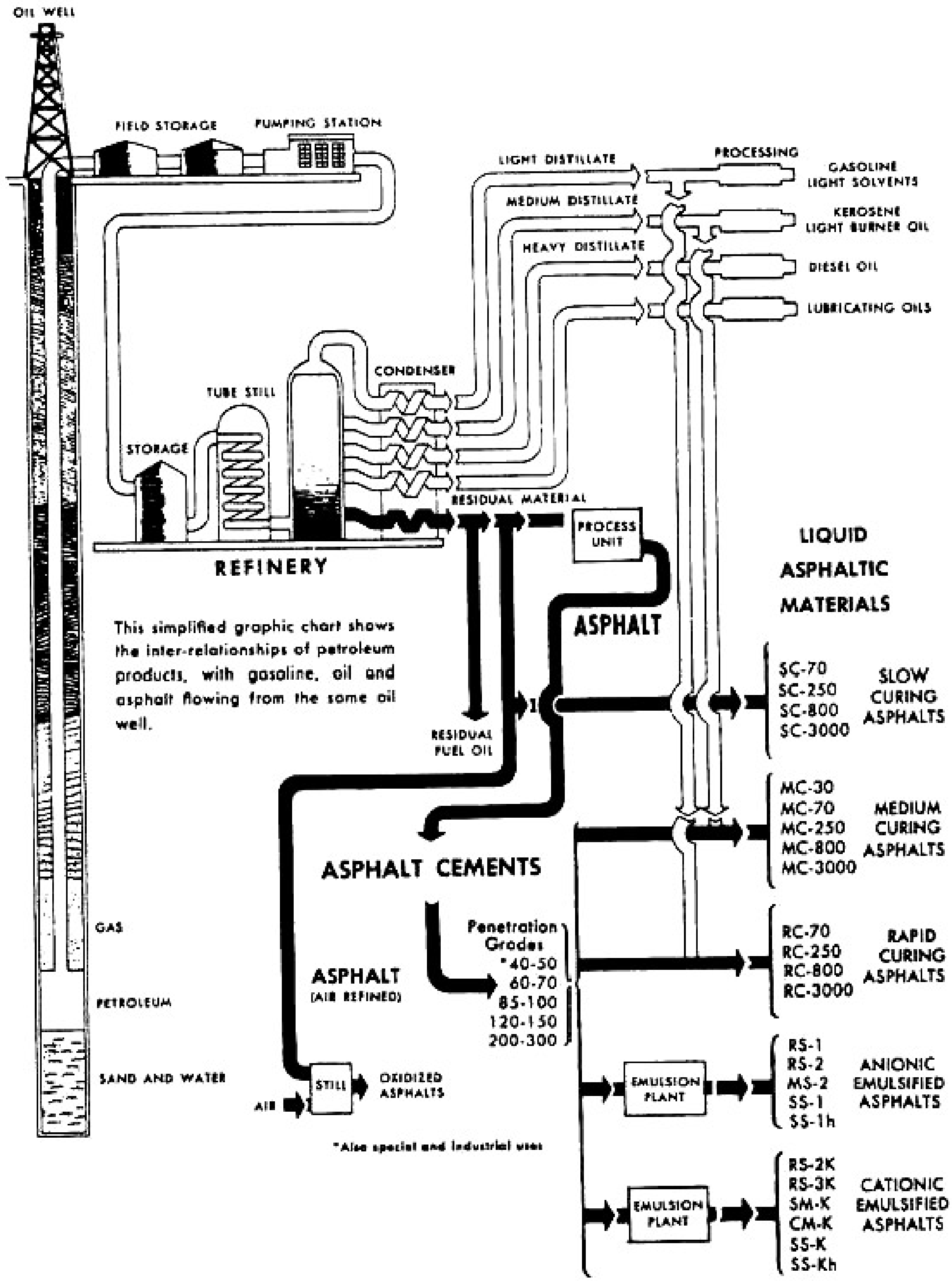

Figure 18.1 A Schematic Example of a Petroleum Distilling Plant

ASPHALT CEMENTS

- Obtained after separation of the lubricating oils
- Semisolid hydrocarbons
- Good cementing agents
- Very viscous

❖ When used as a binder for aggregates in pavement construction, it is necessary to heat both aggregates and the asphalt cement prior to mixing the two materials

✪ Grade → penetration and viscosity

Penetration:

- Penetration is the distance in 0.1 mm that a standard needle will penetrate a given sample under specific conditions of loading, time, and temperature .
- Softest grade used for highway pavement construction has a penetration value of 200 to 300
- Hardest: 60 to 70

✓ Used in: Hot-mix, hot-laid asphalt concrete

ASPHALT CUTBACKS

- Slow-curing asphalts (SC)
- Medium-curing cutback asphalts (MC)
- Rapid-curing cutback asphalts (RC)

Used in:

✓ Cold-laid plant mixes
✓ Road mixes (mixed-in-place(
✓ Surface treatments

Slow-Curing Asphalts (SC)

<u>Obtained:</u>

- Directly as slow-curing straight run asphalts
- Slow-curing cutback asphalts by “cutting back” asphalt cement with a heavy distillate such as diesel oil

❖ Lower viscosities than asphalt cement
❖ Very slow to harden
❖ SC-70, SC-250, SC-800, or SC-3000

- Numbers relate to the approximate kinematic viscosity in centistokes at 60°C (140°F)

Medium-Curing Cutback Asphalts (MC)

- Produced by: fluxing, or cutting back, the residual asphalt (usually 120 to 150 penetration) with light fuel oil or kerosene
- ❑ Medium → Medium volatility of the kerosene-type diluter used
- ❑ MC hardens faster than slow-curing liquid asphalts, although consistencies of the different grades are similar to those of the slow-curing asphalts
- ❑ MC-30 is a unique grade in this series as it is very fluid and has no counterpart in the SC and RC series

☞ Fluidity of MC depends on the amount of solvent in the material.
For example :

- MC-3000 has 20 % of solvent by volume ,
- MC-70 has up to 45%
- MC can be used for the construction of pavement bases, surfaces, and surface treatments

Rapid-Curing Cutback Asphalts (RC)

- ❑ Produced by blending asphalt cement with a petroleum distillate that will evaporate easily

→ Quick change from the liquid form at the time of application to the consistency of the original asphalt cement

- ❑ Solvent: *Gasoline* (Benzene) or naphtha

☞ The grade of rapid-curing asphalt required dictates the amount of solvent to be added to the residual asphalt cement
For example:

- RC-3000 requires about 15% of distillate
- RC-70 requires about 40%
- Used for jobs similar to those for which the MC series is used

Emulsified Asphalts

- ❑ Produced by breaking asphalt cement (usually of 100 to 250 penetration range) into minute particles and dispersing them in water with an emulsifier .
- ☞ These minute particles have like-electrical charges and therefore do not coalesce .
- ☞ They remain in suspension in the liquid phase as long as the water does not evaporate or the emulsifier does not break .
- ❖ Consist of:
- Asphalt about 55 to 70 % by weight
- Water
- Emulsifying agent
- Stabilizer (some cases)

- ❑ <u>Classified as</u>: anionic (Negatively charged), cationic (Positively charged), or nonionic (neutral)
- ❖ Anionic and Cationic are used in highway maintenance and construction
- ❖ Nonionic may be used more frequently in the future as emulsion technology advances

- ❑ <u>Divided into subgroups:</u>
- Rapid-setting (RS), medium-setting (MS), and slow-setting (SS)
- ☞ A cationic → "C" in front of the emulsion type
- ☞ No letter is placed in front of anionic and nonionic emulsions
  For example :
- CRS-2 denotes a cationic emulsion
- RS-2 denotes either an anionic or nonionic emulsion

- ✓ Used in: cold-laid plant mixes and road mixes (mixed- in-place) for several purposes, including the construction of highway pavement surfaces and bases and in surface treatments .
- ✪ Since anionic emulsions contain negative charges, they are more effective in treating aggregates containing electropositive charges (such as limestone)
- ✪ Cationic emulsions are more effective with electronegative aggregates (such as those containing a high percentage of siliceous material)

- ☒ Ordinary emulsions must be protected during very cold spells because they will break down if froze

~~**Blown Asphalts**~~

- ❑ Obtained by blowing air through the semisolid residue obtained during the latter stages of the distillation process
- ❑ The material is maintained at a high temperature while air is blown through it
- ❖ Relatively stiff compared to other types of asphalts
- ❖ Can *maintain* a *firm* consistency at the maximum temperature normally experienced when exposed to the environment
- ☒ *Not* used as a paving material
- ✓ Very useful as a *roofing* material, for automobile undercoating, and as a *joint filler* for concrete pavements .

- ✪ If a catalyst is added during the air-blowing process, the material obtained usually will *maintain* its *plastic* characteristics, even at temperatures much lower than that at which ordinary asphalt cement will become brittle .
- ✪ The *elasticity* of catalytically blown asphalt is similar to that of rubber, and it is used for canal lining.

~~**Road Tars**~~

- ❑ Obtained from the destructive distillation of such organic materials as coal
- Their properties are significantly *different* from petroleum asphalts
- More *susceptible* to weather conditions than similar grades of asphalts
- *Set* more quickly when exposed to the atmosphere
- *Rarely* used for highway pavements

- ❑ Road tars also have been classified by AASHTO into 14 grades :
- RT-1 through RT-12
- RTCB-5
- RTCB-6

- ☞ RT-1
- ✪ Lightest consistency

- ✓ Can be used at normal temperatures for prime or tack coat

- ✪ The viscosity of each grade increases as the number designation increases to RT-12, which is the most viscous

- ✓ RTCB-5 and RTCB-6 are suitable for application during cold weather, since they are produced by **cutting back** the specific grade of tar with easily evaporating solvent.

PROPERTIES OF ASPHALT MATERIALS

Four main categories:

- Consistency
- Aging and temperature sustainability
- Rate of curing
- Resistance to water action

Consistency

- Variation of consistency with temperature
- ☞ Variation of Consistency with Temperature
- ☞ Consistency of any asphalt material changes as the temperature varies .
- ☞ Consistency of blown asphalt is affected less by temperature changes than the consistency of regular paving-grade asphalt
- ☞ This property = temperature susceptibility
- ☞ Temperature susceptibility depends on the crude oil

- Consistency at a specified temperature (Tests ….)

Aging and Temperature Sustainability

When asphaltic materials are exposed to environmental elements, natural deterioration gradually takes place, and the materials eventually *lose* their *plasticity* and become *brittle*

This change is caused primarily by *chemical* and *physical* reactions that take place in the material

This natural deterioration of the asphalt material is known as *weathering*

Weathering *must be* minimized as much as possible .

The ability of an asphalt material to resist weathering is described as the *durability* of the material

Some of the factors that influence weathering are:

Oxidation: is chemical reaction that takes place when the asphalt material is attacked by oxygen in the air

- ☞ This chemical reaction causes gradual hardening (eventually *permanent* hardening) and considerable *loss* of the *plastic* characteristics of the material

Volatilization: is the evaporation of the lighter hydrocarbons

- ➔ causes the *loss* of the *plastic* characteristics

Temperature

- The higher the temperature, the higher the rates of oxidation and volatilization
- Rate of organic and physical reactions in the asphalt material ≈ doubles for each 10°C (50°F) increase in temperature

Surface Area

- There is a direct relationship between surface area and rate of oxygen absorption and loss due to evaporation in grams /cm3/minute
- An inverse relationship exists between volume and rate of oxidation and volatilization .
- This means that the rate of hardening is directly proportional to the ratio of the surface area to the volume
- This fact is taken into consideration when asphalt concrete mixes are designed for pavement construction in that the air voids are kept to the practicable minimum required for stability to reduce the area exposed to oxidation

Rate of Curing

Curing is defined as the process through which an asphalt material increases its consistency as it *loses solvent* by evaporation.

Rate of Curing of Cutbacks

- ❑ Rate of curing depends on the distillate used in the cutting-back process.
- ❑ This is an important characteristic of cutback materials, since the rate of curing indicates the time that should elapse before a cutback will attain a consistency that is thick enough for the binder to perform satisfactorily.
- ❑ Rate of curing is affected by both inherent and external factors.

  The important *inherent* factors are:

- Volatility of the solvent:
  - ☞ Gasoline and naphtha → rapid-curing cutbacks
  - ☞ Light fuel oil and kerosene → medium-curing cutback
- Quantity of solvent in the cutback: (More quantity → more time to cure)
- Consistency of the base material:
  - ☞ The higher the penetration of the base asphalt, the longer it takes for the asphalt cutback to cure

The important *external* factors that affect curing rate are:

- Temperature
- Ratio of surface area to volume
- Wind velocity across exposed surface

Rate of Curing for Emulsified Asphalts

Depend on the rate at which the water evaporates from the mixture:

- Weather
- Surface

✓ A major advantage of cationic emulsions is that they release their water more readily.

Resistance to Water Action

- ✓ It is important that asphalt continues to adhere to the aggregates even with the presence of water.
- ☒ If this bond between the asphalt and the aggregates is lost, the asphalt will strip from the aggregates, resulting in the deterioration of the pavement.

- ✓ In *hot-mix, hot-laid* asphalt concrete, where the aggregates are thoroughly dried before mixing, stripping does not occur and so *no preventive action* is usually taken.

- ✪ When water is added to a hot-mix, *cold-laid* asphalt concrete, commercial antistrip additives usually are added to improve the asphalt's ability to adhere to the aggregates.

Temperature Effect on Volume of Asphaltic Materials

- ❑ Volume increases with an increase in temperature
- ❑ The rate of change in volume is given as the coefficient of expansion
- ❑ Because of this variation of volume with temperature, the volumes of asphalt materials usually are given for a temperature of 60°F (15.6°C)
- ❑ Volumes measured at other temperatures are converted to the equivalent volumes at 60°F by using appropriate multiplication

Table 18.1 Specifications for Rapid-Curing Cutback Asphalts

	RC-70		*RC-250*		*RC-800*		*RC-3000*	
	Min.	*Max.*	*Min.*	*Max.*	*Min.*	*Max.*	*Min.*	*Max.*
Kinematic viscosity at 60°C (140°F) (See Note 1) centistokes	70	140	250	500	800	1600	3000	6000
Flash point (Tag, open-cup), °C (°F)	. . .	. . .	27 (80)	. . .	27 (80)	. . .	27 (80)	. . .
Water, percent	. . .	0.2	. . .	0.2	. . .	0.2	. . .	0.2
Distillation test:								
Distillate, percentage by volume of total distillate to 360°C (680°F)								
to 190°C (374ºF)	10	. . .	. . .	. . .	. . .	. . .	. . .	. . .
to 225°C (437ºF)	50	. . .	35	. . .	15	. . .	. . .	. . .
to 260°C (500ºF)	70	. . .	60	. . .	45	. . .	25	. . .
to 315°C (600ºF)	85	. . .	80	. . .	75	. . .	70	. . .
Residue from distillation to 360ºC (680ºF) volume percentage of sample by difference	55	. . .	65	. . .	75	. . .	80	. . .
Tests on residue from distillation:								
Absolute viscosity at 60°C (140°F) (See Note 3) poises	600	2400	600	2400	600	2400	600	2400
Ductility, 5 cm/min at 25°C (77°F) cm	100	. . .	100	. . .	100	. . .	100	. . .
Solubility in trichloroethylene, percent	99.0	. . .	99.0	. . .	99.0	. . .	99.0	. . .
Spot test (See Note 2) with:								
Standard naphtha	Negative for all grades							
Naphtha-xylene solvent, -percent xylene	Negative for all grades							
Heptane-xylene solvent, -percent xylene	Negative for all grades							

Note 1: As an alternate, Saybolt Furol viscosities may be specified as follows:

Grade RC-70—Furol viscosity at 50°C (122°F)—60 to 120 sec.
Grade RC-250—Furol viscosity at 60°C (140°F)—125 to 250 sec.
Grade RC-800—Furol viscosity at 82.2°C (180°F)—100 to 200 sec.
Grade RC-3000—Furol viscosity at 82.2°C (180°F)—300 to 600 sec.

Note 2: The use of the spot test is optional. When specified, the engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement, and also, in the case of the xylene solvents, the percentage of xylene to be used.

Note 3: In lieu of viscosity of the residue, the specifying agency, at its option, can specify penetration at 100 g; 5 s at 25°C (77°F) of 80 to 120 for Grades RC-70, RC-250, RC-800, and RC-3000. However, in no case will both be required.

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Table 18.2 Specifications for Medium-Curing Cutback Asphalts

	MC-30		*MC-70*		*MC-250*		*MC-800*		*MC-3000*	
	Min.	*Max.*	*Min.*	*Max.*	*Min.*	*Max.*	*Min.*	*Max.*	*Min.*	*Max.*
Kinematic Viscosity at 60°C (140°F) (See Note 1) mm^2/s	30	60	70	140	250	500	800	1600	3000	6000
Flash point (Tag, open-cup), °C (F)	38 (100)	—	38 (100)	—	66 (150)	—	66 (150)	—	66 (150)	—
Water percent	—	0.2	—	0.2	—	0.2	—	0.2	—	0.2
Distillation test										
Distillate, percentage by volume of total distillate to 360°C (680°F)										
to 225°C (437°F)	—	25	0	20	0	10	—	—	—	—
to 260°C (500°F)	40	70	20	60	15	55	0	35	0	15
to 315°C (600°F)	75	93	65	90	60	87	45	80	15	75
Residue from distillation to 360°C (680°F) volume percentage of sample by difference	50	—	55	—	67	—	75	—	80	—
Tests on residue from distillation:										
Absolute viscosity at 60°C (140°F) (See Note 4) Pa · s (Poises)	30 (300)	120 (1200)	30 (300)	120 (1200)	30 (300)	120 (1200)	30 (300)	120 (1200)	30 (300)	120 (1200)
Ductility, 5 cm/min, cm (See Note 2)	100	—	100	—	100	—	100	—	100	—
Solubility in Trichloroethylene, percent	99.0	—	99.0	—	99.0	—	99.0	—	99.0	—
Spot test (See Note 3) with:										
Standard naphtha	Negative for all grades									
Naphtha-xylene solvent, percent xylene	Negative for all grades									
Heptane-xylene solvent, percent xylene	Negative for all grades									

Note 1: As an alternate, Saybolt-Furol viscosities may be specified as follows:
Grade MC-70—Furol viscosity at 50°C (122°F)—60 to 120 s
Grade MC-30—Furol viscosity at 25°C (77°F)—75 to 150 s
Grade MC-250—Furol viscosity at 60°C (140°F)—125 to 250 s
Grade MC-800—Furol viscosity at 82.2°C (180°F)—100 to 200 s
Grade MC-3000—Furol viscosity at 82.2°C (180°F)—300 to 600 s

Note 2: If the ductility at 25°C (77°F) is less than 100, the material will be acceptable if its ductility at 15.5°C (60°F) is more than 100.

Note 3: The use of the spot test is optional. When specified, the engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement and also (in the case of the xylene solvents) the percentage of xylene to be used.

Note 4: In lieu of viscosity of the residue, the specifying agency, at its option, can specify penetration at 100 g; 5 s at 25°C (77°F) of 120 to 250 for Grades MC-30, MC-70, MC-250, MC-800, and MC-3000. However, in no case will both be required.

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TESTS FOR ASPHALT MATERIALS

=== Review Tables 18.1 and 18.2 =====

Consistency Tests

- The consistency of asphalt materials is important in pavement construction because the consistency at a specified temperature will indicate the grade of the material.
- It is important that the temperature at which the consistency is determined be specified, since temperature significantly affects consistency of asphalt materials.

- Liquid state → the viscosity: Saybolt Furol test, or kinematic viscosity test
- Semisolid and solid states ➔ penetration test and the float test
- Ring-and-ball softening point test may be used for blown asphalt

Saybolt Furol Viscosity Test (Figure 18.2)

- Viscometer tube (5 in. long , 1in. in diameter)
- An orifice at bottom of the tube
- The orifice is closed with a stopper, and the tube is filled with a quantity of the material to be tested
- The standard tube then is placed in a larger oil or water bath and fitted with an electric heater and a stirring device
- The material in the tube is brought to the specified temperature by heating the bath.
- Immediately upon reaching the prescribed temperature, the stopper is removed, and time in seconds for exactly ***60 ml*** of the asphalt material to flow through the orifice is recorded.
- Temperatures include: **°25C** (77°F), **50°C** (122°F), and **60°C** (140°F)
- Higher viscosity → Longer time

Figure 18.2 Saybolt Furol Viscometer

Kinematic Viscosity Test (Figure 18.3)

It is defined as the *absolute viscosity* divided by the *density*.

- ❑ The test uses a capillary viscometer tube to measure the time it takes the asphalt sample to flow at a specified temperature between timing marks on the tube.

Three types of viscometer tubes:

- Zeitfuch's cross-arm viscometer: induced by gravitational forces
- Asphalt Institute vacuum viscometer: induced by creating a partial vacuum
- Cannon Manning vacuum viscometer: induced by creating a partial vacuum

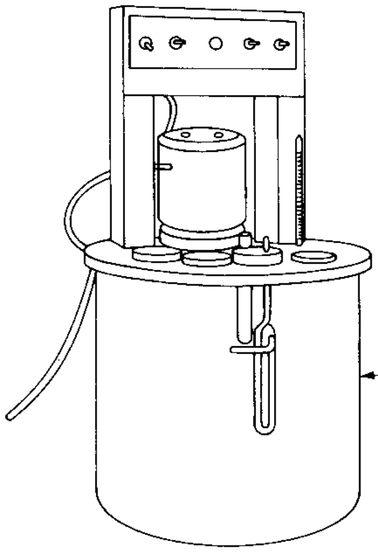


Figure 18.3 Kinematic Viscosity Apparatus

Cross-arm viscometer

- Place the viscometer tube in a thermostatically controlled constant-temperature bath (Figure 18.3)
- Preheated sample and pour it into the large side of the viscometer tube until the filling line level is reached
- Brought the temperature of the bath to 135°C (275°F), and some time is allowed for the viscometer and the asphalt to reach a temperature of 135°C (275°F)
- Induce flow by applying a slight pressure to the large opening or a partial vacuum to the efflux (small) opening of the viscometer tube
- Continuous flow is induced by the action of gravitational forces
- Record the time it takes for the material to flow between two timing marks
- Obtain the kinematic viscosity in *centistokes* by multiplying the time in seconds by a calibration factor for the viscometer used

Asphalt Institute vacuum viscometer / Cannon-Manning vacuum viscometer

- Similar procedure as above
- The product of the time interval and the calibration factor in this test gives the *absolute* viscosity of the material in ***poises***

Rotational Viscosity Test

- Temperatures = 60°C to over 200°C.
- Measure the torque (*Pascal seconds*) required to rotate a cylinder submerged in a heated sample of the asphalt binder at a required speed (*20 rpm*)

Penetration Test (Figure 18.4**)**

- Gives an *empirical measurement* of the consistency
- Place a sample of the asphalt cement in a container which in turn is placed in a temperature-controlled water bath
- Bring the temperature = 25°C (77°F)
- Leave standard needle (loaded to a total weight of 100 g) to penetrate the sample of asphalt for time = 5 seconds
- Record the penetration in *units* of 0.1 mm
- *For example*, if the needle penetrates a distance of exactly 20 mm, the penetration is 200

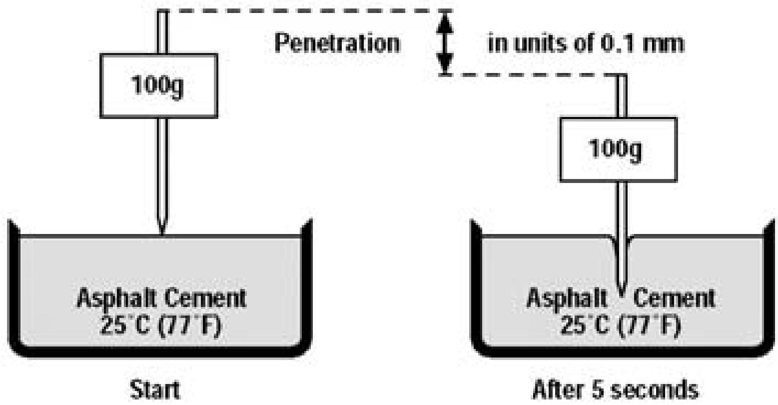


Figure 18.4 Standard Penetration Test and Equipment

Float Test (Figure 18.5)

- ❖ For semisolid asphalt that are *more viscous* than grade 3000 or have *penetration higher* than 300, since these materials cannot be tested conveniently using either the Saybolt Furol viscosity test or the penetration test.

- ❑ The brass collar is filled with a sample
- ❑ It is attached to the bottom of the float and chilled to a temperature of 5°C (41°F) by immersing it in ice water
- ❑ The temperature of the water bath is brought to °50C (122°F), and the collar (still attached to the float) is placed in the water bath which is kept at 50°C (122°F)
- ❑ The head gradually softens the sample until the water eventually forces its way through the plug into the aluminum float
- ❑ The time in *seconds* is a measure of the consistency
  - ☞ The higher the float-test value, the stiffer the material

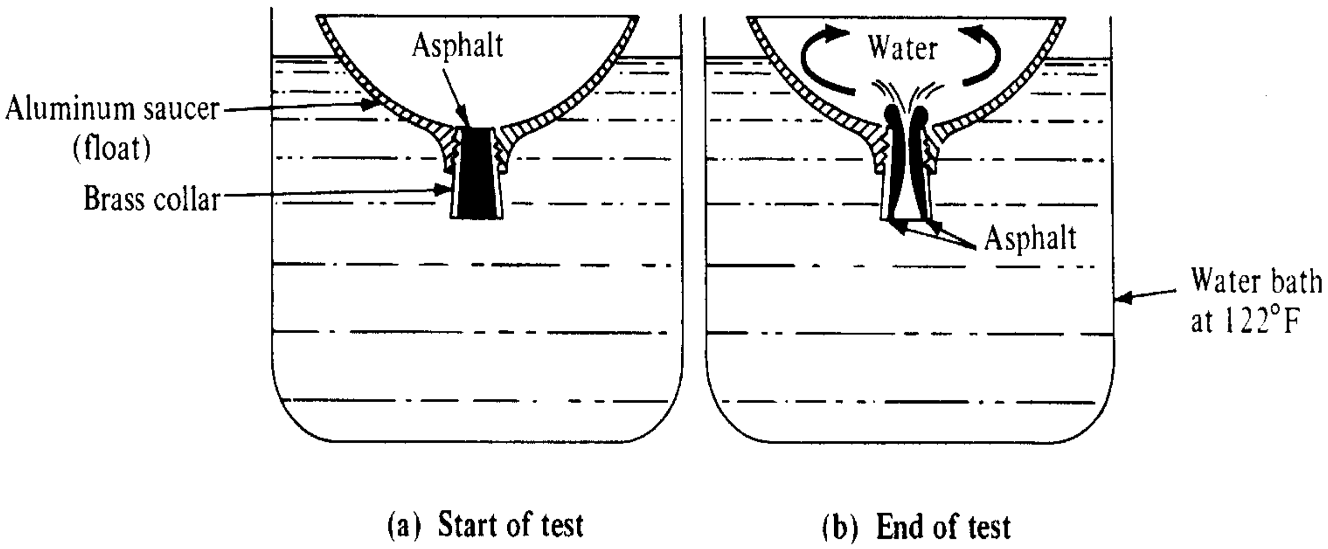


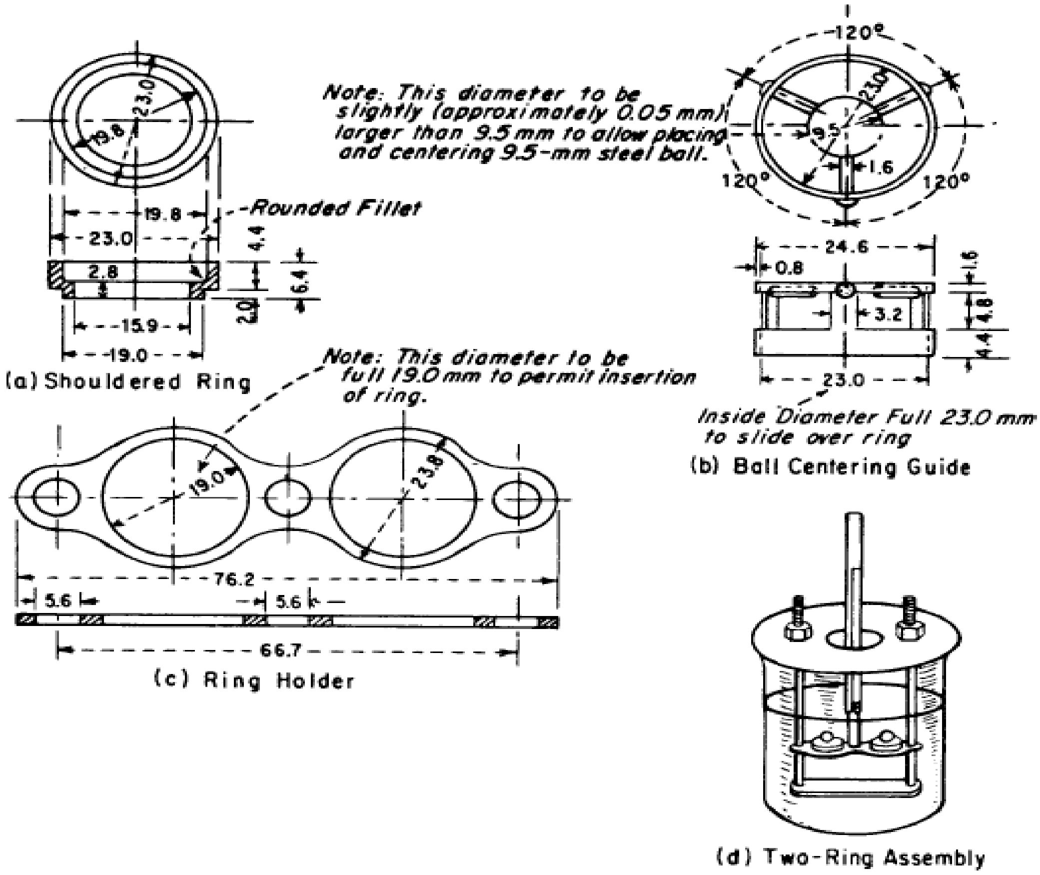
Figure 18.5 Float Test

Ring-and-Ball Softening Point Test (Figure 18.6)

- ✪ The ring-and-ball softening point test is used to measure the susceptibility of blown asphalt to temperature changes by determining the temperature at which the material will be adequately softened to allow a standard ball to sink through it.
- ❖ Two small brass ring of 5⁄8 in. inside diameter and 1⁄4 in. high, a steel ball 3⁄8 in. in diameter, and a water or glycerin bath.

- ❑ Place a sample in the ring which is cooled and immersed in the water or glycerin bath that is maintained at a temperature of 5°C (41°F)
- ❑ Immerse the ring to a depth such that its bottom is exactly 1 in. above the bottom of the bath.
- ❑ Increase temperature of the bath gradually, causing the asphalt to soften and permitting the ball to eventually sink to the bottom of the bath
- ❑ Record the temperature in °C at which the asphalt touches the bottom of the bath as the softening point.

- ✪ The resistance characteristic to temperature shear also can be evaluated using this test.



Durability Tests

- ☞ Asphalt in pavements is subjected to changes in temperature (freezing and thawing) and other weather conditions over a period of time.

These changes cause natural weathering of the material which may lead to → :

- Loss of plasticity
- Cracking
- Abnormal surface abrasion
- Eventual failure of the pavement

- ✪ For evaluation: Use thin- film oven test

Thin-Film Oven Test (TFO)

- ☞ This actually *is not a test* <u>but a procedure</u> that measures the changes that take place in asphalt during the hot-mix process by subjecting the asphalt material to hardening conditions similar to those in a normal hot-mix plant operation.

- ✪ The consistency of the material is determined *before* and *after* the TFO procedure
- ✪ Using either the *penetration* test or a *viscosity* test
- ✪ To estimate the amount of hardening that will take place in the material when used to produce plant hot-mix asphalt

- ❑ Pour 50 cc of material into a cylindrical flat- bottom pan, 5.5 in. (14 cm) inside diameter and 3⁄8 in. (1 cm) high
- ❑ Place the pan containing the sample on a rotating shelf in an oven and rotate for 5 hours while the *temperature* is kept at 163°C (325°F)
- ❑ The amount of penetration after the TFO test is then expressed as a percentage of that before the test to determine percent of penetration retained
- ❖ The minimum allowable percent of penetration retained is usually specified for different grades of asphalt cement

Rate of Curing

- ☞ Tests for curing rates of cutbacks are based on *inherent* factors which can be controlled
- ☞ These tests compare different asphalt materials on the assumption that the *external* factors are held *constant*
- ❖ Volatility and quantity of solvent are used commonly to indicate the rate of curing
- ❖ **Distillation Test** is used for curing rate measuring

Other General Tests

Specific Gravity Test

- ✪ To determine the amount of voids in compacted mixes
- ✪ To correct volumes measured at high temperatures
- ❖ The specific gravity of bituminous materials changes with temperature →

→ The temperature at which the test is con-ducted should be indicated

- ♦ For example, if the test is conducted at 20°C (68°F) and the specific gravity is determined to be 1.41, this should be recorded as 1.41/20°C
- ⚐ Note that both the asphalt material and the water should be at the same temperature
- ⚐ The usual temperature at which the specific gravities of asphalt materials are determined is 25°C (77°F)

- ☞ The test normally is conducted with a pycnometer (Figure 18.8)

- ❑ The dry weight (W1) of the pycnometer and stopper is obtained
- ❑ The pycnometer is filled with distilled water at the prescribed temperature.
- ❑ The weight (W2) of the water and pycnometer together is determined.
- ❑ If the material to be tested can flow easily into the pycnometer, then the pycnometer must be filled completely with the material at the specified temperature after pouring out the water
- ❑ The weight W3 is obtained
- ❑ The specific gravity of the asphalt material is given as

$$G_b = \frac{W_3 - W_1}{W_2 - W_1}$$

- If the asphalt material cannot flow easily, a small sample of the material is heated gradually to facilitate flow and then poured into the pycnometer and left to cool to the specified temperature.
- The weight $W4$ of pycnometer and material then is obtained
- Water is poured into the pycnometer to completely fill the remaining space not occupied by the material.
- The weight $W5$ of the filled pycnometer is obtained.
- The specific gravity then is given as

$$G_b = \frac{W_4 - W_1}{(W_2 - W_1) - (W_5 - W_4)} \tag{18.2}$$

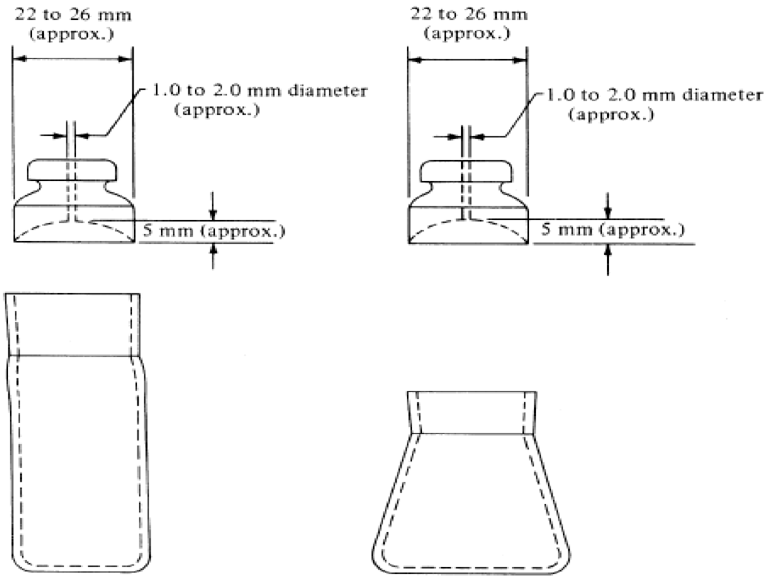


Figure 18.8 Pycnometers for Determining Specific Gravity of Asphalt Materials

Ductility Test

- ✪ Ductility is the distance in centimeters a standard sample of asphalt material will stretch before breaking when tested on standard ductility test equipment at 25°C (77°F).
- ✪ The result of this test indicates the extent to which the material can be deformed without breaking.
- ✪ This is an important characteristic for asphalt materials, although the exact value of ductility is not as important as the existence or nonexistence of the property in the material.
- ✪ The test is used mainly for semi-solid or solid materials.

- ❑ Samples are heated gently to facilitate flow and then poured into a standard mold to form a briquette of at least 1 cm2 in cross section.
- ❑ The material is allowed to cool to 25°C (77°F) in a water bath.
- ❑ The prepared sample is placed in the ductility machine and extended at a specified rate of speed until the thread of material joining the two ends breaks.
- ❑ The distance (in centimeters) moved by the machine is the ductility of the material.

Solubility Test

- ✪ To measure the amount of impurities in the asphalt material
- ✪ Since asphalt is nearly 100 % soluble in certain solvents, the portion of any asphalt material that will be effective in cementing aggregates together can be determined from the solubility test
- ✪ Insoluble materials include free carbon, salts, and other inorganic impurities

- ❑ Dissolve quantity of the material in a solvent (such as trichloroethylene)
- ❑ Filter it through a Gooch crucible
- ❑ Dry and weigh the material retained in the filter is dried
- ❑ The test results are given in terms of the percent of the asphalt material that dissolved in the solvent

Flash-Point Test {open-cup apparatus} (Figure 18.10)

- ✪ The flash point of an asphalt material is the temperature at which its vapors will ignite instantaneously in the presence of an open flame
- ✪ Note that the flash point normally is lower than the temperature at which the material will burn
- ✪ Cleveland open-cup test is more suitable for materials with higher flash points
- ✪ Tagliabue open-cup is more suitable for materials with relatively low flash points, such as cutback asphalts

- ❑ Fill, partly, the cup with the asphalt material
- ❑ Increase, gradually, its temperature at a specified rate
- ❑ Pass small open flame over the surface of the sample at regular intervals as the temperature increases
- ❑ The increase in temperature will cause evaporation of volatile materials from the material until a sufficient quantity of volatile materials is present to cause an instantaneous flash when the open flame is passed over the surface
- ❑ The minimum temperature at which this occurs is the flash point
  - ✪ It can be seen that this temperature gives an indication of the temperature limit at which extreme care should be taken, particularly when heating is done over open flames in open containers

Figure 18.10 Apparatus for Cleveland Open-Cup Test

Loss-on-Heating Test

- ✪ To determine the amount of material that evaporates from a sample of asphalt under a specified temperature and time
- ✪ The result indicates whether asphalt material has been contaminated with lighter materials

- ❑ Pour 50 g of the material into cylindrical tin
- ❑ Leaving it in an oven for 5 hours at a temperature of 163°C (325°F)
- ❑ Determine the weight of the material remaining in the tin
- ❑ The loss in weight is expressed as a percentage of the original weight
- ♦ The penetration of the sample also may be determined before and after the test in order to determine the loss of penetration due to the evaporation of the volatile material.
- ♦ This loss in penetration may be used as an indication of the weathering characteristics of the asphalt.

Water Content Test

- ✪ The presence of large amounts of water in asphalt materials used in pavement construction is *undesirable*
- ✪ To ensure that only a limited quantity of water is present, specifications for these materials usually include the maximum percentage of water by volume that is allowable

- ❑ Mix quantity of the sample with an equal quantity of a suitable distillate in a distillation flask that is connected with a condenser and a trap for collecting the water
- ❑ Heat the sample gradually in the flask causing all of the water to evaporate and be collected. The quantity of water in the sample is then expressed as a percentage of the total sample volume

ASPHALT MIXTURES

✪ Asphalt mixtures are a uniformly mixed combination of asphalt cement, coarse aggregate, fine aggregate, and other materials, depending on the type of asphalt mixture.

Types:

- Hot-mix, hot-laid
- Cold-mix, cold-laid

When used in the construction of highway pavements, it must:

- Resist deformation from imposed traffic loads
- Be skid resistant even when wet,
- Not be affected easily by weathering forces

☞ The degree to which an asphalt mixture achieves these characteristics mainly is dependent on the design of the mix used in producing the material.

Hot-Mix, Hot-Laid Asphalt Mixture

Blending:

- Asphalt cement
- Coarse aggregate
- Fine aggregate
- *Filler* (dust)
- at temperatures ranging from about: 79 to 163 °C (175to 325°F) depending on the type of asphalt cement used

Suitable types of asphalt materials include:

- AC-20, AC-10
- AR-8000 with penetration grades of:
  - ♦ 60 to 70
  - ♦ 85 to 100
  - ♦ 120 to 150
  - ♦ 200 to 300

- ❖ Hot-mix, hot-laid asphalt mixture normally is used for high-type pavement construction

<u>The mixture can be described as:</u>

- Open- graded
- Coarse-graded
- Dense-graded
- Fine-graded

<u>For high-type *surfacing*, maximum sizes of aggregates normally range from:</u>

- ♦ 3⁄8 to 3⁄4 in. for open-graded mixtures
- ♦ 1⁄2 to 3⁄4 in. for coarse-graded mixtures
- ♦ 1⁄2 to 1 in. for dense graded mixtures,
- ♦ 1⁄2 to 3⁄4 in. for fine-graded mixtures

<u>When used as *base*, maximum sizes of aggregates are:</u>

- ♦ 3⁄4 to 1.5 in. for open- and coarse-graded mixtures
- ♦ 1 to 1.5 in. for dense-grades
- ♦ 3⁄4 in. for fine-graded mixtures

- ✪ When designing a hot-mix asphalt mixture, a favorable balance must be found between a highly *stable* product and a *durable* one.
- ✪ Therefore, the overall objective of the mix design is to determine an optimum blend of the different components that will satisfy the requirements of the given specifications.

Aggregate Gradation

- Aggregates usually are categorized as crushed rock, sand, and filler
- The rock material is predominantly coarse aggregate retained in a No. 8 sieve
- Sand is predominantly fine aggregate passing the No. 8 sieve
- Filler is predominantly mineral dust that passes the No. 200 sieve

- It is customary for gradations of the combined aggregate and the individual fractions to be specified.
- Table 18.3 gives suggested grading requirements of aggregate material based on the ASTM Designation ڗ3515
- The first phase in any mix design is the selection and combination of aggregates to obtain a gradation within the limits prescribed.
- This sometimes is referred to as mechanical stabilization.
- The procedure used to select and combine aggregates will be illustrated in the following example.

Table 18.3 Examples of Composition of Asphalt Paving Mixtures

	Dense Mixtures								
	Mix Designation and Nominal Size of Aggregate								
Sieve Size	*2 in. (50 mm)*	*1½ in. (37.5 mm)*	*1 in. (25 mm)*	*¾ in. (19 mm)*	*½ in. (12.5 mm)*	*⅜ in. (9.5 mm)*	*No. 4 (4.75 mm) (Sand Asphalt)*	*No. 8 (2.36 mm)*	*No. 16 (1.18 mm) (Sheet Asphalt)*
	Grading of Total Aggregate (Coarse Plus Fine, Plus Filler if Required) Amounts Finer Than Each Laboratory Sieve (Square Opening), Weight %								
2½ in. (63 mm)	100	...	...	...	...	...	...	...	...
2 in. (50 mm)	90 to 100	100	...	...	...	...	...	...	...
1½ in. (37.5 mm)	...	90 to 100	100	...	...	...	...	...	...
1 in. (25 mm)	60 to 80	...	90 to 100	...	...	...	...	...	...
¾ in. (19 mm)	...	56 to 80	...	90 to 100	100	...	...	...	...
½ in. (12.5 mm)	35 to 65	...	56 to 80	...	90 to 100	100	...	...	...
⅜ in. (9.5 mm)	...	...	...	56 to 80	...	90 to 100	100	...	...
No. 4 (4.75 mm)	17 to 47	23 to 53	29 to 59	35 to 65	44 to 74	55 to 85	80 to 100	...	100
No. 8 (2.36 mm)[A]	10 to 36	15 to 41	19 to 45	23 to 49	28 to 58	32 to 67	65 to 100	...	95 to 100
No. 16 (1.18 mm)	...	...	...	...	...	...	40 to 80	...	85 to 100
No. 30 (600 μm)	...	...	...	...	...	...	25 to 65	...	70 to 95
No. 50 (300 μm)	3 to 15	4 to 16	5 to 17	5 to 19	5 to 21	7 to 23	7 to 40	...	45 to 75
No. 100 (150 μm)	...	...	...	...	...	...	3 to 20	...	20 to 40
No. 200 (75 μm)[B]	0 to 5	0 to 6	1 to 7	2 to 8	2 to 10	2 to 10	2 to 10	...	9 to 20

	Open Mixtures								
	Mix Designation and Nominal Maximum Size of Aggregate								
Sieve Size	*2 in. (50 mm)*	*1½ in. (37.5 mm)*	*1 in. (25 mm)*	*¾ in. (19 mm)*	*½ in. (12.5 mm)*	*⅜ in. (9.5 mm)*	*No. 4 (4.75 mm) (Sand Asphalt)*	*No. 8 (2.36 mm)*	*No. 16 (1.18 mm) (Sheet Asphalt)*
	Base and Binder Courses						*Surface and Leveling Courses*		
2½ in. (63 mm)	100	...	...	...	...	...	...	...	...
2 in. (50 mm)	90 to 100	100	...	...	...	...	...	...	...

1½ in. (37.5 mm)	. . .	90 to 100	100	. . .	. . .	. . .	. . .	. . .	. . .
1 in. (25 mm)	40 to 70	. . .	90 to 100	100	. . .	. . .	. . .	. . .	. . .
¾ in. (19 mm)	. . .	40 to 70	. . .	90 to 100	100	. . .	. . .	. . .	. . .
½ in. (12.5 mm)	18 to 48	. . .	40 to 70	. . .	85 to 100	100	. . .	. . .	. . .
⅜ in. (9.5 mm)	. . .	18 to 48	. . .	40 to 70	60 to 90	85 to 100	. . .	. . .	. . .
No. 4 (4.75 mm)	5 to 25	6 to 29	10 to 34	15 to 39	20 to 50	40 to 70	. . .	100	. . .
No. 8 (2.36 mm)[A]	0 to12	0 to 14	1 to 17	2 to 18	5 to 25	10 to 35	. . .	75 to 100	. . .
No. 16 (1.18 mm)	. . .	. . .	. . .	. . .	3 to 19	5 to 25	. . .	50 to 75	. . .
No. 30 (600 μm)	0 to 8	0 to 8	0 to 10	0 to 10	. . .	. . .	. . .	28 to 53	. . .
No. 50 (300 μm)	. . .	. . .	. . .	. . .	0 to 10	0 to 12	. . .	8 to 30	. . .
No. 100 (150 μm)	. . .	. . .	. . .	. . .	. . .	. . .	. . .	0 to 12	. . .
No. 200 (75 μm)[B]	. . .	. . .	. . .	. . .	. . .	. . .	. . .	0 to 5	. . .
	Bitumen, Weight % of Total Mixture[C]								
	2 to 7	3 to 8	3 to 9	4 to 10	4 to 11	5 to 12	6 to 12	7 to 12	8 to 12
	Suggested Coarse Aggregate Sizes								
	3 and 57	4 and 67 or 4 and 68	5 and 7 or 57	67 or 68 or 6 and 8	7 or 78	8			

[A]In considering the total grading characteristics of a bituminous paving mixture, the amount passing the No. 8 (2.36 mm) sieve is a significant and convenient field control point between fine and coarse aggregate. Gradings approaching the maximum amount permitted to pass the No. 8 sieve will result in pavement surfaces having comparatively fine texture, while coarse gradings approaching the minimum amount passing the No. 8 sieve will result in surfaces with comparatively coarse texture.

[B]The material passing the No. 200 (75 μm) sieve may consist of fine particles of the aggregates or mineral filler, or both, but shall be free of organic matter and clay particles. The blend of aggregates and filler, when tested in accordance with Test Method D4318, shall have a plasticity index of not greater than 4, except that this plasticity requirement shall not apply when the filler material is hydrated lime or hydraulic cement.

[C]The quantity of bitumen is given in terms of weight % of the total mixture. The wide difference in the specific gravity of various aggregates, as well as a considerable difference in absorption, results in a comparatively wide range in the limiting amount of bitumen specified. The amount of bitumen required for a given mixture should be determined by appropriate laboratory testing, on the basis of past experience with similar mixtures, or by a combination of both.

SOURCE: *Annual Book of ASTM Standards, Section 4, Construction,* Vol. 04.03, Road and Paving Materials; Pavement Management Technologies, American Society for Testing and Materials, Philadelphia, PA, 2007.

Q Table below gives the specifications for the aggregates and mix composition for highway pavement asphaltic concrete and shows the results of a sieve analysis of samples from the materials available. Determine the proportions of the separate aggregates that will give a gradation within the specified limits. Use any method.

	Percent Passing by Weight		
Sieve Size	A	B	Required Mix
1 in. (25 mm)	96	100	96-100
3/8 in. (9.5 mm)	74	83	70-80
No. 4 (4.25 mm)	43	53	40-52
No. 10 (2 mm)	30	46	35-40
No. 40 (0.425 mm)	27	23	15-30
No. 200 (0.075 mm)	10	4	4-10

Solution

Trial 1 : Assume the proportion : a = 0.5 and b = 0.5

	Percent Passing by Weight						
Sieve Size	A	B	Required Mix	a×A	b×B	Blended a×A+ b×B	Check
1 in. (25 mm)	96	100	96-100	0.5x96 = 48	0.5x100 = 50	48+50 = 98	ok
3/8 in. (9.5 mm)	74	83	60-85	37	41.5	78.5	ok
No. 4 (4.25 mm)	43	53	37-57	21.5	26.5	48	ok
No. 10 (2 mm)	30	46	30-43	15	23	38	ok
No. 40 (0.425 mm)	27	23	12-39	13.5	11.5	25	ok
No. 200 (0.075 mm)	10	4	4-10	5	2	7	ok

Proportion : a = 0.5 and b = 0.5

Example 18.1 Determining Proportions of Different Aggregates to Obtain a Required Gradation

Table 18.4 gives the specifications for the aggregates and mix composition for highway pavement asphaltic concrete and Table 18.5 shows the results of a sieve analysis of samples from the materials available. We must determine the proportions of the separate aggregates that will give a gradation within the specified limits.

Table 18.4 Required Limits for Mineral Aggregates Gradation and Mix Composition for an Asphalt Mixture for Example 18.1

Passive Sieve Designation	*Retained on Sieve Designation*	*Percent by Weight*
¾ in. (19 mm)	½ in.	0–5
½ in. (12.5 mm)	⅜ in.	8–42
⅜ in. (9.5 mm)	No. 4	8–48
No. 4 (4.75 mm)	No. 10	6–28
Total coarse aggregates	No. 10	48–65
No. 10 (2 mm)	No. 40	5–20
No. 40 (0.425 mm)	No. 80	9–30
No. 80 (0.180 mm)	No. 200	3–20
No. 200 (0.075 mm)	—	2–6
Total fine aggregate and filler	Passing No. 10	35–50
Total mineral aggregate in asphalt concrete		90–95
Asphalt cement in asphalt concrete		5–7
Total mix		100

Table 18.5 Sieve Analysis of Available Materials for Example 18.1

Passing Sieve Designation	*Retained on Sieve Designation*	*Percent by Weight*		
		Coarse Aggregate	*Fine Aggregate*	*Mineral Filler*
¾ in. (19 mm)	½ in.	5	—	—
½ in. (12.5 mm)	⅜ in.	35	—	—
⅜ in. (9.5 mm)	No. 4	38	—	—
No. 4 (4.75 mm)	No. 10	17	8	—
No. 10 (2 mm)	No. 40	5	30	—
No. 40 (0.425 mm)	No. 80	—	35	5
No. 80 (0.180 mm)	No. 200	—	26	35
No. 200 (0.075 mm)	—	—	1	60
Total		100	100	100

Solution: It can be seen that the amount of the different sizes selected should not only give a mix that meets the prescribed limits but also should be such that allowance is made for some variation during actual production of the mix. It also can be seen from Table 18.4 that to obtain the required specified gradation some combination of all three materials is required, since the coarse and fine aggregates do not together meet the requirement of 2 to 6 percent by weight of filler material. Therefore, a trial mix is selected arbitrarily within the prescribed limits. Let this mix be

Coarse aggregates = 55% (48–65% specified)
Fine aggregates = 39% (35–50% specified)
Filler = 6% (5–8% specified)

The selected proportions are then used to determine the combination of the different sizes as shown in Table 18.6. The calculation is based on the fundamental equation for the percentage of material P passing a given sieve for the aggregates 1, 2, 3 and is given as

$$P = aA_1 + bA_2 + cA_3 + \cdots \tag{18.3}$$

where

A_1, A_2, A_3 = the percentages of material passing a given sieve for aggregates 1, 2, 3
a, b, c = the proportions of aggregates 1, 2, 3 used in the combination
$a + b + c + \cdots = 100$

Note that this is true for any number of aggregates combined.

It can be seen that the combination obtained, as shown in the last column of Table 18.6, meets the specified limits as shown in the last column of Table 18.4. The trial combination is therefore acceptable. Note, however, that the first trial may not always meet the specified limits. In such cases, other combinations must be tried until a satisfactory one is obtained.

Table 18.6 Computation of Percentages of Different Aggregate Sizes for Example 18.1

Passing Sieve Size	*Retained on Sieve Size*	*Percent by Weight*			
		Coarse Aggregate	*Fine Aggregate*	*Mineral Filler*	*Total Aggregate*
¾ in.	½ in.	0.55 × 5 = 2.75	—	—	2.75
½ in.	⅜ in.	0.55 × 35 = 19.25	—	—	19.25
⅜ in.	No. 4	0.55 × 38 = 20.90	—	—	20.90
No. 4	No. 10	0.55 × 17 = 9.35	0.39 × 8 = 3.12	—	12.47
No. 10	No. 40	0.55 × 5 = 2.75	0.39 × 30 = 11.70	—	14.45
No. 40	No. 80	—	0.39 × 35 = 13.65	0.06 × 5 = 0.3	13.95
No. 80	No. 200	—	0.39 × 26 = 10.14	0.06 × 35 = 2.10	12.24
No. 200	—	—	0.39 × 1 = 0.39	0.06 × 60 = 3.60	3.99
Total		55.0	39.0	6.0	100.00

Several graphical methods have been developed for obtaining a suitable mixture of different aggregates to obtain a desired gradation. These methods tend to be rather complicated when the number of batches of aggregates is high. They generally can be of advantage over the trial-and-error method described here when the number of

Asphalt Content

- ✪ Determine the optimum percentage of asphalt that should be used in the asphalt mixture.
- ✪ The gradation of the aggregates determined earlier and the optimum amount of asphalt cement determined combine to give the proportions of the different materials to be used in producing the hot-mix, hot-laid mixture for the project under consideration.
- ✪ These determined proportions usually are referred to as the job-mix formula.

Two commonly used methods:

- Marshall method (more widely used)
- Hveem method

Mar shall Method Procedure

- ❑ Test specimens of 4 in. diameter and 21/2 in. height are used in this method.
- ❑ They are prepared by a specified procedure of heating, mixing, and compacting the mixture of asphalt and aggregates which is then subjected to a stability- flow test and a density-voids analysis.
- ❑ The stability is defined as the maximum load resistance N in pounds that the specimen will achieve at (60°C) 140°F under specified conditions.
- ❑ The flow is the total movement of the specimen in units of (0.25 mm) 0.01 in. during the stability test as the load is increased from zero to the maximum.
- ❑ Test specimens for the Marshall method are prepared for a range of asphalt contents within the prescribed limits.
- ❑ Usually the asphalt content is measured by 0.5 percent increments from the minimum prescribed, ensuring that at least two are below the optimum and two above the optimum so that the curves obtained from the result will indicate a well-defined optimum.
- ❑ For example, for a specified amount of 5 to 7percent, mixtures of 5, 5.5, 6, 6.5, and 7 are prepared.
- ❑ At least three specimens are provided for each asphalt content to facilitate the provision of adequate data.

- ❑ For this example of *five* different asphalt contents, therefore, a total minimum of *15* specimens are required.
- ❑ The amount of aggregates required for each specimen is about 1.2 kg.
- ❑ A quantity of the aggregates having the designed gradation is dried at a temperature between 105°C (221°F) and 110°C (230°F) until a constant weight is obtained.
- ❑ The mixing temperature for this procedure is set as the temperature that will produce a kinematic viscosity of 170 ± 20 centistokes or a Saybolt Furol viscosity of 85 ± 10 seconds.
- ❑ The compacting temperature is that which will produce a kinematic viscosity of 280 ± 30 centistokes or a Saybolt Furol viscosity of 160 ± 15 seconds.
- ❑ These temperatures are determined and recorded.
- ❑ The specimens containing the appropriate amounts of aggregates and asphalt are prepared by thoroughly mixing and compacting each mixture.
- ❑ The compaction effort used is <u>35, 50, or 75</u> blows of the hammer falling a distance of 18 in., depending on the design traffic category.
- ❑ After the application on one face, the sample mold is reversed and the same number of blows is applied to the other face of the sample.
- ❑ The specimen then is cooled and tested for stability and flow *after* determining its bulk *density*.
- ❑ The bulk density of the sample usually is determined by weighing the sample in air and in water.
- ❑ It may be necessary to *coat* samples made from <u>open-graded</u> mixtures with *paraffin* <u>before</u> determining the density.
- ❑ The bulk specific gravity G_{mb} of the sample—that is, the compacted mixture—is given as

$$G_{mb} = \frac{W_a}{W_a - W_w} \tag{18.4}$$

where

W_a = weight of sample in air (g)
W_w = weight of sample in water (g)

Stability Test (Figure 18.12)

- The specimen is immersed in a bath of water at a temperature of 60 ± 1°C (140 ± 1.8°F) for a period of 30 to 40 minutes.
- It is then placed in the Marshall stability testing machine and loaded at a constant rate of deformation of 2 in. (5 mm) per minute until failure occurs.
- The total load N in pounds that causes failure of the specimen at 60°C (140°F) is noted as the Marshall Stability value of the specimen.
- The total amount of deformation in units of 0.01 in. that occurs up to the point the load starts decreasing is recorded as the flow value.
- The total time between removing the specimen from the bath and completion of the test should not exceed 30 seconds.

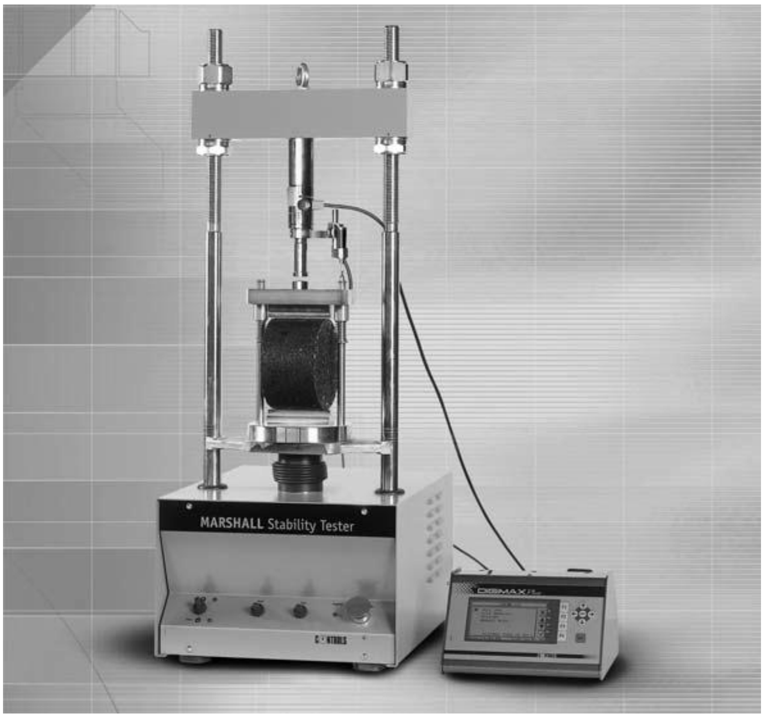


Figure 18.12 Marshall Stability Equipment

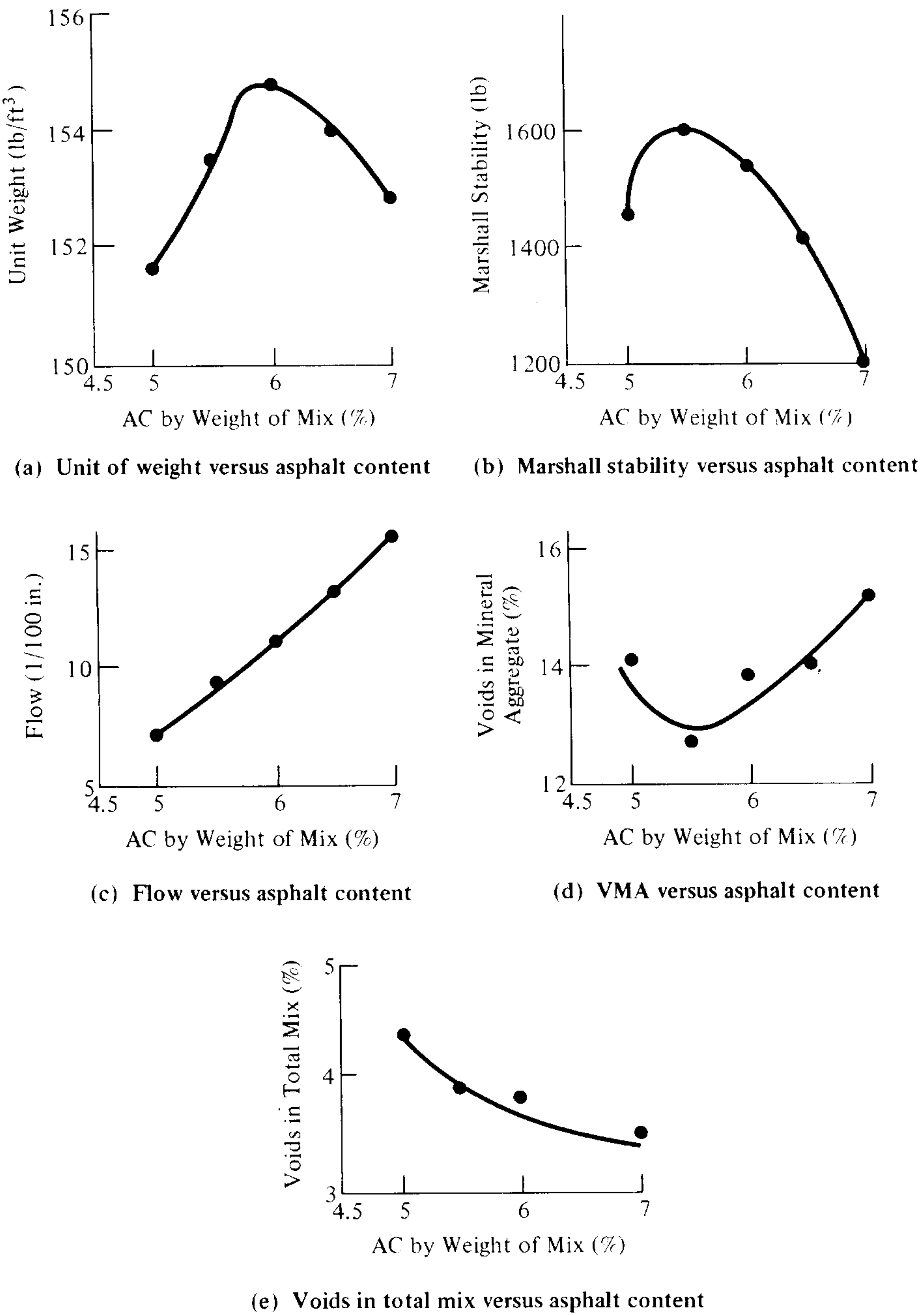
Analysis of Results from Marshall Test

Steps:

- Determine average bulk specific gravity for all test specimens having the same asphalt content
- Obtain the average unit weight of each mixture by multiplying its average specific gravity by the density of water γ_w
- Determine a smooth curve that represents the best fit of plots of unit weight versus percentage of asphalt (Figure 18.13(a)). This curve is used to obtain the bulk specific gravity values that are used in further computations as in Example 18.2
- For different asphalt contents, Compute:

❖ the bulk specific gravity of the aggregate mixture
❖ the apparent specific gravity of the aggregate mixture
❖ the effective specific gravity of the aggregate mixture,
❖ the maximum specific gravity of the paving mixtures
❖ the percent air voids
❖ the percent voids in the mineral aggregate
❖ the absorbed asphalt in pounds of the dry aggregate

These different measures of the specific gravity of the aggregates take into consideration the variation with which mineral aggregates can absorb water and asphalt (see Figure 18.14).



(a) **Unit of weight versus asphalt content**

(b) **Marshall stability versus asphalt content**

(c) **Flow versus asphalt content**

(d) **VMA versus asphalt content**

(e) **Voids in total mix versus asphalt content**

Figure 18.13 Marshall Test Property Curves for Example 18.2

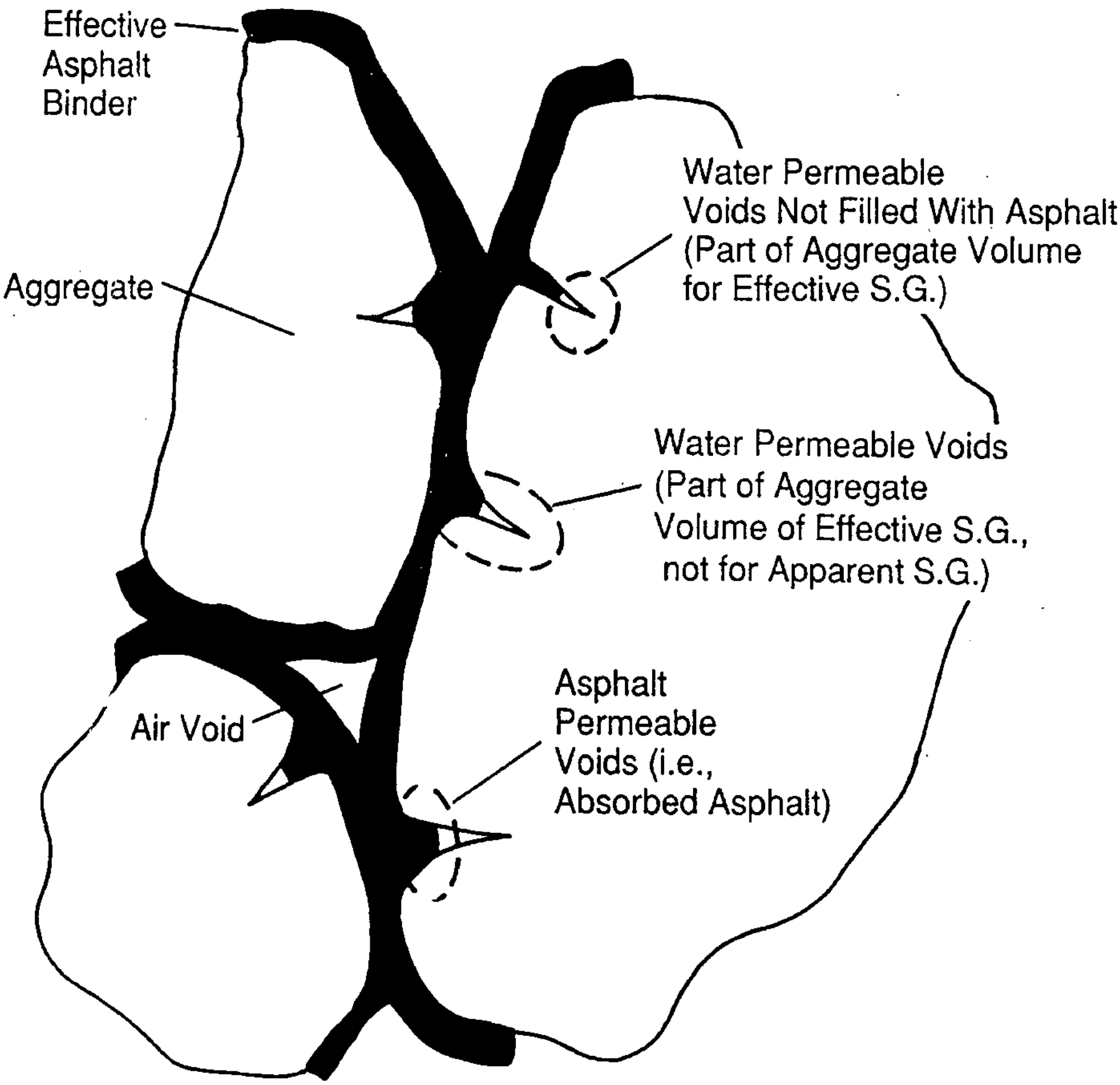


Figure 18.14 Bulk, Effective, and Apparent Specific Gravities; Air Voids; and Effective Asphalt Content in Compacted Asphalt Paving Mixture

Bulk Specific Gravity of Aggregate

- ✪ It is the weight in air of a unit volume (including all normal voids) of a permeable material at a selected temperature, divided by the weight in air of the same volume of gas-free distilled water at the same selected temperature.
- ✪ Since the aggregate mixture consists of different fractions of coarse aggregate, fine aggregate, and mineral fillers with different specific gravities

$$G_{sb} = \frac{P_{ca} + P_{fa} + P_{mf}}{\frac{P_{ca}}{G_{bca}} + \frac{P_{fa}}{G_{bfa}} + \frac{P_{mf}}{G_{bmf}}} \tag{18.5}$$

where

G_{sb} = bulk specific gravity of aggregates in the paving mixture

P_{ca}, P_{fa}, P_{mf} = percent by weight of coarse aggregate, fine aggregate, and mineral filler, respectively, in the paving mixture. (Note that P_{ca}, P_{fa}, and P_{mf} could be found either as a percentage of the paving mixture or as a percentage of only the total aggregates. The same results will be obtained for G_{sb})

$G_{bca}, G_{bfa}, G_{bmf}$ = bulk specific gravities of coarse aggregate, fine aggregate, and mineral filler, respectively

- ☒ It is not easy to accurately determine the bulk specific gravity of the mineral filler. The apparent specific gravity may therefore be used with very little error

Apparent Specific Gravity of Aggregates

It is the ratio of the weight in air of an impermeable material to the weight of an equal volume of distilled water at a specified temperature

$$G_{asb} = \frac{P_{ca} + P_{fa} + P_{mf}}{\dfrac{P_{ca}}{G_{aca}} + \dfrac{P_{fa}}{G_{afa}} + \dfrac{P_{mf}}{G_{amf}}} \tag{18.6}$$

where

G_{asb} = *apparent* specific gravity of the aggregate mixture
P_{ca}, P_{fa}, P_{mf} = percent by weight of coarse aggregate, fine aggregate, and mineral filler, respectively, in the mixture
G_{aca}, G_{afa}, G_{amf} = apparent specific gravities of coarse aggregate, fine aggregate, and mineral filler, respectively

Effective Specific Gravity of Aggregate

It is based on the maximum specific gravity of the paving mixture. It is therefore the specific gravity of the aggregates when all void spaces in the aggregate particles are included, with the exception of those that are filled with asphalt (see Figure 18.14.)

It is given as:

$$G_{se} = \frac{100 - P_b}{(100/G_{mm}) - (P_b/G_b)} \tag{18.7}$$

where

G_{se} = effective specific gravity of the aggregates
G_{mm} = maximum specific gravity of paving mixture (no air voids)
P_b = asphalt percent by total weight of paving mixture (thus $100 - P_b$ is the percent by weight of the base mixture that is not asphalt)
G_b = specific gravity of the asphalt

Maximum Specific Gravity of the Paving Mixture (G_{mm})

- ✪ G_{mm} assumes that there are no air voids in the asphalt concrete.
- ✪ Although the G_{mm} can be determined in the laboratory by conducting the standard test (ASTM Designation D2041), the best accuracy is attained at mixtures near the optimum asphalt content.
- ❑ Since it is necessary to determine the G_{mm} for all samples, some of which contain much lower or much higher quantities than the optimum asphalt content, the following procedure can be used to determine the G_{mm} for each sample.
- ❑ The ASTM Designation D2041 test is conducted on all specimens containing selected asphalt cement content and the mean of these is determined.
- ❑ This value is then used to determine G_{se} using Eq. 18.7.
- ❑ G_{se} can be considered constant, since varying the asphalt content in the paving mixture does not significantly vary the asphalt absorption.
- ❑ G_{se} obtained then is used to determine the G_{mm} with different asphalt cement contents using Eq. 18.8.

$$G_{mm} = \frac{100}{(P_s/G_{se}) + (P_b/G_b)} \tag{18.8}$$

where

G_{mm} = maximum specific gravity of paving mixture (no air voids)
P_s = percent by weight of aggregates in paving mixture
P_b = percent by weight of asphalt in paving mixture
G_{se} = effective specific gravity of the aggregates (assumed to be constant for different asphalt cement contents)
G_b = specific gravity of asphalt

Once these different specific gravities have been determined, the asphalt absorption, the effective asphalt content, the percent voids in mineral aggregates (*VMA*), and the percent air voids in the compacted mixture all can be determined.

Asphalt absorption is the percent by weight of the asphalt that is absorbed by the aggregates based on the total weight of the aggregates. This is given as

$$P_{ba} = 100\frac{G_{se} - G_{sb}}{G_{sb}G_{se}} G_b \tag{18.9}$$

where

P_{ba} = amount of asphalt absorbed as a percentage of the total weight of aggregates
G_{se} = effective specific gravity of the aggregates
G_{sb} = bulk specific gravity of the aggregates
G_b = specific gravity of asphalt

Effective Asphalt Content

It is the difference between the total amount of asphalt in the mixture and that absorbed into the aggregate particles. It is therefore that which coats the outside of the aggregate particles and influences the pavement performance. It is given as:

$$P_{be} = P_b - \frac{P_{ba}}{100} P_s \qquad (18.10)$$

where

P_{be} = effective asphalt content in paving mixture (percent by weight)
P_b = percent by weight of asphalt in paving mixture
P_s = aggregate percent by weight of paving mixture
P_{ba} = amount of asphalt absorbed as a percentage of the total weight of aggregates

Percent Voids in Compacted Mineral Aggregates (VMA)

VMA is the percentage of void spaces between the granular particles in the compacted paving mixture, including the air voids and the volume occupied by the effective asphalt content.

It usually is calculated as a percentage of the bulk volume of the compacted mixture based on the bulk specific gravity of the aggregates. It is given as

$$VMA = 100 - \frac{G_{mb} P_s}{G_{sb}} \qquad (18.11)$$

where

VMA = percent voids in compacted mineral aggregates (percent of bulk volume)
G_{mb} = bulk specific gravity of compacted mixture
G_{sb} = bulk specific gravity of aggregate
P_s = aggregate percent by weight of total paving mixture

Percent Air Voids in Compacted Mixture (P_a)

This is the ratio (expressed as a percentage) between the volume of the small air voids between the coated particles and the total volume of the mixture. It can be obtained from:

$$P_a = 100 \frac{G_{mm} - G_{mb}}{G_{mm}} \quad (18.12)$$

where

P_a = percent air voids in compacted paving mixture
G_{mm} = maximum specific gravity of the compacted paving mixture
G_{mb} = bulk specific gravity of the compacted paving mixture

<u>To select the optimum Asphalt Content:</u>

- ❑ Draw the 5 graphs (Fig. 18.13)
- ❑ Use these graphs to select the asphalt contents for:
- ☞ Maximum stability
- ☞ Maximum unit weight
- ☞ Percent voids in the total mix within the limits specified (usually the median of the limits)
- ☞ The average of the asphalt contents is the optimum asphalt content
- ❑ The stability and flow for this optimum content then can be obtained from the appropriate graphs to determine whether the required criteria are met
- ✪ AASHTO suggested criteria for these test limits are given in Table 18.7
- ✪ It should be noted that all criteria should be satisfied and not just the criterion for stability
- ⚐ This analysis is illustrated further in Example 18.2

Table 18.7 Suggested Test Limits

(a) Maximum and Minimum Values

Marshall Method Mix Criteria	*Light Traffic ESAL < 10^4 (see Chapter 19)*	*Medium Traffic 10^4 < ESAL < 10^6 (see Chapter 19)*	*Heavy Traffic ESAL > 10^6 (see Chapter 19)*
Compaction (No. of blows each end of Specimen)	35	50	75
Stability N (lb)	3336 (750)	5338 (1200)	8006 (1800)
Flow, 0.25 mm (0.1 in.)	8 to 18	8 to 16	8 to 14
Air Voids (%)	3 to 5	3 to 5	3 to 5

(b) Mineral Percent Voids in Mineral Aggregates

Standard Sieve Designation	*Percent*
No. 16	23.5
No. 4	21
No. 8	18
⅜ in.	16
½ in.	15
¾ in.	14
1 in.	13
1½ in.	12
2 in.	11.5
2½ in.	11

Evaluation and Adjustment of Mix Design

<u>The mixture should have:</u>

- ❑ Adequate amount of asphalt to ensure a durable pavement
- ❑ Adequate mix stability to prevent unacceptable distortion and displacement when traffic load is applied
- ❑ Adequate voids in the total compacted mixture to permit a small amount of compaction when traffic load is applied without:
- ☞ loss of stability
- ☞ blushing
- ☞ bleeding
- ❖ But at the same time insufficient voids to prevent harmful penetration of air and moisture into the compacted mixture
- ❑ Adequate workability to facilitate placement of the mix without segregation

- ☒ When the mix design for the optimum asphalt content *does not satisfy* all of the requirements given in Table 18.7, it is necessary to *adjust* the original blend of aggregates.

<u>Trial mixes can be adjusted by using the following general guidelines</u>

Low Voids and Low Stability

<u>In this situation:</u>

- ❑ Increase the voids in the mineral aggregates by adding more coarse aggregates
- ❑ Reduce the asphalt content (if it is high)

- × Be careful; this can lead to:
  - ☒ decrease in durability
  - ☒ increase in permeability

Low Voids and Satisfactory Stability

- ☒ Additional compaction with time ➔ instability or flushing

- ❑ Add more aggregates

High Voids and Satisfactory Stability

  - ☒ High permeability
  - ☒ Allow water and air to circulate through the pavement
  - ☒ Hardening of the asphalt.
- ❑ Increase filler

Satisfactory Voids and Low Stability

  - ☒ This condition suggests low quality aggregates
- ❑ Improved the quality

High Voids and Low Stability

- ❑ Adjust the voids
- ❑ If fail, improve aggregate quality

Example 18.2 Designing an Asphalt Concrete Mixture

In designing an asphalt concrete mixture for a highway pavement to support medium traffic, data in Table 18.8 showing the aggregate characteristics and Table 18.9 (page 984) showing data obtained using the Marshall method were used. Determine the optimum asphalt content for this mix for the specified limits given in Table 18.7.

Solution: The bulk specific gravity of the mix for each asphalt cement content is determined by calculating the average value for the specimens with the same asphalt cement content using Eq. 18.4.

Table 18.9 Marshall Test Data for Example 18.2

Asphalt % by Weight of Total Mix	*Weight of Specimen (g)*						*Stability (lb)*			*Flow (0.01 in.)*			*Maximum Specific Gravity of Paving Mixture*
	in Air			*in Water*									
	1	*2*	*3*	*1*	*2*	*3*	*1*	*2*	*3*	*1*	*2*	*3*	
5.0	1325.6	1325.4	1325.0	780.1	780.3	779.8	1460	1450	1465	7	7.5	7.	2.54
5.5	1331.3	1330.9	1331.8	789.6	789.3	790.0	1600	1610	1595	10	9.	9.5	2.56
6.0	1338.2	1338.5	1338.1	798.6	798.3	797.3	1560	1540	1550	11	11.5	11.	2.58
6.5	1343.8	1344.0	1343.9	799.8	797.3	799.9	1400	1420	1415	13	13.	13.5	2.56
7.0	1349.0	1349.3	1349.8	798.4	799.0	800.1	1200	1190	1210	16	15.	16.	2.54

Table 18.8 Aggregate Characteristics for Example 18.2

Aggregate Type	*Percent by Weight of Total Paving Mixture*	*Bulk Specific Gravity*
Coarse	52.3	2.65
Fine	39.6	2.75
Filler	8.1	2.70

Note: The nominal maximum particle size in the aggregate mixture is 1 in.

For 5% asphalt content, the average bulk specific gravity is given as

$$G_{mb} = \frac{1}{3}\left(\frac{1325.6}{1325.6 - 780.1} + \frac{1325.4}{1325.4 - 780.3} + \frac{1325.0}{1325.0 - 779.8}\right)$$
$$= \frac{1}{3}(2.43 + 2.43 + 2.43)$$
$$= 2.43$$

Therefore, the bulk density is $2.43 \times 62.4 = 151.6\ \text{lb/ft}^3$.

For 5.5% asphalt content,

$$G_{mb} = \frac{1}{3}\left(\frac{1331.3}{1331.3 - 789.6} + \frac{1330.9}{1330.9 - 789.3} + \frac{1325.0}{1325.0 - 779.8}\right)$$
$$= \frac{1}{3}(2.46 + 2.46 + 2.46)$$
$$= 2.46$$

Therefore, the bulk density is $153.5\ \text{lb/ft}^3$.

For 6.0% asphalt content,

$$G_{mb} = \frac{1}{3}\left(\frac{1338.2}{1338.2 - 798.6} + \frac{1338.5}{1338.5 - 798.3} + \frac{1338.1}{1338.1 - 797.3}\right)$$
$$= \frac{1}{3}(2.48 + 2.48 + 2.47)$$
$$\cong 2.48$$

Therefore, the bulk density is $154.8\ \text{lb/ft}^3$.

For 6.5% asphalt content,

$$G_{mb} = \frac{1}{3}\left(\frac{1343.8}{1343.8 - 799.8} + \frac{1344.0}{1344.0 - 797.3} + \frac{1343.9}{1343.9 - 799.9}\right)$$
$$= \frac{1}{3}(2.47 + 2.46 + 2.47)$$
$$\cong 2.47$$

Therefore, the bulk density is $154.1\ \text{lb/ft}^3$.

For 7.0% asphalt content,

$$\begin{aligned} G_{\mathrm{mb}} &= \frac{1}{3}\left(\frac{1349.0}{1349.0 - 798.4} + \frac{1349.3}{1349.3 - 799} + \frac{1349.8}{1349.8 - 800.1}\right) \\ &= \frac{1}{3}(2.45 + 2.45 + 2.46) \\ &\cong 2.45 \end{aligned}$$

Therefore, the bulk density is 152.9 $\mathrm{lb/ft^3}$.

Average bulk density then is plotted against asphalt content as shown in Figure 18.13(a). Similarly, the average stability and flow for each asphalt cement content are as follows.

%	*Stability*	*Flow*
5.0	1458	7.2
5.5	1602	9.5
6.0	1550	11.2
6.5	1412	13.2
7.0	1200	15.7

These values are plotted in Figures 18.13(b) and (c).

We now have to compute percent voids in the mineral aggregate *VMA* and the percent voids in the compacted mixture for each asphalt cement mixture. First, use Eq. 18.11.

$$VMA = 100 - \frac{G_{\mathrm{mb}} P_{\mathrm{s}}}{G_{\mathrm{sb}}}$$

For 5% asphalt content,

$$\begin{aligned} G_{\mathrm{mb}} &= 2.43 \\ P_{\mathrm{ta}} &= 95.0 \text{ (total aggregate percent)} \end{aligned}$$

Use Eq. 18.5 to calculate G_{sb}.

$$G_{\mathrm{sb}} = \frac{P_{\mathrm{ca}} + P_{\mathrm{fa}} + P_{\mathrm{mf}}}{(P_{\mathrm{ca}}/G_{\mathrm{bca}}) + (P_{\mathrm{fa}}/G_{\mathrm{bfa}}) + (P_{\mathrm{mf}}/G_{\mathrm{bmf}})}$$

Determine P_{ca}, P_{fa}, and P_{mf} in terms of total aggregates.

$$\begin{aligned} P_{\mathrm{ca}} &= 0.523 \times 95.0 = 49.7 \\ P_{\mathrm{fa}} &= 0.396 \times 95.0 = 37.6 \\ P_{\mathrm{mf}} &= 0.081 \times 95.0 = 7.7 \end{aligned}$$

Therefore,

$$G_{sb} = \frac{49.7 + 37.6 + 7.7}{(49.7/2.65) + (37.6/2.75) + (7.7/2.70)} = 2.69$$

$$P_{ta} = (100 - 5) = 95$$

and

$$VMA = 100 - \frac{2.43 \times 95}{2.69} = 14.18$$

For 5.5% asphalt content,

$$P_{ca} = 0.523 \times 94.5 = 49.4$$
$$P_{fa} = 0.396 \times 94.5 = 37.4$$
$$P_{mf} = 0.081 \times 94.5 = 7.7$$

therefore,

$$G_{sb} = \frac{49.4 + 37.4 + 7.7}{(49.4/2.65) + (37.4/2.75) + (7.7/2.70)} = 2.69$$

and

$$VMA = 100 - \frac{2.46 \times 94.5}{2.69} = 13.58$$

For 6% asphalt cement,

$$P_{ca} = 0.523 \times 94 = 49.2$$
$$P_{fa} = 0.396 \times 94 = 37.2$$
$$P_{mf} = 0.081 \times 94 = 7.6$$

therefore,

$$G_{sb} = \frac{49.2 + 37.2 + 7.7}{(49.2/2.65) + (37.2/2.75) + (7.6/2.70)} = 2.69$$

and

$$VMA = 100 - \frac{2.48 \times 94}{2.69} = 13.34$$

For 6.5% asphalt content,

$$P_{ca} = 0.523 \times 93.5 = 48.9$$
$$P_{fa} = 0.396 \times 93.5 = 37.0$$
$$P_{mf} = 0.081 \times 93.5 = 7.6$$

therefore,

$$G_{sb} = \frac{48.9 + 37.0 + 7.6}{\frac{48.9}{2.65} + \frac{37.0}{2.75} + \frac{7.6}{2.7}} = 2.69$$

and

$$VMA = 100 - \frac{2.47 \times 93.5}{2.69} = 14.15$$

For 7.0% asphalt content,

$$P_{ca} = 0.523 \times 93.0 = 48.6$$
$$P_{fa} = 0.396 \times 93.0 = 36.8$$
$$P_{mf} = 0.081 \times 93.0 = 7.5$$

therefore,

$$G_{sb} = \frac{48.6 + 36.8 + 7.5}{\dfrac{48.6}{2.65} + \dfrac{36.8}{2.75} + \dfrac{7.5}{2.7}} = 2.69$$

and

$$VMA = 100 - \frac{2.45 \times 93}{2.69} = 15.30$$

A plot of *VMA* versus asphalt content based on these calculations is shown in Figure 18.13(d).

We now have to determine the percentage of air voids in each of the paving mixtures using Eq. 18.12.

$$P_a = 100 \frac{G_{mm} - G_{mb}}{G_{mm}}$$

For 5% asphalt content,

$$P_a = 100 \frac{2.54 - 2.43}{2.54} = 4.33$$

For 5.5% asphalt content,

$$P_a = 100 \frac{2.56 - 2.46}{2.56} = 3.91$$

For 6.0% asphalt content,

$$P_a = 100 \frac{2.58 - 2.48}{2.58} = 3.88$$

For 6.5% asphalt content,

$$P_a = 100 \frac{2.56 - 2.47}{2.57} = 3.50$$

For 7.0% asphalt content,

$$P_a = 100 \frac{2.54 - 2.45}{2.54} = 3.54$$

A plot of P_a versus asphalt content based on these calculations is shown in Figure 18.13(e).

The asphalt content that meets the design requirements for unit weight, stability, and percent air voids then is selected from the appropriate plot in Figure 18.14. The asphalt content having the maximum value of unit weight and stability is selected from each of the respective plots.

1. Maximum unit weight = 6.0% [Figure 18.13(a)]
2. Maximum stability = 5.5% [Figure 18.13(b)]
3. Percent air voids in compacted mixture using mean of limits [that is, (3 + 5)/2 = 4] = 5.4% [Figure 18.13(e)]. (Note the limits of 3 and 5% given in Table 18.7.)

The optimum asphalt content is determined as the average.

Therefore, the optimum asphalt cement content is

$$\frac{6.0 + 5.5 + 5.4}{3} = 5.6\%$$

The properties of the paving mixture containing the optimum asphalt content now can be determined from Figure 18.13 and compared with the suggested criteria given in Table 18.8. The values for this mixture are

Unit weight = 153.8 lb/ft^3

Stability = 1600 lb

Flow = 9.5 units of 0.01 in.

Percent void total mix = 3.9

Percent voids in mineral aggregates = 13

This mixture meets all the requirements given in Table 18.7 for stability, flow, and percent voids in total mix.

Example 18.3 Computing the Percent of Asphalt Absorbed

Using the information given in Example 18.2, determine the asphalt absorbed for the optimum mix. The maximum specific gravity for this mixture is 2.57 and the specific gravity of the asphalt cement is 1.02. From Eq. 18.9, the absorbed asphalt is given as

$$P_{ba} = 100 \frac{G_{se} - G_{sb}}{G_{sb} \times G_{se}} G_b$$

Solution: It is first necessary to determine the effective specific gravity of aggregates using Eq. 18.7.

$$G_{se} = \frac{100 - P_b}{(100/G_{mm}) - (P_b/G_b)}$$

$$G_{se} = \frac{100 - 5.6}{(100/2.57) - (5.6/1.02)} = 2.82$$

The bulk specific gravity of the aggregates in this mixture is

$$G_{sb} = \frac{0.523 \times 94.4 + 0.396 \times 94.4 + 0.081 \times 94.4}{\dfrac{0.523 \times 94.4}{2.65} + \dfrac{0.396 \times 94.4}{2.75} + \dfrac{0.081 \times 94.4}{2.7}} = 2.69$$

The asphalt absorbed is obtained from Eq. 18.9.

$$P_{ba} = 100\,\frac{2.82 - 2.69}{2.60 \times 2.82}\,1.02 \cong 1.75\%$$

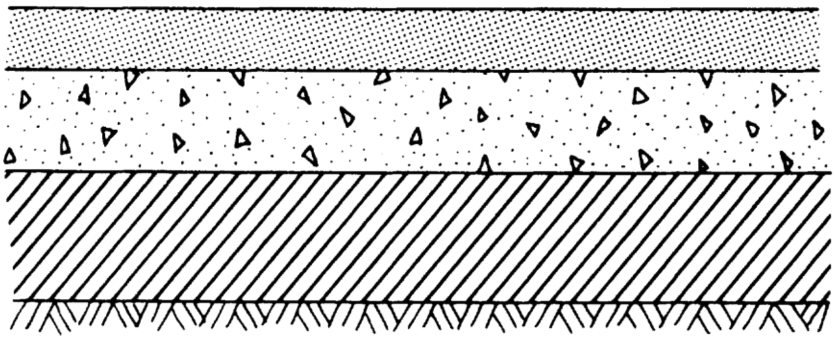
Abstract of Calculations for Example 18.2

AC %	Gmb	Stability (Ib)	Flow (0.25 mm)	Gmm	Ps %	Gsb	VMA %	Pa %
5	2.43	1458	7.2	2.54	95	2.69	14.18	4.33
5.5	2.46	1602	9.5	2.56	94.5	2.69	13.58	3.91
6	2.48	1550	11.2	2.58	94	2.69	13.34	3.88
6.5	2.47	1412	13.2	2.56	93.5	2.69	14.15	3.5
7	2.45	1200	15.7	2.54	93	2.69	15.3	3.54

Gmb = Wa/(Wa-Ww) → Average Stability → Average   Flow → Average

$$G_{sb} = \frac{P_{ca} + P_{fa} + P_{mf}}{\frac{P_{ca}}{G_{bca}} + \frac{P_{fa}}{G_{bfa}} + \frac{P_{mf}}{G_{bmf}}} \qquad P_a = 100\,\frac{G_{mm} - G_{mb}}{G_{mm}} \qquad VMA = 100 - \frac{G_{mb} P_s}{G_{sb}}$$

DESIGN OF FLEXIBLE PAVEMENTS



Subgrade (Prepared Road Bed)

- Natural material
- Foundation of the pavement structure
- It may consist of a layer of selected borrow materials, well compacted
- It may be necessary to treat the subgrade material to achieve certain strength properties required for the type of pavement being constructed

Subbase Course

- Immediately above the subgrade
- Consists of material of a <u>superior quality</u>
- Requirements: gradation, plastic characteristics, and strength

Base Course

- Immediately above the subbase
- It is placed immediately above the subgrade *if* a *subbase course is not used*
- Granular materials: crushed stone, crushed or uncrushed slag, crushed or uncrushed gravel, and sand
- Material stabilized with Portland cement, asphalt, or lime
- Requirements: gradation, plastic characteristics, and strength

Surface Course

- Upper course of the road pavement
- Immediately above the base course
- Mixture of mineral aggregates and asphalt

<u>It should be capable of withstanding:</u>

- high tire pressures
- resisting abrasive forces due to traffic
- providing a skid resistant
- driving surface
- preventing the penetration of surface water into the underlying layers

🏳 Thickness = 7.5 to more than 15 cm depending on the expected traffic on the pavement

🏳 Surface may consists of 2 layers or more (~~Base~~, Binder, Wearing)

AASHTO Design Method

Design Considerations:

- ♦ Pavement Performance
- ☞ Carry the traffic load
- ☞ Resist: cracking, faulting, raveling, and so forth …

✪ Present ***<u>serviceability</u>*** index (PSI) = 0 – 5

Initial serviceability index (***pi***): *immediately* after *construction*

Terminal serviceability index (***pt***): which is the minimum acceptable value *before resurfacing* or *reconstruction*

pi =4.2

pt = 2.5 or 3.0 for major highways

= 2.0 for highways with a lower classification

= 1.5 economic constraints restrict capital expenditures for construction

Traffic Load

Traffic load is determined in terms of the number of ***<u>repetitions</u>*** of an 18,000-lb (80 kilonewtons (kN)) single –axle load applied to the pavement on two sets of dual tires *[equivalent single-axle load* (ESAL)]

✪ Table Appendix D

The growth factors:

$$G_{rn} = [(1 + r)^n - 1]/r$$

r = i/100 and is not zero. If annual growth is zero, growth factor = design period

i =growth rate

n = design life, yrs

<u>A general equation for the accumulated **ESAL** for each category of axle load is obtained as:</u>

$$\text{ESAL}_i = f_d \times G_{rn} \times \text{AADT}_i \times 365 \times N_i \times F_{Ei} \qquad (19.2)$$

where

ESAL_i = equivalent accumulated 18,000-lb (80 kN) single-axle load for the axle category i
f_d = design lane factor
G_{rn} = growth factor for a given growth rate r and design period n
AADT_i = first year annual average daily traffic for axle category i
N_i = number of axles on each vehicle in category i
F_{Ei} = load equivalency factor for axle category i

$f_d = D_D \times D_L$

- D_D = 0.5 typically
- D_L:

Number of Lanes in Each Direction	Percent of 18-kip ESAL in Design Lane
1	100
2	80–100
3	60–80
4	50–75

- The equivalent 18,000-lb load can also be determined from the vehicle type, *if the axle load is unknown*, by using a truck factor for that vehicle type
- The truck factor is defined as the number of 18,000-lb single-load applications caused by a single passage of a vehicle
- These have been determined for each class of vehicle from the expression

$$\text{Truck factor} = \frac{\sum(\text{number of axles} \times \text{load equivalency factor})}{\text{number of vehicles}}$$

Example 19.1 Computing Accumulated Equivalent Single-Axle Load for a Proposed Eight-Lane Highway Using Load Equivalency Factors

An eight-lane divided highway is to be constructed on a new alignment. Traffic volume forecasts indicate that the average annual daily traffic (AADT) in both directions during the first year of operation will be 12,000 with the following vehicle mix and axle loads.

Passenger cars (1000 lb/axle) = 50%
2-axle single-unit trucks (6000 lb/axle) = 33%
3-axle single-unit trucks (10,000 lb/axle) = 17%

The vehicle mix is expected to remain the same throughout the design life of the pavement. If the expected annual traffic growth rate is 4% for all vehicles, determine the design ESAL, given a design period of 20 years. The percent of traffic on the design lane is 45%, and the pavement has a terminal serviceability index (p_t) of 2.5 and SN of 5.

The following data apply:

Growth factor = 29.78 (from Table 19.4)
Percent truck volume on design lane = 45
Load equivalency factors (from Table 19.3)
Passenger cars (1000 lb/axle) = 0.00002 (negligible)
2-axle single-unit trucks (6000 lb/axle) = 0.010
3-axle single-unit trucks (10,000 lb/axle) = 0.088

Solution: The ESAL for each class of vehicle is computed from Eq. 19.2.

$$\text{ESAL} = f_d \times G_{jt} \times \text{AADT} \times 365 \times N_i \times F_{Ei}$$

$$\begin{aligned}\text{2-axle single-unit trucks} &= 0.45 \times 29.78 \times 12{,}000 \times 0.33 \times 365 \times 2 \times 0.010 \\ &= 0.3874 \times 10^6 \\ \text{3-axle single-unit trucks} &= 0.45 \times 29.78 \times 12{,}000 \times 0.17 \times 365 \times 3 \times 0.0877 \\ &= 2.6343 \times 10^6\end{aligned}$$

Thus,

$$\text{Total ESAL} = 3.0217 \times 10^6$$

It can be seen that the contribution of passenger cars to the ESAL is negligible. Passenger cars are therefore omitted when computing ESAL values. This example illustrates the conversion of axle loads to ESAL using axle load equivalency factors.

Triaxial Resilient Modulus Test

Triaxial resilient modulus test a repeated axial cyclic stress of fixed magnitude, load duration and cyclic duration is applied to a cylindrical test specimen. While the specimen is subjected to this dynamic cyclic stress, it is also subjected to a static confining stress provided by a triaxial pressure chamber. The total resilient (recoverable) axial deformation response of the specimen is measured and used to calculate the resilient modulus using the following equation:

$$M_R \left(or\ E_R\right) = \frac{\sigma_d}{\varepsilon_r}$$

Test Description

1. The specimen is a cylindrical sample normally 100 mm (4 in.) in diameter by 200 mm (8 in.) high (Figure 1a). The sample is generally compacted in the laboratory; however, undisturbed samples are best if available (which is rare).
2. The specimen is enclosed vertically by a thin “rubber” membrane and on both ends by rigid surfaces (platens) as sketched in Figure 1b.
3. The sample is placed in a pressure chamber and a confining pressure is applied (s_3) as sketched in Figure 1c.
4. The deviator stress is the axial stress applied by the testing apparatus (s_1) minus the confining stress (s_3). In other words, the deviator stress is the repeated stress applied to the sample. These stresses are further illustrated in Figure 2a.
5. The resulting strains are calculated over a gauge length, which is designated by “L” (refer to Figure 2b).
6. Basically, the initial condition of the sample is unloaded (no induced stress). When the deviator stress is applied, the sample deforms, changing in length as shown in Figure 2c. This change in sample length is directly proportional to the stiffness.

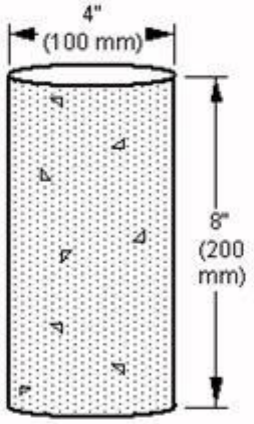


Figure 1a: Basic Triaxial Specimen Configuration

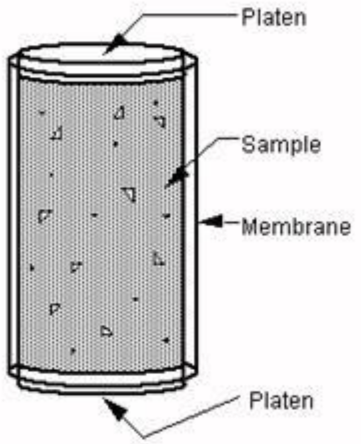


Figure 1b: Enclosure of Triaxial Specimen

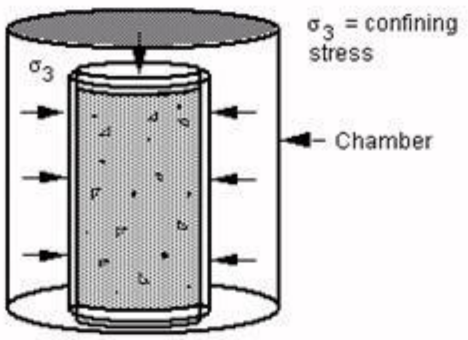


Figure 1c: Triaxial Specimen in Pressure Chamber

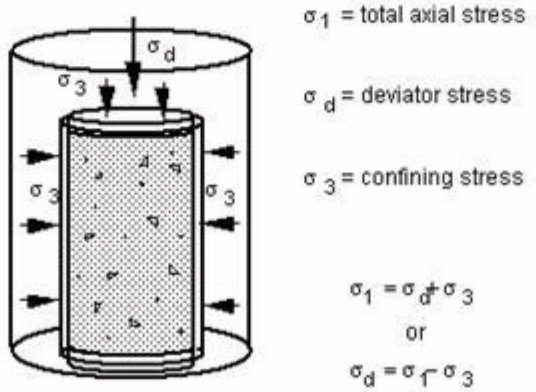


Figure 2a: Stresses Acting on Triaxial Specimen

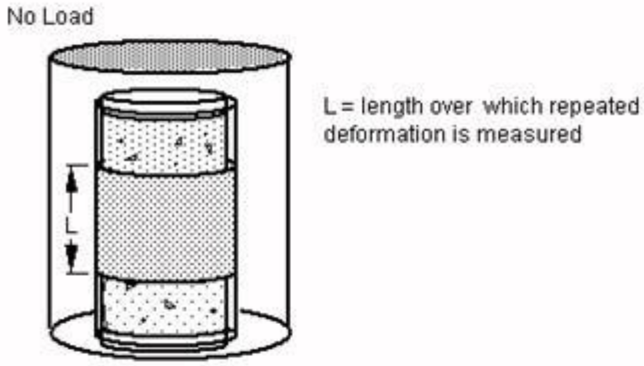


Figure 2b: Gage Length for Measurement of Strain on Triaxial Specimen

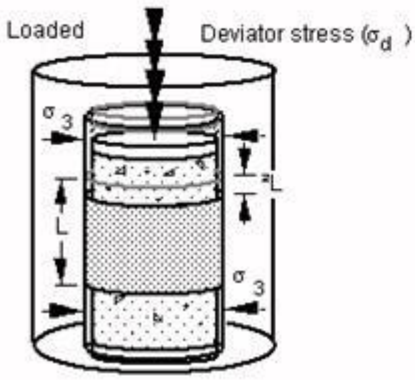


Figure 2c: Deformation of Triaxial Specimen Under Load

California Bearing Ratio (CBR) Test

$$\text{CBR} = \frac{(\text{unit load for 0.1 piston penetration in test specimen})\ (\text{lb/in}^2.)}{(\text{unit load for 0.1 piston penetration in standard crushed rock})\ (\text{lb/in}^2.)}$$

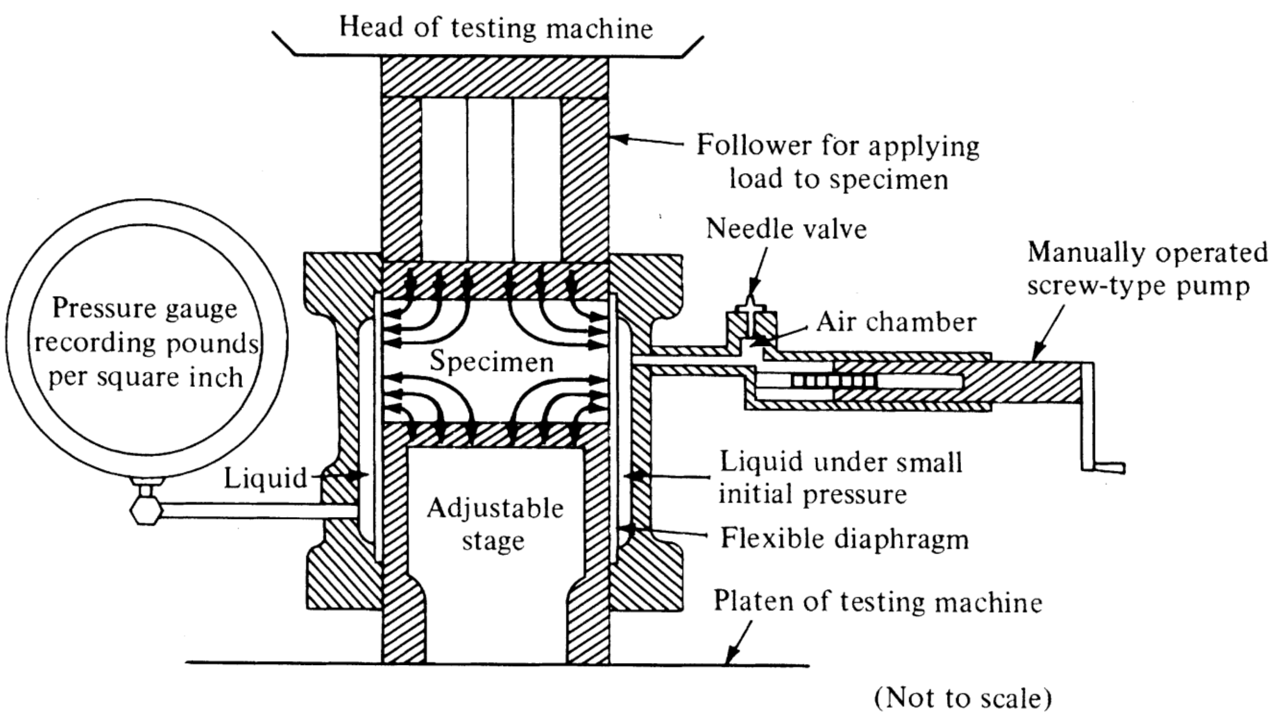
$$\text{CBR} = \frac{(\text{unit load for 0.1 piston penetration in test sample})}{1000} \times 100$$

Load Related to penetration (0.1/0.2 in = 2.5/5 mm)

Hveem Stabilometer Test

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_v}{P_h} - 1\right) + 1}$$

R = resistance value
P_v = vertical pressure (160 lb/in.^{2})
P_h = horizontal pressure at P_v of 160 lb/in.^{2} (lb/in.^{2})
D = number of turns of displacement pump



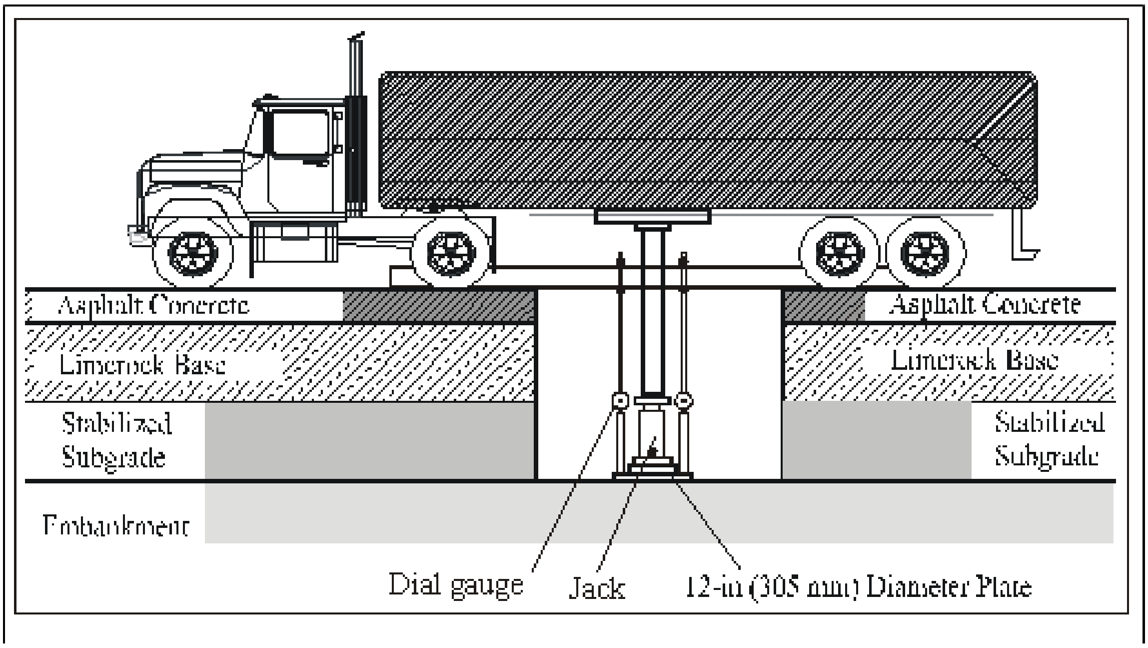
Field Static Plate Bearing Load Test

$$k = \frac{\sigma_0}{\Delta}$$

where k = modulus of subgrade reaction,
σ_0 = pressure applied to the surface of the plate, and
Δ = deflection of the plate.

The k values were calculated with σ_0 = 10 psi (68.9 kPa) on 12 in. diameter plate.

$$k(\text{pci}) = \frac{M_R(\text{psi})}{19.4}$$



Roadbed Soils (Subgrade Material)

- ✪ Resilient modulus (Mr) of the soil to define its property

$$M_r\,(\text{lb/in}^2) = 1500\text{ CBR (for fine-grain soils with soaked CBR of 10 or less)}$$

$$M_r\,(\text{lb/in}^2) = 1000 + 555\,R\text{ value (for }R \leq 20)$$

Materials of Construction

The materials used for construction can be classified under three general groups:

- ✪ Subbase (coefficient: a3) (a3 = 0.11 for sandy gravel). Figure 19.3
- ✪ Base (coefficient: a2). Figure 19.4
- ✪ Surface (coefficient: a1). Figure 19.5

Environment

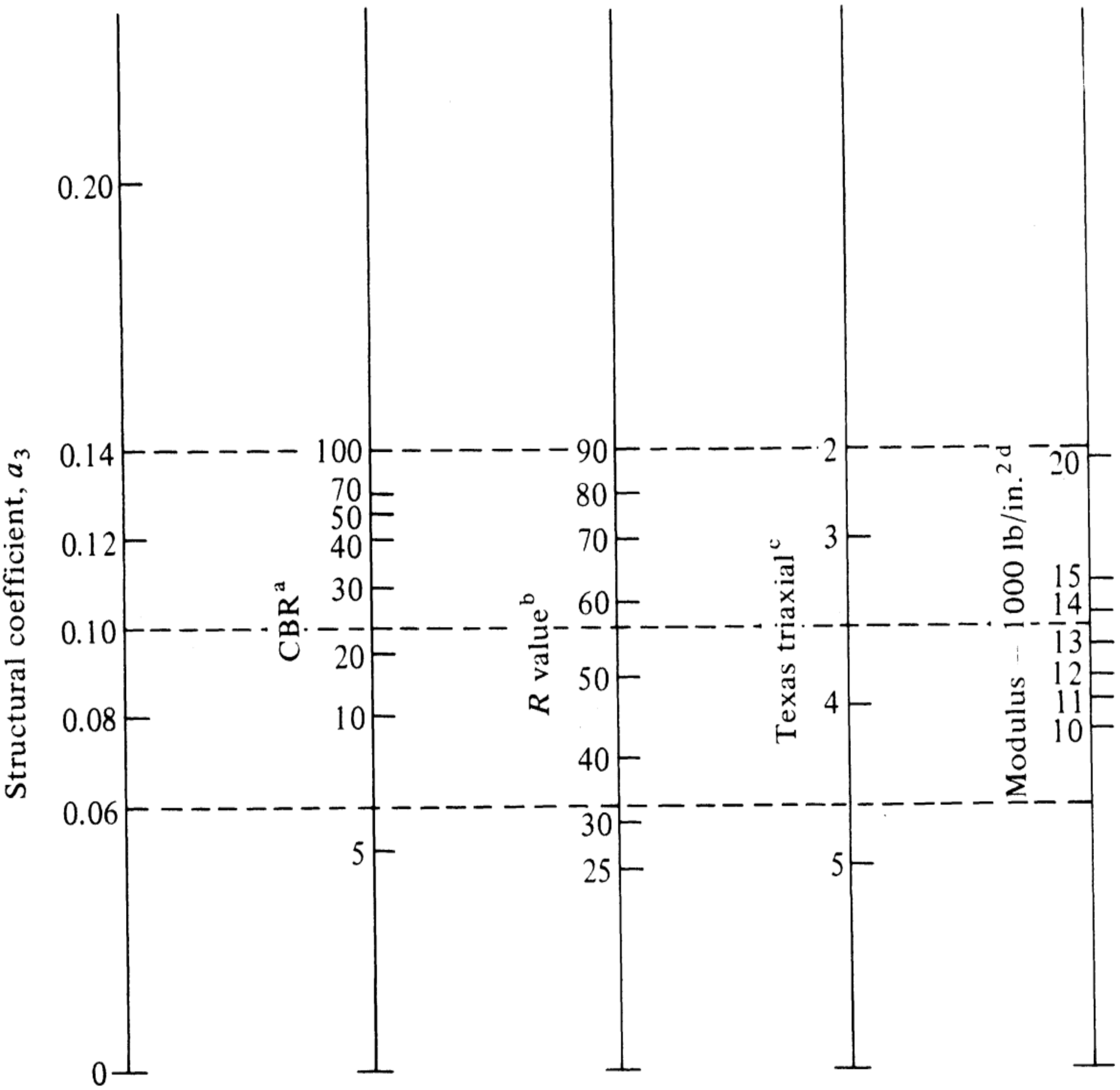
Effects of temperature on asphalt pavements include stresses induced by thermal action, changes in the creep properties, and the effect of freezing and thawing of the subgrade soil

- ✪ Procedure → variation of Mr during the year
- ☞ Find Mr for each different season (for example month) in laboratory
- ☞ Find relative damage u_f for each time period (month). **Figure 19.6**
- ☞ Find mean of u_f
- ☞ Determin effective Mr based on mean u_f using Figure 19.6

Example 19.2 Computing Effective Resilient Modulus

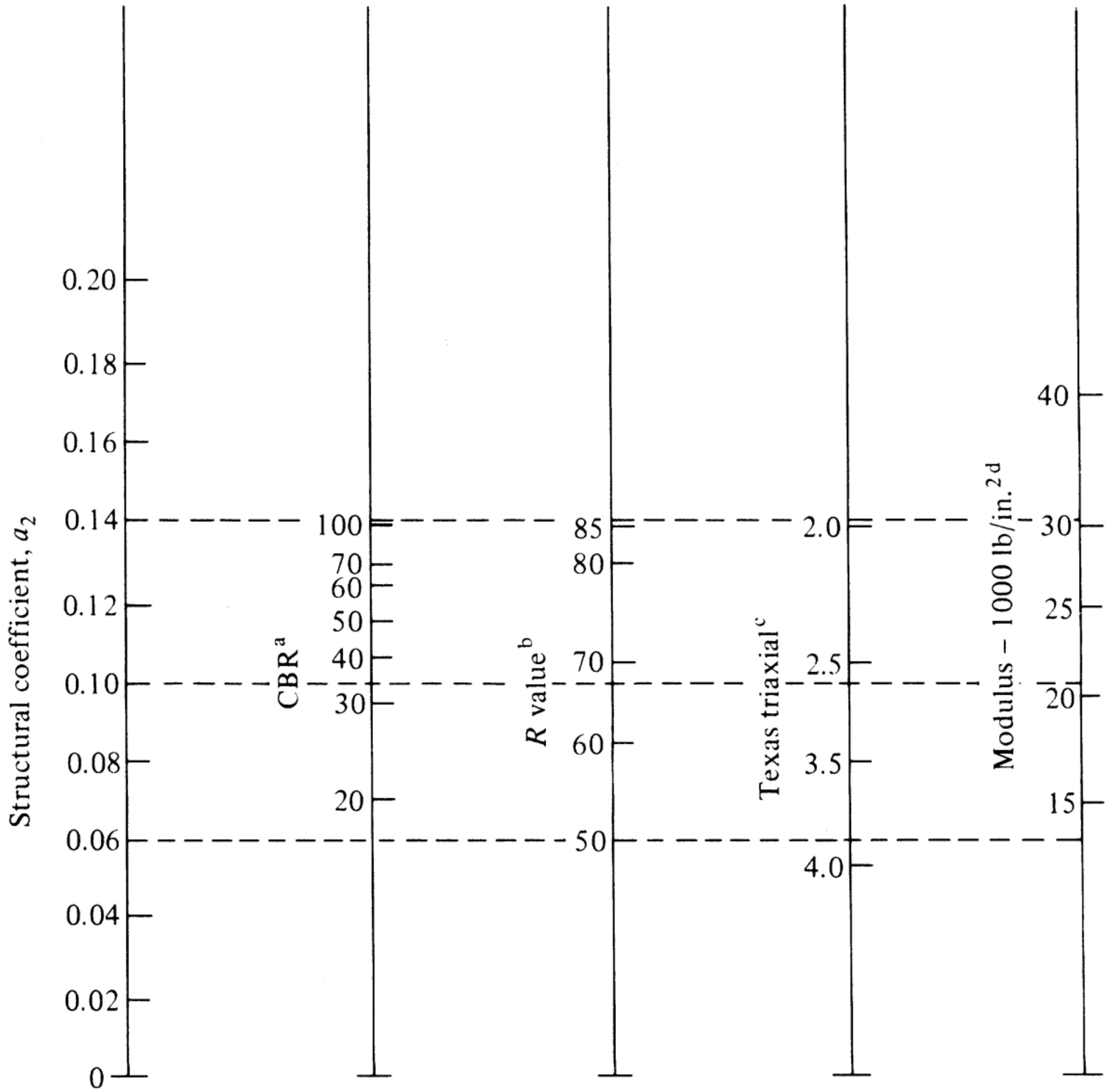
Figure 19.6 shows roadbed soil resilient modulus M_r for each month estimated from laboratory results correlating M_r with moisture content. Determine the effective resilient modulus of the subgrade.

Solution: Note that in this case, the moisture content does not vary within any one month. The solution of the problem is given in Figure 19.6. The value of u_f for each M_r is obtained directly from the chart. The mean relative damage u_f is 0.133, which in turn gives an effective resilient modulus of 7250 lb/in^2.



[a] Scale derived from correlations from Illinois.
[b] Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico, and Wyoming.
[c] Scale derived from correlations obtained from Texas.
[d] Scale derived on NCHRP project 128, 1972.

Fig 19.3



[a] Scale derived by averaging correlations obtained from Illinois.
[b] Scale derived by averaging correlations obtained from California, New Mexico, and Wyoming.
[c] Scale derived by averaging correlations obtained from Texas.
[d] Scale derived on NCHRP project 128, 1972.

Fig 19.4

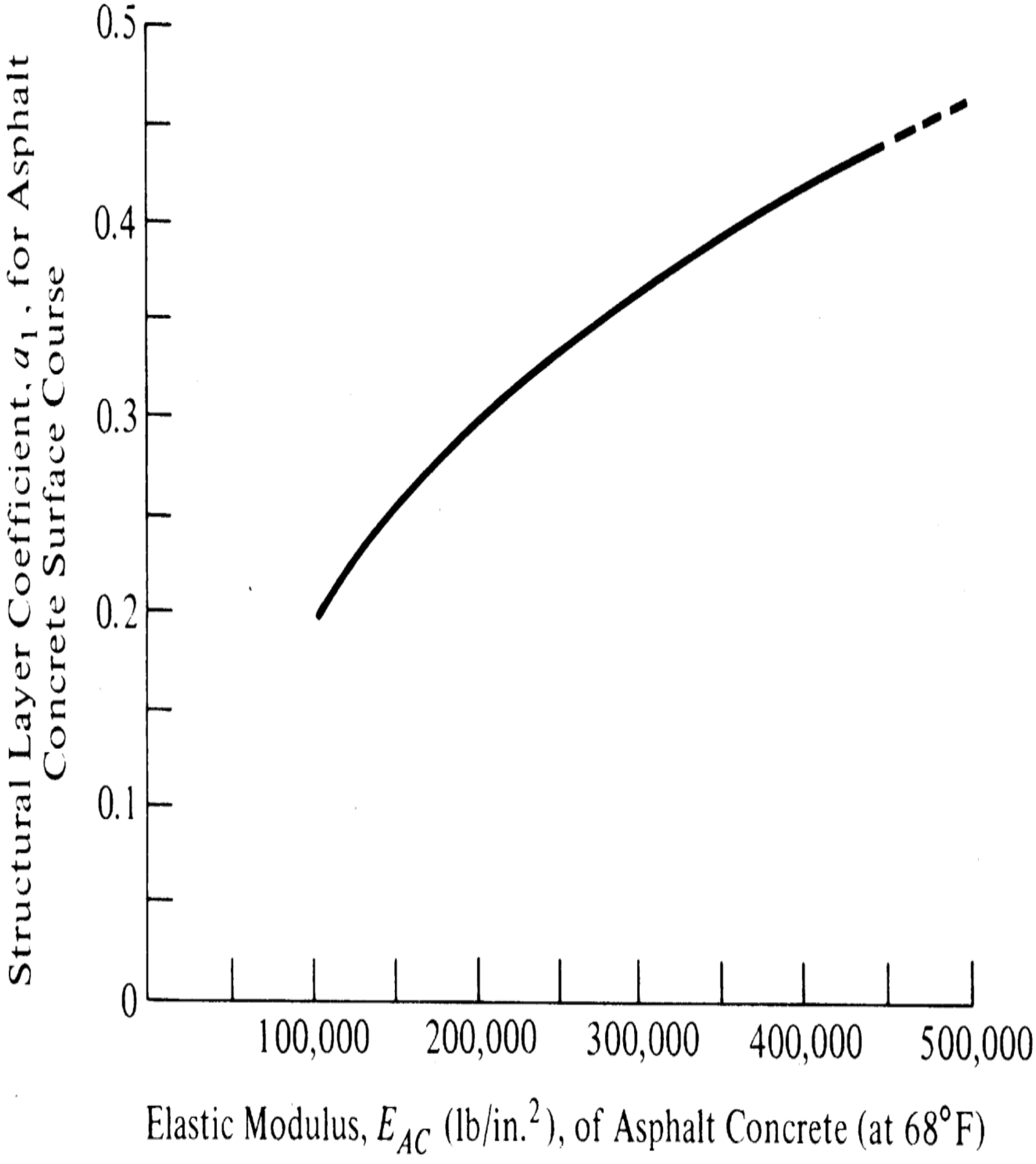
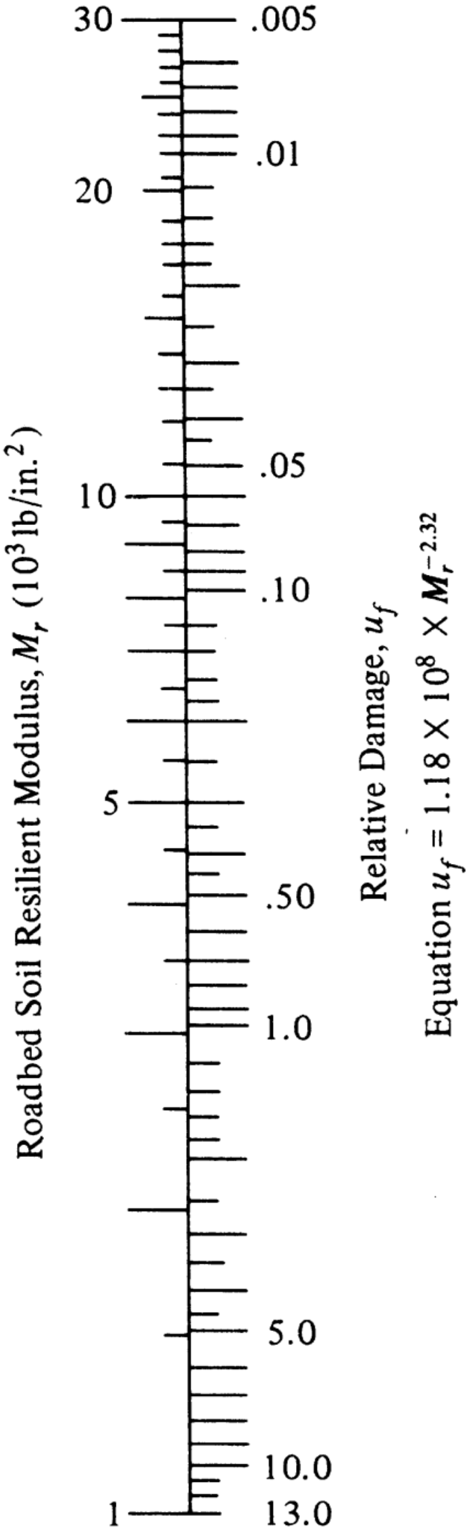

Structural Layer Coefficient, a_1, for Asphalt Concrete Surface Course
0.5
0.4
0.3
0.2
0.1
0
100,000
200,000
300,000
400,000
500,000
Elastic Modulus, E_{AC} (lb/in.^{2}), of Asphalt Concrete (at 68°F)

Fig 19.5

Month	Roadbed Soil Modulus M_r (lb/in.^{2})	Relative Damage u_f
Jan.	22000	0.01
Feb.	22000	0.01
Mar.	5500	0.25
Apr.	5000	0.30
May	5000	0.30
June	8000	0.11
July	8000	0.11
Aug.	8000	0.11
Sept.	8500	0.09
Oct.	8500	0.09
Nov.	6000	0.20
Dec.	22000	0.01
Summation: Σu_f =		1.59



$$\text{Average } \bar{u}_f = \frac{\Sigma u_f}{n} = \frac{1.59}{12} = 0.133$$

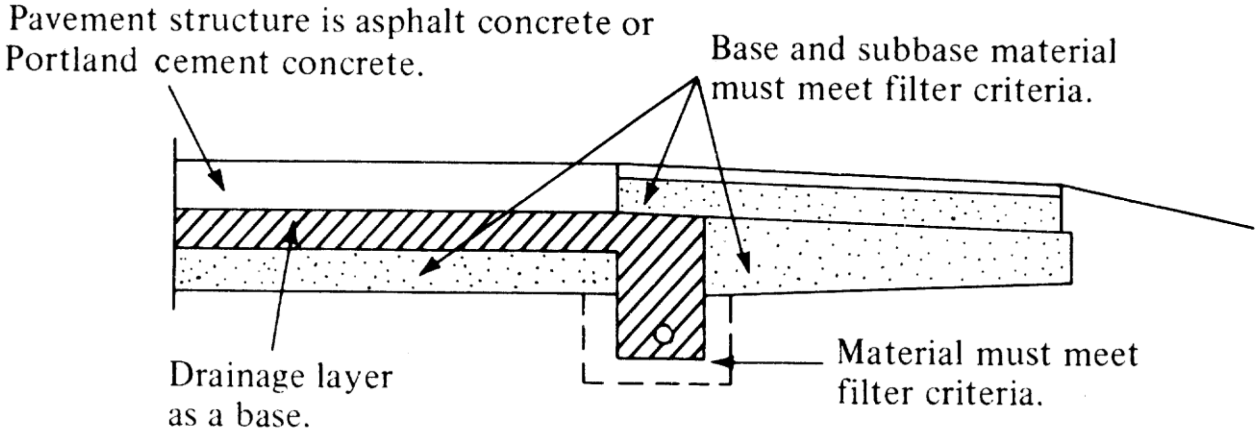
Effective Roadbed Soil Resilient Modulus, M_r (lb/in.^{2}) = <u>7250</u> (corresponds to $\bar{u}_f$)

Fig. 19.6

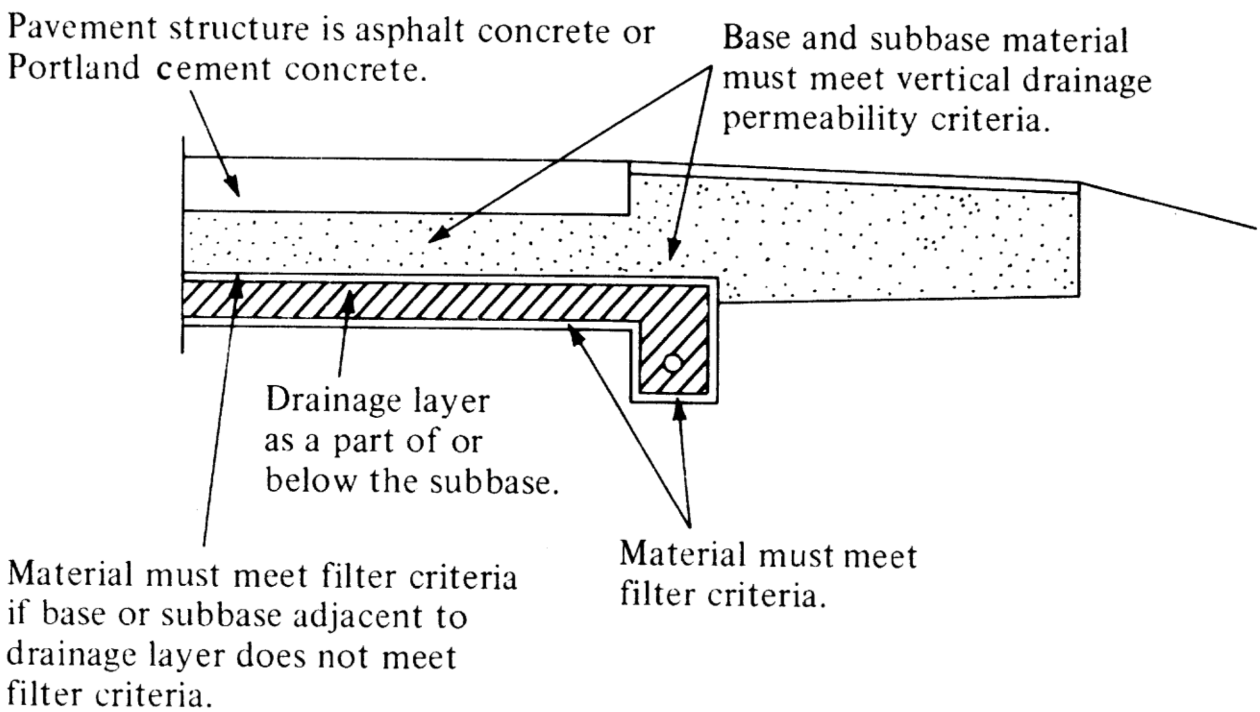
Drainage

Provide for the rapid drainage of the free water (non-capillary) from the pavement structure by providing a suitable drainage layer, as shown in Figure below

- ✪ Use tables 19.5 and 19.6 to find *mi*



(a) Base is used as the drainage layer.



(b) Drainage layer is part of or below the subbase.

Note: Filter fabrics may be used in lieu of filter material, soil, or aggregate, depending on economic considerations.

Table 19.5 Definition of Drainage Quality

Quality of Drainage	*Water Removed Within**
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	(water will not drain)

Table 19.6 Recommended m_i Values

Quality of Drainage	*Percent of Time Pavement Structure Is Exposed to Moisture Levels Approaching Saturation*			
	Less Than 1%	*1 to 5%*	*5 to 25%*	*Greater Than 25%*
Excellent	1.40–1.35	1.35–1.30	1.30–1.20	1.20
Good	1.35–1.25	1.25–1.15	1.15–1.00	1.00
Fair	1.25–1.15	1.15–1.05	1.00–0.80	0.80
Poor	1.15–1.05	1.05–0.80	0.80–0.60	0.60
Very poor	1.05–0.95	0.95–0.75	0.75–0.40	0.40

Reliability

Table 19.7 Suggested Levels of Reliability for Various Functional Classifications

	Recommended Level of Reliability	
Functional Classification	*Urban*	*Rural*
Interstate and other freeways	85–99.9	80–99.9
Other principal arterials	80–99	75–95
Collectors	80–95	75–95
Local	50–80	50–80

Note: Results based on a survey of the AASHTO Pavement Design Task Force.
SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

The reliability factor F_R is given as

$$\log_{10} F_R = -Z_R S_o \qquad (19.5)$$

where

Z_R = standard normal variate for a given reliability (R%)
S_o = estimated overall standard deviation

Table 19.8 on page 1048 gives values of Z_R for different reliability levels R.

Overall standard deviation ranges have been identified for flexible and rigid pavements as

	Standard Deviation, S_o
Flexible pavements	0.40–0.50
Rigid pavements	0.30–0.40

Table 19.8 Standard Normal Deviation (Z_R) Values Corresponding to Selected Levels of Reliability

Reliability (R%)	*Standard Normal Deviation, Z_R*
50	−0.000
60	−0.253
70	−0.524
75	−0.674
80	−0.841
85	−1.037
90	−1.282
91	−1.340
92	−1.405
93	−1.476
94	−1.555
95	−1.645
96	−1.751
97	−1.881
98	−2.054
99	−2.327
99.9	−3.090
99.99	−3.750

Structural Design

✪ Find Structural Number (SN)

$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$

where

m_i = drainage coefficient for layer i

a_1, a_2, a_3 = layer coefficients representative of surface, base, and subbase course, respectively

D_1, D_2, D_3 = actual thickness in inches of surface, base, and subbase courses, respectively

The basic design equation given in the 1993 guide is

$$\log_{10}W_{18} = Z_R S_o + 9.36\log_{10}(SN + 1) - 0.20 + \frac{\log_{10}[\Delta PSI/(4.2 - 1.5)]}{0.40 + [1094/(SN + 1)^{5.19}]} + 2.32\log_{10}M_r - 8.07 \quad (19.7)$$

where

W_{18} = predicted number of 18,000-lb (80 kN) single-axle load applications

Z_R = standard normal deviation for a given reliability

S_o = overall standard deviation

SN = structural number indicative of the total pavement thickness

$\Delta PSI = p_i - p_t$

p_i = initial serviceability index

p_t = terminal serviceability index

M_r = resilient modulus (lb/in^2)

======= ================ = ==================== ========== ======

Minimum Thickness (inches)

Traffic, ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0 (or surface treatment)	4
50,001–150,000	2.0	4
150,001–500,000	2.5	4
500,001–2,000,000	3.0	6
2,000,001–7,000,000	3.5	6
Greater than 7,000,000	4.0	6

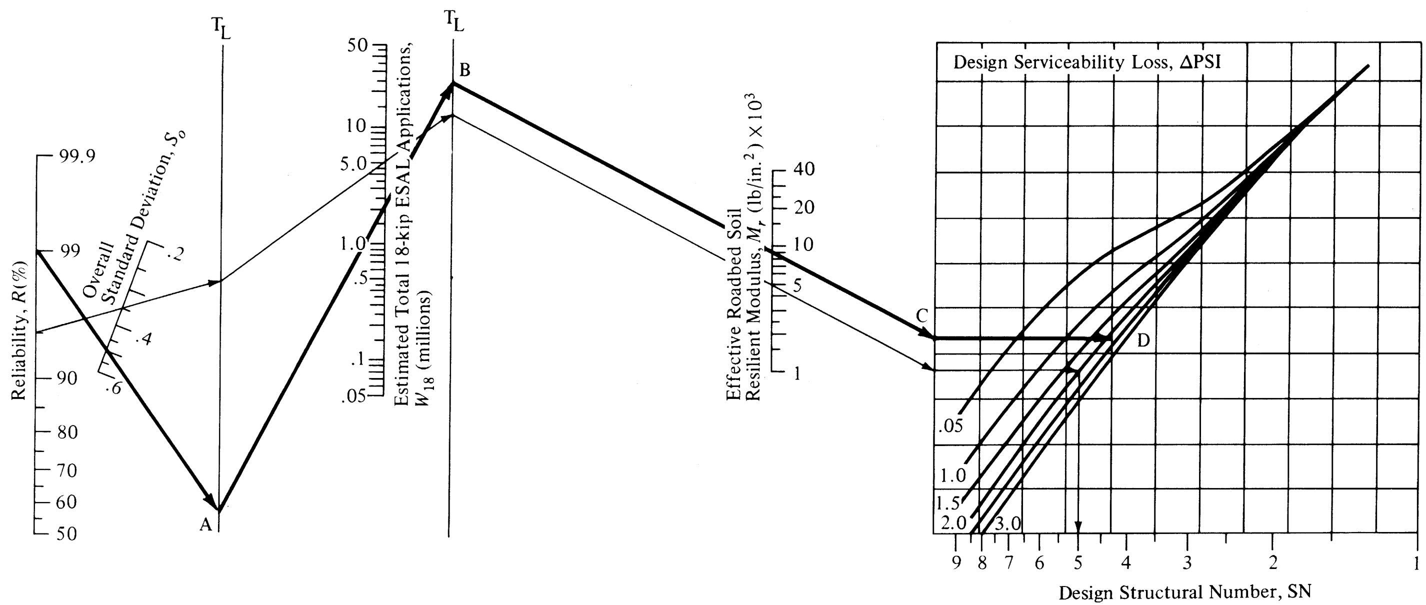


Figure 19.8 Design Chart for Flexible Pavements Based on Using Mean Values for each Input

SOURCE: Redrawn from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

Example 19.3 Designing a Flexible Pavement Using the AASHTO Method

A flexible pavement for an urban interstate highway is to be designed using the 1993 AASHTO guide procedure to carry a design ESAL of 2×10^6. It is estimated that it takes about a week for water to be drained from within the pavement and the pavement structure will be exposed to moisture levels approaching saturation for 30% of the time. The following additional information is available:

Resilient modulus of asphalt concrete at 68°F = 450,000 lb/in^2
CBR value of base course material = 100, M_r = 31,000 lb/in^2
CBR value of subbase course material = 22, M_r = 13,500 lb/in^2
CBR value of subgrade material = 6

Determine a suitable pavement structure, M_r of subgrade = 6 × 1500 lb/in^2 = 9000 lb/in^2.

Solution: Since the pavement is to be designed for an interstate highway, the following assumptions are made.

Reliability level (R) = 99% (range is 85 to 99.9 from Table 19.7)
Standard deviation (S_o) = 0.49 (range is 0.4 to 0.5)
Initial serviceability index $p_i = 4.5$
Terminal serviceability index $p_t = 2.5$

The nomograph in Figure 19.8 is used to determine the design SN through the following steps.

Step 1. Draw a line joining the reliability level of 99% and the overall standard deviation S_o of 0.49, and extend this line to intersect the first T_L line at point A.
Step 2. Draw a line joining point A to the ESAL of 2×10^6, and extend this line to intersect the second T_L line at point B.
Step 3. Draw a line joining point B and resilient modulus (M_r) of the roadbed soil, and extend this line to intersect the design serviceability loss chart at point C.
Step 4. Draw a horizontal line from point C to intersect the design serviceability loss (ΔPSI) curve at point D. In this problem, ΔPSI = 4.5 – 2.5 = 2.

Step 5. Draw a vertical line to intersect the design SN, and read this value SN = 4.4.

Step 6. Determine the appropriate structure layer coefficient for each construction material.

(a) Resilient value of asphalt = 450,000 lb/in^2. From Figure 19.5, $a_1 = 0.44$.
(b) CBR of base course material = 100. From Figure 19.4, $a_2 = 0.14$.
(c) CBR of subbase course material = 22. From Figure 19.3, $a_3 = 0.10$.

Step 7. Determine appropriate drainage coefficient m_i. Since only one set of conditions is given for both the base and subbase layers, the same value will be used for m_1 and m_2. The time required for water to drain from within pavement = 1 week, and from Table 19.5, drainage quality is fair. The percentage of time pavement structure will be exposed to moisture levels approaching saturation = 30, and from Table 19.6, $m_i = 0.80$.

Step 8. Determine appropriate layer thicknesses from Eq. 19.6:

$$= a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

It can be seen that several values of D_1, D_2, and D_3 can be obtained to satisfy the SN value of 4.40. Layer thicknesses, however, are usually rounded up to the nearest 0.5 inches.

The selection of different layer thicknesses also should be based on constraints associated with maintenance and construction practices so that a practical design is obtained. For example, it is normally impractical and uneconomical to construct any layer with a thickness less than some minimum value. Table 19.9 on page 1052 lists minimum thicknesses suggested by AASHTO.

Taking into consideration that a flexible pavement structure is a layered system, the determination of the different thicknesses should be carried out as indicated in Figure 19.9. The required SN above the subgrade is first determined, and then the required SNs above the base and subbase layers are determined using the appropriate strength of each layer. The minimum allowable thickness of each layer can then be determined using the differences of the computed SNs as shown in Figure 19.9.

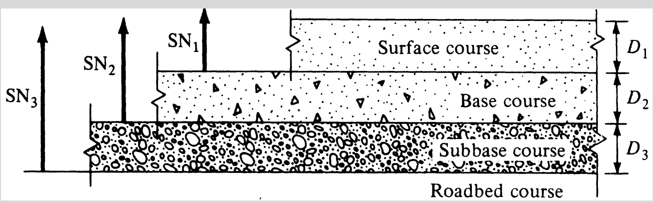


Figure 19.9 Procedure for Determining Thicknesses of Layers Using a Layered Analysis Approach

SOURCE: Redrawn from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

Table 19.9 AASHTO-Recommended Minimum Thicknesses of Highway Layers

	Minimum Thickness (in.)	
Traffic, ESALs	*Asphalt Concrete*	*Aggregate Base*
Less than 50,000	1.0 (or surface treatment)	4
50,001–150,000	2.0	4
150,001–500,000	2.5	4
500,001–2,000,000	3.0	6
2,000,001–7,000,000	3.5	6
Greater than 7,000,000	4.0	6

SOURCE: Adapted with permission from *AASTHO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

Using the appropriate values for M_r in Figure 19.8, we obtain $SN_3 = 4.4$ and $SN_2 = 3.8$. Note that when SN is assumed to compute ESAL, the assumed and computed SN_3 values must be approximately equal. If these are significantly different, the computation must be repeated with a new assumed SN.

We know

$$M_r \text{ for base course} = 31{,}000 \text{ lb/in.}^2$$

Using this value in Figure 19.8, we obtain

$$SN_1 = 2.6$$

giving

$$D_1 = \frac{2.6}{0.44} = 5.9 \text{ in.}$$

Using 6 in., for the thickness of the surface course,

$$D_1^* = 6 \text{ in.}$$

$$SN_1^* = a_1 D_1^* = 0.44 \times 6 = 2.64$$

$$D_2^* \geq \frac{SN_2 - SN_1^*}{a_2 m_2} \geq \frac{3.8 - 2.64}{0.14 \times 0.8} \geq 10.36 \text{ in.} \qquad \text{(use 12 in.)}$$

$$SN_2^* = 0.14 \times 0.8 \times 12 + 2.64 = 1.34 + 2.64$$

$$D_3^* = \frac{SN_3 - SN_2^*}{a_3 m_3} = \frac{4.4 - (2.64 + 1.34)}{0.1 \times 0.8} = 5.25 \text{ in.} \qquad \text{(use 6 in.)}$$

$$SN_3^* = 2.64 + 1.34 + 6 \times 0.8 \times 0.1 = 4.46$$

The pavement will therefore consist of 6 in. asphalt concrete surface, 12 in. granular base, and 6 in. subbase.

*An asterisk with *D* or SN indicates that it represents the value actually used which must be equal to or greater than the required value.

Table D.1. Axle load equivalency factors for flexible pavements, single axles and p_t of 2.0.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0002	.0002	.0002	.0002	.0002	.0002
4	.002	.003	.002	.002	.002	.002
6	.009	.012	.011	.010	.009	.009
8	.030	.035	.035	.033	.031	.029
10	.075	.085	.090	.085	.079	.076
12	.166	.177	.189	.183	.174	.158
14	.325	.338	.354	.350	.338	.331
16	.589	.598	.613	.612	.603	.595
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.81	1.69	1.56	1.55	1.57	1.59
22	2.49	2.44	2.35	2.31	2.35	2.41
24	3.71	3.52	3.43	3.33	3.40	3.51
26	5.36	5.21	4.88	4.68	4.77	4.96
28	7.54	7.31	6.78	6.42	6.52	8.83
30	10.4	10.0	9.2	8.6	8.7	9.2
32	14.0	13.5	12.4	11.5	11.5	12.1
34	18.5	17.9	16.3	15.0	14.9	15.5
36	24.2	23.3	21.2	19.3	19.0	19.9
38	31.1	29.9	27.1	24.5	24.0	25.1
40	39.5	38.0	34.3	30.9	30.0	31.2
42	49.7	47.7	43.0	38.6	37.2	38.5
44	61.8	69.3	53.4	47.6	45.7	47.1
46	76.1	73.0	65.6	58.3	55.7	57.0
48	92.9	89.1	80.0	70.9	57.3	68.6
50	113.	108.	97.	88.	81.	82.

Table D.2. Axle load equivalency factors for flexible pavements, tandem axles and p_t of 2.0.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0000	.0000	.0000	.0000	.0000	.0000
4	.0003	.0003	.0003	.0002	.0002	.0002
6	.001	.001	.001	.001	.001	.001
8	.003	.003	.003	.003	.003	.002
10	.007	.008	.008	.007	.006	.006
12	.013	.016	.016	.014	.013	.012
14	.024	.029	.029	.026	.024	.023
16	.041	.048	.050	.046	.042	.040
18	.066	.077	.081	.075	.069	.066
20	.103	.117	.124	.117	.109	.105
22	.156	.171	.183	.174	.164	.158
24	.227	.244	.260	.252	.239	.231
26	.322	.340	.360	.353	.338	.329
28	.447	.465	.487	.481	.466	.455
30	.607	.623	.646	.643	.627	.617
32	.810	.823	.843	.842	.829	.819
34	1.06	1.07	1.08	1.08	1.08	1.07
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.76	1.75	1.73	1.72	1.73	1.74
40	2.22	2.19	2.15	2.13	2.16	2.18
42	2.77	2.73	2.64	2.62	2.66	2.70
44	3.42	3.36	3.23	3.18	3.24	3.31
46	4.20	4.11	3.92	3.83	3.91	4.02
48	5.10	4.98	4.72	4.58	4.68	4.83
50	6.15	5.99	5.64	5.44	5.56	5.77
52	7.37	7.16	6.71	6.43	6.56	6.83
54	8.77	8.51	7.93	7.55	7.69	8.03
56	10.4	10.1	9.3	8.8	9.0	9.4
58	12.2	11.8	10.9	10.3	10.4	10.9
60	14.3	13.8	12.7	11.9	12.0	12.6
62	16.6	16.0	14.7	13.7	13.8	14.6
64	19.3	18.6	17.0	15.8	15.8	16.6
66	22.2	21.4	19.6	18.0	18.0	18.9
68	25.5	24.6	22.4	20.6	20.5	21.5
70	29.2	28.1	25.6	23.4	23.2	24.3
72	33.3	32.0	29.1	26.5	26.2	27.4
74	37.8	36.4	33.0	30.0	29.4	30.8
76	42.8	41.2	37.3	33.8	33.1	34.5
78	48.4	46.5	42.0	38.0	37.0	38.6
80	54.4	52.3	47.2	42.5	41.3	43.0
82	61.1	58.7	52.9	47.6	46.0	47.8
84	68.4	65.7	59.2	53.0	51.2	53.0
86	76.3	73.3	66.0	59.0	56.8	58.6
88	85.0	81.6	73.4	65.5	62.8	64.7
90	94.4	90.6	81.5	72.6	69.4	71.3

Table D.3. Axle load equivalency factors for flexible pevements, triple axles and p_t of 2.0.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0000	.0000	.0000	.0000	.0000	.0000
4	.0001	.0001	.0001	.0001	.0001	.0001
6	.0004	.0004	.0003	.0003	.0003	.0003
8	.0009	.0010	.0009	.0008	.0007	.0007
10	.002	.002	.002	.002	.002	.001
12	.004	.004	.004	.003	.003	.003
14	.006	.007	.007	.006	.006	.005
16	.010	.012	.012	.010	.009	.009
18	.016	.019	.019	.017	.015	.015
20	.024	.029	.029	.026	.024	.023
22	.034	.042	.042	.038	.035	.034
24	.049	.058	.060	.066	.061	.048
26	.068	.080	.083	.077	.071	.068
28	.093	.107	.113	.105	.098	.094
30	.126	.140	.149	.140	.131	.128
32	.164	.182	.194	.184	.173	.167
34	.213	.233	.248	.238	.225	.217
36	.273	.294	.313	.303	.288	.279
38	.346	.368	.390	.381	.364	.363
40	.434	.456	.481	.473	.464	.443
42	.538	.560	.587	.680	.561	.548
44	.662	.682	.710	.705	.686	.673
46	.807	.826	.852	.849	.831	.818
48	.976	.992	1.015	1.014	.999	.987
50	1.17	1.18	1.20	1.20	1.19	1.18
52	1.40	1.40	1.42	1.42	1.41	1.40
54	1.68	1.66	1.86	1.66	1.66	1.66
56	1.96	1.96	1.93	1.93	1.94	1.94
58	2.29	2.27	2.24	2.23	2.26	2.27
60	2.67	2.64	2.59	2.57	2.80	2.63
62	3.10	3.06	2.98	2.95	2.99	3.04
64	3.59	3.53	3.41	3.37	3.42	3.49
66	4.13	4.06	3.89	3.83	3.90	3.99
68	4.73	4.63	4.43	4.34	4.42	4.54
70	5.40	5.28	5.03	4.90	5.00	5.15
72	6.15	6.00	5.68	5.62	5.63	5.82
74	6.97	6.79	6.41	6.20	6.33	6.56
76	7.88	7.67	7.21	6.94	7.08	7.36
78	8.88	8.63	8.09	7.75	7.90	8.23
80	9.98	9.69	9.05	8.63	8.79	9.18
82	11.2	10.8	10.1	9.6	9.8	10.2
84	12.5	12.1	11.2	10.6	10.8	11.3
86	13.9	13.5	12.5	11.8	11.9	12.5
88	15.5	16.0	13.8	13.0	13.2	13.8
90	17.2	18.6	15.3	14.3	14.5	15.2

Table D.4. Axle load equivalency factors for flexible pavements, single axles and p_t 2.5.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0004	.0004	.0003	.0002	.0002	.0002
4	.003	.004	.004	.003	.002	.002
6	.011	.017	.017	.013	.010	.009
8	.032	.047	.051	.041	.034	.031
10	.078	.102	.118	.102	.088	.080
12	.168	.198	.229	.213	.189	.176
14	.328	.358	.399	.388	.360	.342
16	.591	.613	.646	.645	.623	.608
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.57	1.49	1.47	1.51	1.56
22	2.48	2.38	2.17	2.09	2.18	2.30
24	3.69	3.49	3.09	2.89	3.03	3.27
26	5.33	4.99	4.31	3.91	4.09	4.48
28	7.49	6.98	5.90	5.21	5.39	5.98
30	10.3	9.5	7.9	6.8	7.0	7.8
32	13.9	12.8	10.5	8.8	8.9	10.0
34	18.4	16.9	13.7	11.3	11.2	12.5
36	24.0	22.0	17.7	14.4	13.9	15.5
38	30.9	28.3	22.6	18.1	17.2	19.0
40	39.3	35.9	28.5	22.6	21.1	23.0
42	49.3	45.0	35.6	27.8	25.6	27.7
44	61.3	55.9	44.0	34.0	31.0	33.1
46	75.5	68.8	54.0	41.4	37.2	39.3
48	92.2	83.9	65.7	50.1	44.5	46.5
50	112.	102.	79.	60.	53.	55.

Table D.5. Axle load equivalency factors for flexible pavements, tandem axles and p_t of 2.5.

Axle Load (kips)	Pavement Structural Number (SN) 1	2	3	4	5	6
2	.0001	.0001	.0001	.0000	.0000	.0000
4	.0005	.0005	.0004	.0003	.0003	.0002
6	.002	.002	.002	.001	.001	.001
8	.004	.006	.005	.004	.003	.003
10	.008	.013	.011	.009	.007	.006
12	.015	.024	.023	.018	.014	.013
14	.026	.041	.042	.033	.027	.024
16	.044	.065	.070	.057	.047	.043
18	.070	.097	.109	.092	.077	.070
20	.107	.141	.162	.141	.121	.110
22	.160	.198	.229	.207	.180	.166
24	.231	.273	.315	.292	.260	.242
26	.327	.370	.420	.401	.364	.342
28	.461	.493	.548	.534	.495	.470
30	.611	.648	.703	.695	.658	.633
32	.813	.843	.889	.887	.867	.834
34	1.06	1.08	1.11	1.11	1.09	1.08
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.76	1.73	1.69	1.68	1.70	1.73
40	2.21	2.16	2.06	2.03	2.08	2.14
42	2.76	2.67	2.49	2.43	2.51	2.61
44	3.41	3.27	2.99	2.88	3.00	3.16
46	4.18	3.98	3.58	3.40	3.55	3.79
48	5.08	4.80	4.25	3.98	4.17	4.49
50	6.12	5.76	5.03	4.64	4.86	5.28
52	7.33	6.87	5.93	5.38	5.63	6.17
54	8.72	8.14	6.95	6.22	6.47	7.15
56	10.3	9.6	8.1	7.2	7.4	8.2
58	12.1	11.3	9.4	8.2	8.4	9.4
60	14.2	13.1	10.9	9.4	9.6	10.7
62	16.5	15.3	12.6	10.7	10.8	12.1
64	19.1	17.6	14.5	12.2	12.2	13.7
66	22.1	20.3	16.6	13.8	13.7	15.4
68	25.3	23.3	18.9	15.6	15.4	17.2
70	29.0	26.6	21.5	17.6	17.2	19.2
72	33.0	30.3	24.4	19.8	19.2	21.3
74	37.5	34.4	27.6	22.2	21.3	23.6
76	42.5	38.9	31.1	24.8	23.7	26.1
78	48.0	43.9	35.0	27.8	26.2	28.8
80	54.0	49.4	39.2	30.9	29.0	31.7
82	60.6	55.4	43.9	34.4	32.0	34.8
84	67.8	61.9	49.0	38.2	35.3	38.1
86	75.7	69.1	54.5	42.3	38.8	41.7
88	84.3	76.9	60.6	46.8	42.6	45.6
90	93.7	85.4	67.1	51.7	46.8	49.7

Table D.6. Axle load equivalency factors for flexible pavements, triple axles and p_t of 2.5.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0000	.0000	.0000	.0000	.0000	.0000
4	.0002	.0002	.0002	.0001	.0001	.0001
6	.0006	.0007	.0005	.0004	.0003	.0003
8	.001	.002	.001	.001	.001	.001
10	.003	.004	.003	.002	.002	.002
12	.005	.007	.006	.004	.003	.003
14	.008	.012	.010	.008	.006	.005
16	.012	.019	.018	.013	.011	.010
18	.018	.029	.028	.021	.017	.016
20	.027	.042	.042	.032	.027	.024
22	.038	.058	.060	.048	.040	.038
24	.053	.078	.084	.068	.057	.051
26	.072	.103	.114	.095	.080	.072
28	.098	.133	.151	.128	.109	.089
30	.129	.169	.196	.170	.146	.133
32	.169	.213	.247	.220	.191	.175
34	.219	.266	.308	.281	.246	.228
36	.279	.329	.379	.362	.313	.292
38	.352	.403	.461	.436	.393	.368
40	.439	.491	.554	.533	.487	.459
42	.543	.594	.661	.644	.597	.567
44	.666	.714	.781	.769	.723	.692
46	.811	.864	.918	.911	.868	.838
48	.979	1.015	1.072	1.069	1.033	1.005
50	1.17	1.20	1.24	1.25	1.22	1.20
52	1.40	1.41	1.44	1.44	1.43	1.41
54	1.66	1.66	1.66	1.66	1.66	1.66
56	1.95	1.93	1.90	1.90	1.91	1.93
58	2.29	2.25	2.17	2.16	2.20	2.24
60	2.67	2.60	2.48	2.44	2.51	2.58
62	3.09	3.00	2.82	2.76	2.85	2.95
64	3.57	3.44	3.19	3.10	3.22	3.38
66	4.11	3.94	3.61	3.47	3.62	3.81
68	4.71	4.49	4.06	3.88	4.05	4.30
70	5.38	5.11	4.57	4.32	4.52	4.84
72	6.12	5.79	5.13	4.80	5.03	5.41
74	6.93	6.54	5.74	5.32	5.57	6.04
76	7.84	7.37	6.41	5.88	6.15	6.71
78	8.83	8.28	7.14	6.49	6.78	7.43
80	9.92	9.28	7.95	7.15	7.45	8.21
82	11.1	10.4	8.8	7.9	8.2	9.0
84	12.4	11.6	9.8	8.6	8.9	9.9
86	13.8	12.9	10.8	9.5	9.8	10.9
88	15.4	14.3	11.9	10.4	10.6	11.9
90	17.1	15.8	13.2	11.3	11.6	12.9

Table D.7. Axle load equivalency factors for flexible pavements, single axles and p_t of 3.0.

Axle Load (kips)	Pavement Structural Number (SN) 1	2	3	4	5	6
2	.0008	.0009	.0006	.0003	.0002	.0002
4	.004	.008	.006	.004	.002	.002
6	.014	.030	.028	.018	.012	.010
8	.035	.070	.080	.055	.040	.034
10	.082	.132	.168	.132	.101	.086
12	.173	.231	.296	.260	.212	.187
14	.332	.388	.468	.447	.391	.358
16	.594	.633	.695	.693	.651	.622
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.60	1.53	1.41	1.38	1.44	1.51
22	2.47	2.29	1.96	1.83	1.97	2.16
24	3.67	3.33	2.69	2.39	2.60	2.98
26	5.29	4.72	3.65	3.08	3.33	3.91
28	7.43	6.56	4.88	3.93	4.17	5.00
30	10.2	8.9	6.5	5.0	5.1	6.3
32	13.8	12.0	8.4	6.2	6.3	7.7
34	18.2	15.7	10.9	7.8	7.8	9.3
36	23.8	20.4	14.0	9.7	9.1	11.0
38	30.6	26.2	17.7	11.9	11.0	13.0
40	38.8	33.2	22.2	14.6	13.1	15.3
42	48.8	41.6	27.6	17.8	15.6	17.8
44	60.6	51.6	34.0	21.6	18.4	20.6
46	74.7	63.4	41.5	26.1	21.6	23.8
48	91.2	77.3	50.3	31.3	25.4	27.4
50	110.	94.	61.	37.	30.	32.

Table D.8. Axle load equivalency factors for flexible pavements, tandem axles and p_t of 3.0.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0002	.0002	.0001	.0001	.0000	.0000
4	.001	.001	.001	.000	.000	.000
6	.003	.004	.003	.002	.001	.001
8	.006	.011	.009	.005	.003	.003
10	.011	.024	.020	.012	.008	.007
12	.019	.042	.039	.024	.017	.014
14	.031	.066	.068	.045	.032	.026
16	.049	.096	.109	.076	.055	.046
18	.075	.134	.164	.121	.090	.076
20	.113	.181	.232	.182	.139	.119
22	.166	.241	.313	.260	.205	.178
24	.238	317	.407	.358	.292	.257
26	.333	.413	.517	.476	.402	.360
28	.457	.534	.643	.614	.538	.492
30	.616	.684	.788	.773	.702	.656
32	.817	.870	.956	.953	.896	.856
34	1.07	1.10	1.16	1.15	1.12	1.09
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.75	1.71	1.64	1.62	1.66	1.70
40	2.21	2.11	1.94	1.89	1.98	2.08
42	2.75	2.59	2.29	2.19	2.33	2.50
44	3.39	3.15	2.70	2.52	2.71	2.97
46	4.15	3.81	3.16	2.89	3.13	3.50
48	5.04	4.58	3.70	3.29	3.57	4.07
50	6.08	5.47	4.31	3.74	4.05	4.70
52	7.27	6.49	5.01	4.24	4.57	5.37
54	8.65	7.67	5.81	4.79	5.13	6.10
56	10.2	9.0	6.7	5.4	5.7	6.9
58	12.0	10.6	7.7	6.1	6.4	7.7
60	14.1	12.3	8.9	6.8	7.1	8.6
62	16.3	14.2	10.2	7.7	7.8	9.5
64	18.9	16.4	11.6	8.6	8.6	10.5
66	21.8	18.9	13.2	9.6	9.5	11.6
68	25.1	21.7	15.0	10.7	10.5	12.7
70	28.7	24.7	17.0	12.0	11.5	13.9
72	32.7	28.1	19.2	13.3	12.6	15.2
74	37.2	31.9	21.6	14.8	13.8	16.5
76	42.1	36.0	24.3	16.4	15.1	17.9
78	47.5	40.6	27.3	18.2	16.5	19.4
80	53.4	45.7	30.5	20.1	18.0	21.0
82	60.0	51.2	34.0	22.2	19.6	22.7
84	67.1	57.2	37.9	24.6	21.3	24.5
86	74.9	63.8	42.1	27.1	23.2	26.4
88	83.4	71.0	46.7	29.8	25.2	28.4
90	92.7	78.8	51.7	32.7	27.4	30.5

Table D.9. Axle load equivalency factors for flexible pavements, ~~tandem~~ axles and p_t of 3.0.
TRIPLE

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0001	.0001	.0001	.0000	.0000	.0000
4	.0005	.0004	.0003	.0002	.0001	.0001
6	.001	.001	.001	.001	.000	.000
8	.003	.004	.002	.001	.001	.001
10	.005	.008	.005	.003	.002	.002
12	.007	.014	.010	.006	.004	.003
14	.011	.023	.018	.011	.007	.006
16	.016	.035	.030	.018	.013	.010
18	.022	.050	.047	.029	.020	.017
20	.031	.069	.069	.044	.031	.026
22	.043	.090	.097	.065	.046	.039
24	.059	.116	.132	.092	.066	.056
26	.079	.145	.174	.126	.092	.078
28	.104	.179	.223	.168	.126	.107
30	.136	.218	.279	.219	.167	.143
32	.176	.265	.342	.279	.218	.188
34	.226	.319	.413	.350	.279	.243
36	.286	.382	.491	.432	.352	.310
38	.359	.456	.577	.524	.437	.389
40	.447	.543	.671	.626	.536	.483
42	.550	.643	.775	.740	.649	.593
44	.673	.760	.889	.865	.777	.720
46	.817	.894	1.014	1.001	.920	.865
48	.984	1.048	1.152	1.148	1.080	1.030
50	1.18	1.23	1.30	1.31	1.26	1.22
52	1.40	1.43	1.47	1.48	1.45	1.43
54	1.66	1.66	1.66	1.66	1.66	1.66
56	1.95	1.92	1.86	1.85	1.88	1.91
58	2.28	2.21	2.09	2.06	2.13	2.20
60	2.66	2.54	2.34	2.28	2.39	2.50
62	3.08	2.92	2.61	2.52	2.66	2.84
64	3.56	3.33	2.92	2.77	2.96	3.19
66	4.09	3.79	3.25	3.04	3.27	3.58
68	4.68	4.31	3.62	3.33	3.60	4.00
70	5.34	4.88	4.02	3.64	3.94	4.44
72	6.08	5.51	4.46	3.97	4.31	4.91
74	6.89	6.21	4.94	4.32	4.69	5.40
76	7.78	6.98	5.47	4.70	5.09	5.93
78	8.76	7.83	6.04	5.11	5.51	6.48
80	9.84	8.75	6.67	5.54	5.96	7.06
82	11.0	9.8	7.4	6.0	6.4	7.7
84	12.3	10.9	8.1	6.5	6.9	8.3
86	13.7	12.1	8.9	7.0	7.4	9.0
88	15.3	13.4	9.8	7.6	8.0	9.6
90	16.9	14.8	10.7	8.2	8.5	10.4

Table D.10. Axle load equivalency factors for rigid pavements, singla axles and p_t of 2.0.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002
4	.002	.002	.002	.002	.002	.002	.002	.002	.002
6	.011	.010	.010	.010	.010	.010	.010	.010	.010
8	.035	.033	.032	.032	.032	.032	.032	.032	.032
10	.087	.084	.082	.081	.080	.080	.080	.080	.080
12	.186	.180	.176	.175	.174	.174	.173	.173	.173
14	.353	.346	.341	.338	.337	.336	.338	.336	.336
16	.614	.609	.604	.601	.599	.599	.598	.598	.598
18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.65	1.56	1.67	1.58	1.58	1.59	1.69	1.59	1.69
22	2.32	2.32	2.35	2.38	2.40	2.41	2.41	2.41	2.42
24	3.37	3.34	3.40	3.47	3.51	3.53	3.54	3.56	3.56
26	4.76	4.69	4.77	4.88	4.97	5.02	5.04	5.06	5.06
28	6.58	6.44	6.62	6.70	6.86	6.94	7.00	7.02	7.04
30	8.92	8.68	8.74	8.98	9.23	9.39	9.48	9.54	9.56
32	11.9	11.5	11.5	11.8	12.2	12.4	12.6	12.7	12.7
34	16.5	15.0	14.9	15.3	15.8	16.2	16.4	16.6	16.7
36	20.1	19.3	19.2	19.5	20.1	20.7	21.1	21.4	21.5
38	25.6	24.6	24.3	24.6	25.4	26.1	26.7	27.1	27.4
40	32.2	30.8	30.4	30.7	31.6	32.6	33.4	34.0	34.4
42	40.1	38.4	37.7	38.0	38.9	40.1	41.3	42.1	42.7
44	49.4	47.3	46.4	46.6	47.8	49.0	50.4	61.6	52.4
46	60.4	57.7	66.6	56.7	57.7	59.3	61.1	62.6	63.7
48	73.2	69.9	68.4	68.4	69.4	71.2	73.3	76.3	76.8
50	88.0	84.1	82.2	82.0	83.0	84.9	87.4	89.8	91.7

Table D.11. Axle loed aqulvalency factors for rigid pavemants, tandem exles and p_t of 2.0.

Axle Loed (kips)	Sleb Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0006	.0006	.0006	.0005	.0005	.0006	.0005	.0006	.0006
5	.002	.002	.002	.002	.002	.002	.002	.002	.002
8	.006	.006	.005	.006	.006	.006	.006	.006	.005
10	.014	.013	.013	.012	.012	.012	.012	.012	.012
12	.028	.026	.026	.025	.025	.026	.025	.025	.026
14	.051	.049	.048	.047	.047	.047	.047	.047	.047
16	.087	.084	.082	.081	.081	.080	.080	.080	.080
18	.141	.136	.133	.132	.131	.131	.131	.131	.131
20	.216	.210	.206	.204	.203	.203	.203	.203	.203
22	.319	.313	.307	.306	.304	.303	.303	.303	.303
24	.454	.449	.444	.441	.440	.439	.439	.439	.439
26	.629	.626	.622	.620	.618	.618	.618	.618	.618
28	.852	.851	.850	.860	.850	.849	.849	.849	.849
30	1.13	1.13	1.14	1.14	1.14	1.14	1.14	1.14	1.14
32	1.48	1.48	1.49	1.50	1.51	1.51	1.51	1.61	1.61
34	1.90	1.90	1.93	1.95	1.96	1.97	1.97	1.97	1.97
36	2.42	2.41	2.45	2.49	2.51	2.52	2.53	2.53	2.53
38	3.04	3.02	3.07	3.13	3.17	3.19	3.20	3.20	3.21
40	3.79	3.74	3.80	3.89	3.95	3.98	4.00	4.01	4.01
42	4.67	4.69	4.66	4.78	4.87	4.93	4.95	4.97	4.97
44	6.72	5.59	5.67	6.82	5.95	6.03	6.07	6.09	6.10
46	6.94	6.76	6.83	7.02	7.20	7.31	7.37	7.41	7.43
48	8.36	8.12	8.17	8.40	8.63	8.79	8.88	8.93	8.96
50	10.00	9.69	9.72	9.98	10.27	10.49	10.62	10.69	10.73
52	11.9	11.5	11.5	11.8	12.1	12.4	12.6	12.7	12.8
64	14.0	13.5	13.5	13.8	14.2	14.6	14.9	16.0	16.1
56	16.5	16.9	15.8	16.1	16.6	17.1	17.4	17.6	17.7
58	19.3	18.5	18.4	18.7	19.3	19.8	20.3	20.5	20.7
60	22.4	21.5	21.3	21.6	22.3	22.9	23.5	23.8	24.0
62	26.9	24.9	24.6	24.9	25.5	26.4	27.0	27.5	27.7
64	29.9	28.6	28.2	28.5	29.3	30.2	31.0	31.6	31.9
68	34.3	32.8	32.3	32.6	33.4	34.4	36.4	36.1	36.6
68	39.2	37.6	36.8	37.1	37.9	39.1	40.2	41.1	41.6
70	44.6	42.7	41.9	42.1	42.9	44.2	46.6	48.6	47.3
72	60.6	48.4	47.5	47.6	48.6	49.9	61.4	62.8	63.5
74	67.3	54.7	53.6	63.6	64.6	66.1	57.7	69.2	60.3
76	64.6	61.7	60.4	60.3	61.2	62.8	64.7	66.4	67.7
78	72.5	69.3	67.8	67.7	68.6	70.2	72.3	74.3	75.8
80	81.3	77.6	75.9	76.7	76.6	78.3	80.6	82.8	84.7
82	90.9	86.7	84.7	84.4	85.3	87.1	89.6	92.1	94.2
84	101.	97.	94.	94.	95.	97.	99.	102.	106.
86	113.	107.	105.	104.	106.	107.	110.	113.	116.
88	125.	119.	116.	116.	116.	118.	121.	126.	128.
90	138.	132.	129.	128.	129.	131.	134.	137.	141.

Table D.12. Axle load aquivalancy factora for rigid pavements, triple axlas and p_t of 2.0.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003
6	.0010	.0009	.0009	.0009	.0009	.0009	.0009	.0009	.0009
8	.002	.002	.002	.002	.002	.002	.002	.002	.002
10	.005	.005	.005	.005	.005	.005	.005	.005	.005
12	.010	.010	.009	.009	.009	.009	.009	.009	.009
14	.018	.017	.017	.016	.016	.016	.016	.016	.016
16	.030	.029	.028	.027	.027	.027	.027	.027	.027
18	.047	.045	.044	.044	.043	.043	.043	.043	.043
20	.072	.069	.067	.066	.066	.066	.066	.066	.066
22	.105	.101	.099	.098	.097	.097	.097	.097	.097
24	.149	.144	.141	.139	.139	.138	.138	.138	.138
26	.205	.199	.195	.194	.193	.192	.192	.192	.192
28	.276	.270	.265	.263	.262	.262	.262	.262	.261
30	.364	.359	.354	.351	.350	.349	.349	.349	.349
32	.472	.468	.463	.460	.459	.458	.458	.458	.458
34	.603	.600	.596	.594	.593	.592	.592	.592	.592
36	.759	.758	.757	.756	.756	.755	.755	.756	.755
38	.946	.947	.949	.950	.951	.951	.951	.951	.951
40	1.17	1.17	1.18	1.18	1.18	1.18	1.18	1.18	1.19
42	1.42	1.43	1.44	1.46	1.46	1.46	1.46	1.46	1.46
44	1.73	1.73	1.75	1.77	1.78	1.78	1.79	1.79	1.79
46	2.08	2.07	2.10	2.13	2.15	2.16	2.16	2.16	2.17
48	2.48	2.47	2.51	2.55	2.58	2.59	2.60	2.60	2.61
50	2.95	2.92	2.97	3.03	3.07	3.09	3.10	3.11	3.11
52	3.48	3.44	3.50	3.58	3.63	3.66	3.68	3.69	3.69
54	4.09	4.03	4.09	4.20	4.27	4.31	4.33	4.35	4.35
56	4.78	4.69	4.76	4.89	4.99	5.05	5.08	5.09	5.10
58	5.57	5.44	5.51	5.66	5.79	5.87	5.91	5.94	5.95
60	6.45	6.29	6.35	6.53	6.69	6.79	6.86	6.88	6.90
62	7.43	7.23	7.28	7.49	7.69	7.82	7.90	7.94	7.97
64	8.54	8.28	8.32	8.55	8.80	8.97	9.07	9.13	9.16
66	9.76	9.46	9.48	9.73	10.02	10.24	10.37	10.44	10.48
68	11.1	10.8	10.8	11.0	11.4	11.6	11.8	11.9	12.0
70	12.6	12.2	12.2	12.5	12.8	13.2	13.4	13.5	13.6
72	14.3	13.8	13.7	14.0	14.5	14.9	15.1	15.3	15.4
74	16.1	15.5	15.4	15.7	16.2	16.7	17.0	17.2	17.3
76	18.2	17.5	17.3	17.6	18.2	18.7	19.1	19.3	19.5
78	20.4	19.6	19.4	19.7	20.3	20.9	21.4	21.7	21.8
80	22.8	21.9	21.6	21.9	22.6	23.3	23.8	24.2	24.4
82	25.4	24.4	24.1	24.4	25.0	25.8	26.5	26.9	27.2
84	28.3	27.1	26.7	27.0	27.7	28.6	29.4	29.9	30.2
86	31.4	30.1	29.6	29.9	30.7	31.6	32.5	33.1	33.5
88	34.8	33.3	32.8	33.0	33.8	34.8	35.8	36.6	37.1
90	38.5	36.8	36.2	36.4	37.2	38.3	39.4	40.3	40.9

Teble D.13. Axle loed equivelency fectors for rigid pevements, single axles end p_t of 2.5.

Axle Load (kips)	Sleb Thickness, D (inches)								
	8	7	8	9	10	11	12	13	14
2	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002
4	.003	.002	.002	.002	.002	.002	.002	.002	.002
5	.012	.011	.010	.010	.010	.010	.010	.010	.010
8	.039	.035	.033	.032	.032	.032	.032	.032	.032
10	.097	.089	.084	.082	.081	.080	.080	.080	.080
12	.203	.189	.181	.176	.175	.174	.174	.173	.173
14	.376	.360	.347	.341	.338	.337	.335	.336	.336
16	.634	.623	.610	.604	.501	.699	.599	.599	.598
18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.52	1.55	1.57	1.58	1.58	1.59	1.59	1.59
22	2.21	2.20	2.28	2.34	2.38	2.40	2.41	2.41	2.41
24	3.16	3.10	3.22	3.36	3.45	3.50	3.53	3.54	3.65
25	4.41	4.26	4.42	4.57	4.85	4.95	5.01	5.04	5.05
28	8.06	5.76	6.92	5.29	6.61	5.81	5.92	5.98	7.01
30	8.16	7.57	7.79	8.28	8.79	9.14	9.35	9.45	9.52
32	10.8	10.1	10.1	10.7	11.4	12.0	12.3	12.6	12.7
34	14.1	13.0	12.9	13.5	14.5	15.4	15.0	16.4	16.5
35	18.2	16.7	16.4	17.1	18.3	19.5	20.4	21.0	21.3
38	23.1	21.1	20.5	21.3	22.7	24.3	25.6	26.4	27.0
40	29.1	26.5	25.7	26.3	27.9	29.9	31.6	32.9	33.7
42	36.2	32.9	31.7	32.2	34.0	36.3	38.7	40.4	41.8
44	44.6	40.4	38.8	39.2	41.0	43.8	46.7	49.1	50.8
48	54.5	49.3	47.1	47.3	48.2	52.3	55.9	59.0	51.4
48	85.1	59.7	55.9	56.8	58.7	62.1	66.3	70.3	73.4
50	79.4	71.7	68.2	87.8	89.5	73.3	78.1	83.0	87.1

Table D.14. Axle load equivalency factors for rigid pavements, tandem axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches) 6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0008	.0006	.0005	.0005	.0005	.0005	.0005	.0005	.0005
6	.002	.002	.002	.002	.002	.002	.002	.002	.002
8	.007	.005	.006	.005	.005	.005	.005	.005	.005
10	.015	.014	.013	.013	.012	.012	.012	.012	.012
12	.031	.028	.026	.025	.025	.025	.025	.025	.025
14	.057	.052	.049	.048	.047	.047	.047	.047	.047
16	.097	.089	.084	.082	.081	.081	.080	.080	.080
18	.155	.143	.136	.133	.132	.131	.131	.131	.131
20	.234	.220	.211	.206	.204	.203	.203	.203	.203
22	.340	.325	.313	.308	.305	.304	.303	.303	.303
24	.475	.462	.450	.444	.441	.440	.439	.439	.439
26	.644	.637	.627	.622	.620	.619	.618	.618	.618
28	.855	.854	.852	.850	.850	.850	.849	.849	.849
30	1.11	1.12	1.13	1.14	1.14	1.14	1.14	1.14	1.14
32	1.43	1.44	1.47	1.49	1.50	1.51	1.51	1.51	1.51
34	1.82	1.82	1.87	1.92	1.95	1.96	1.97	1.97	1.97
36	2.29	2.27	2.35	2.43	2.48	2.51	2.52	2.52	2.53
38	2.85	2.80	2.91	3.03	3.12	3.16	3.18	3.20	3.20
40	3.52	3.42	3.55	3.74	3.87	3.94	3.98	4.00	4.01
42	4.32	4.16	4.30	4.55	4.74	4.86	4.91	4.95	4.96
44	5.26	5.01	5.16	5.48	5.75	5.92	6.01	6.06	6.09
46	6.36	6.01	6.14	6.53	6.90	7.14	7.28	7.36	7.40
48	7.64	7.16	7.27	7.73	8.21	8.55	8.75	8.86	8.92
50	9.11	8.50	8.55	9.07	9.58	10.14	10.42	10.58	10.66
52	10.8	10.0	10.0	10.5	11.3	11.9	12.3	12.5	12.7
54	12.8	11.8	11.7	12.3	13.2	13.9	14.5	14.8	14.9
56	15.0	13.8	13.8	14.2	15.2	16.2	16.8	17.3	17.5
58	17.5	16.0	15.7	16.3	17.5	18.6	19.5	20.1	20.4
60	20.3	18.5	18.1	18.7	20.0	21.4	22.5	23.2	23.6
62	23.5	21.4	20.8	21.4	22.8	24.4	25.7	26.7	27.3
64	27.0	24.6	23.8	24.4	25.8	27.7	29.3	30.5	31.3
66	31.0	28.1	27.1	27.6	29.2	31.3	33.2	34.7	35.7
68	36.4	32.1	30.9	31.3	32.9	35.2	37.5	39.3	40.5
70	40.3	36.5	35.0	35.3	37.0	39.5	42.1	44.3	45.9
72	45.7	41.4	39.6	39.8	41.5	44.2	47.2	49.8	51.7
74	51.7	46.7	44.6	44.7	46.4	49.3	52.7	56.7	58.0
76	58.3	52.6	50.2	50.1	51.8	54.9	58.6	62.1	64.8
78	65.5	59.1	56.3	56.1	57.7	60.9	65.0	69.0	72.3
80	73.4	66.2	62.9	62.5	64.2	67.5	71.9	75.4	80.2
82	82.0	73.9	70.2	69.6	71.2	74.7	79.4	84.4	88.8
84	91.4	82.4	78.1	77.3	78.9	82.4	87.4	93.0	98.1
86	102.	92.	87.	86.	87.	91.	96.	102.	108.
88	113.	102.	96.	95.	96.	100.	105.	112.	119.
90	125.	112.	106.	105.	106.	110.	115.	123.	130.

Table D.15. Axle load equivalency factors for rigid pavements, triple axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches) 6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003
6	.001	.001	.001	.001	.001	.001	.001	.001	.001
8	.003	.002	.002	.002	.002	.002	.002	.002	.002
10	.006	.005	.005	.005	.005	.005	.005	.005	.005
12	.011	.010	.010	.009	.009	.009	.009	.009	.009
14	.020	.018	.017	.017	.016	.016	.016	.016	.016
16	.033	.030	.029	.028	.027	.027	.027	.027	.027
18	.053	.048	.045	.044	.044	.043	.043	.043	.043
20	.080	.073	.069	.067	.066	.066	.066	.066	.066
22	.116	.107	.101	.099	.098	.097	.097	.097	.097
24	.163	.151	.144	.141	.139	.139	.138	.138	.138
26	.222	.209	.200	.195	.194	.193	.192	.192	.192
28	.296	.281	.271	.265	.263	.262	.262	.262	.262
30	.384	.371	.359	.354	.351	.350	.349	.349	.349
32	.490	.480	.468	.463	.460	.459	.458	.458	.458
34	.616	.609	.601	.596	.594	.593	.592	.592	.592
36	.765	.762	.759	.757	.756	.755	.755	.755	.755
38	.939	.941	.946	.948	.950	.951	.951	.951	.951
40	1.14	1.15	1.16	1.17	1.18	1.18	1.18	1.18	1.18
42	1.38	1.38	1.41	1.44	1.45	1.46	1.46	1.46	1.46
44	1.65	1.66	1.70	1.74	1.77	1.78	1.78	1.78	1.79
46	1.97	1.96	2.03	2.09	2.13	2.15	2.16	2.16	2.16
48	2.34	2.31	2.40	2.49	2.55	2.58	2.59	2.60	2.60
50	2.76	2.71	2.81	2.94	3.02	3.07	3.09	3.10	3.11
52	3.24	3.15	3.27	3.44	3.56	3.62	3.66	3.68	3.68
54	3.79	3.66	3.79	4.00	4.16	4.26	4.30	4.33	4.34
56	4.41	4.23	4.37	4.63	4.84	4.97	5.03	5.07	5.09
58	5.12	4.87	5.00	5.32	5.59	5.76	5.85	5.90	5.93
60	5.91	5.59	5.71	6.08	6.42	6.64	6.77	6.84	6.87
62	6.80	6.39	6.50	6.91	7.33	7.62	7.79	7.88	7.93
64	7.78	7.29	7.37	7.82	8.33	8.70	8.92	9.04	9.11
66	8.90	8.28	8.33	8.83	9.42	9.88	10.17	10.33	10.42
68	10.1	9.4	9.4	9.9	10.6	11.2	11.5	11.7	11.9
70	11.6	10.6	10.6	11.1	11.9	12.6	13.0	13.3	13.5
72	13.0	12.0	11.8	12.4	13.3	14.1	14.7	15.0	15.2
74	14.6	13.5	13.2	13.8	14.8	15.8	16.5	16.9	17.1
76	16.6	15.1	14.8	15.4	16.5	17.6	18.4	18.9	19.2
78	18.5	16.9	16.5	17.1	18.2	19.6	20.5	21.1	21.5
80	20.6	18.8	18.3	18.9	20.2	21.6	22.7	23.5	24.0
82	23.0	21.0	20.3	20.9	22.2	23.8	25.2	26.1	26.7
84	25.6	23.3	22.5	23.1	24.5	26.2	27.8	28.9	29.6
86	28.4	25.8	24.9	25.4	26.9	28.8	30.5	31.9	32.6
88	31.6	28.6	27.5	27.9	29.4	31.5	33.6	35.1	36.1
90	34.8	31.5	30.3	30.7	32.2	34.4	36.7	38.5	39.8

Table D.16. Axle load equivalency factors for rigid pavements, single axles and p_t of 3.0.

Axle Load (kips)	Slab Thickness, D (inches) 6	7	8	9	10	11	12	13	14
2	.0003	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002
4	.003	.003	.002	.002	.002	.002	.002	.002	.002
6	.014	.012	.011	.010	.010	.010	.010	.010	.010
8	.045	.038	.034	.033	.032	.032	.032	.032	.032
10	.111	.095	.087	.083	.081	.081	.080	.080	.080
12	.228	.202	.186	.179	.176	.174	.174	.174	.173
14	.408	.378	.365	.344	.340	.337	.337	.336	.336
16	.660	.640	.619	.608	.603	.600	.599	.599	.599
18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.46	1.47	1.52	1.55	1.57	1.58	1.58	1.59	1.59
22	2.07	2.06	2.18	2.29	2.35	2.38	2.40	2.41	2.41
24	2.90	2.81	3.00	3.23	3.38	3.47	3.51	3.53	3.54
26	4.00	3.77	4.01	4.40	4.70	4.87	4.96	5.01	5.04
28	5.43	4.99	5.23	5.80	6.31	6.66	6.83	6.93	6.98
30	7.27	6.53	6.72	7.48	8.26	8.83	9.17	9.36	9.48
32	9.59	8.47	8.53	9.42	10.54	11.44	12.03	12.37	12.56
34	12.5	10.9	10.7	11.7	13.2	14.5	15.5	16.0	16.4
36	16.0	13.8	13.4	14.4	16.2	18.1	19.5	20.4	21.0
38	20.4	17.4	16.7	17.7	19.8	22.2	24.2	25.8	26.4
40	25.6	21.8	20.6	21.5	23.8	26.8	29.5	31.5	32.9
42	31.8	26.9	25.3	26.0	28.5	32.0	35.5	38.4	40.3
44	39.2	33.1	30.8	31.3	33.9	37.9	42.3	46.1	48.8
46	47.8	40.3	37.2	37.5	40.1	44.5	49.8	54.7	58.5
48	57.9	48.6	44.8	44.7	47.3	52.1	58.2	64.3	69.4
50	69.6	58.4	53.6	53.1	55.6	60.6	67.6	76.0	81.4

Table D.17. Axla loed equivalency factors for rigid pavaments, tandem exles and p_t of 3.0.

Axle Loed (kips)	Sleb Thicknees, D (Inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0007	.0006	.0005	.0006	.0006	.0005	.0005	.0006	.0005
6	.003	.002	.002	.002	.002	.002	.002	.002	.002
8	.008	.006	.006	.006	.005	.005	.005	.005	.005
10	.018	.015	.013	.013	.013	.012	.012	.012	.012
12	.036	.030	.027	.026	.026	.025	.025	.026	.025
14	.066	.066	.060	.048	.047	.047	.047	.047	.047
16	.111	.095	.087	.083	.081	.081	.081	.080	.080
18	.174	.163	.140	.135	.132	.131	.131	.131	.131
20	.260	.234	.217	.209	.206	.204	.203	.203	.203
22	.368	.341	.321	.311	.307	.305	.304	.303	.303
24	.502	.479	.468	.447	.443	.440	.440	.439	.439
26	.664	.651	.634	.625	.621	.619	.618	.618	.618
28	.869	.857	.853	.861	.860	.850	.860	.849	.849
30	1.09	1.10	1.12	1.13	1.14	1.14	1.14	1.14	1.14
32	1.38	1.38	1.44	1.47	1.49	1.60	1.51	1.61	1.51
34	1.72	1.71	1.80	1.88	1.93	1.95	1.96	1.97	1.97
36	2.13	2.10	2.23	2.36	2.46	2.49	2.61	2.62	2.62
38	2.62	2.54	2.71	2.92	3.06	3.13	3.17	3.19	3.20
40	3.21	3.06	3.26	3.55	3.76	3.89	3.95	3.98	4.00
42	3.90	3.65	3.87	4.26	4.68	4.77	4.87	4.92	4.96
44	4.72	4.35	4.67	5.06	5.50	5.78	6.94	6.02	6.06
46	6.68	6.16	5.36	5.95	6.64	6.94	7.17	7.29	7.36
48	6.80	6.10	6.26	6.93	7.69	8.24	8.57	8.76	8.86
60	8.09	7.17	7.26	8.03	8.96	9.70	10.17	10.43	10.68
62	9.57	8.41	8.40	9.24	10.36	11.32	11.96	12.33	12.64
64	11.3	9.8	9.7	10.6	11.9	13.1	14.0	14.6	14.8
56	13.2	11.4	11.2	12.1	13.6	15.1	16.2	16.9	17.3
68	16.4	13.2	12.8	13.7	15.4	17.2	18.6	19.5	20.1
60	17.9	16.3	14.7	15.6	17.4	19.5	21.3	22.6	23.2
62	20.6	17.6	16.8	17.6	19.6	22.0	24.1	25.7	26.6
64	23.7	20.2	19.1	19.9	22.0	24.7	27.3	29.2	30.4
66	27.2	23.1	21.7	22.4	24.6	27.6	30.6	33.0	34.6
68	31.1	26.3	24.6	25.2	27.4	30.8	34.3	37.1	39.2
70	36.4	29.8	27.8	28.2	30.6	34.2	38.2	41.6	44.1
72	40.1	33.8	31.3	31.6	34.0	37.9	42.3	46.4	49.4
74	46.3	38.1	35.2	36.4	37.7	41.8	46.8	51.5	66.2
76	61.1	42.9	39.5	39.6	41.8	46.1	61.5	56.9	61.3
78	57.4	48.2	44.3	44.0	46.3	60.7	56.6	62.7	67.9
80	64.3	53.9	49.4	48.9	51.1	55.8	62.1	68.9	74.9
82	71.8	60.2	55.1	64.3	66.5	61.2	67.9	76.6	82.4
84	80.0	67.0	61.2	60.2	62.2	67.0	74.2	82.4	90.3
86	89.0	74.5	67.9	66.5	68.5	73.4	80.8	89.8	98.7
88	98.7	82.5	75.2	73.5	75.3	80.2	88.0	97.7	107.6
90	109.	91.	83.	81.	83.	88.	98.	106.	117.

Table D.18. Axle load equivalency factors for rigid pavements, triple axles and p_t of 3.0.

Axle Load (kips)	Slab Thickness, D (inches) 6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0004	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003
6	.001	.001	.001	.001	.001	.001	.001	.001	.001
8	.003	.003	.002	.002	.002	.002	.002	.002	.002
10	.007	.006	.005	.005	.005	.005	.005	.005	.005
12	.013	.011	.010	.009	.009	.009	.009	.009	.009
14	.023	.020	.018	.017	.017	.016	.016	.016	.016
16	.039	.033	.030	.028	.028	.027	.027	.027	.027
18	.061	.052	.047	.045	.044	.044	.043	.043	.043
20	.091	.078	.071	.088	.067	.068	.066	.066	.066
22	.132	.114	.104	.100	.098	.097	.097	.097	.097
24	.183	.161	.148	.143	.140	.139	.139	.138	.138
26	.245	.221	.205	.198	.195	.193	.193	.182	.192
28	.322	.295	.277	.258	.265	.253	.262	.252	.282
30	.411	.387	.357	.357	.363	.351	.350	.349	.349
32	.515	.495	.476	.466	.462	.460	.459	.458	.458
34	.634	.622	.607	.599	.595	.594	.593	.592	.592
36	.772	.768	.762	.758	.765	.758	.765	.755	.755
38	.930	.934	.942	.947	.949	.950	.951	.951	.951
40	1.11	1.12	1.15	1.17	1.18	1.18	1.18	1.18	1.18
42	1.32	1.33	1.38	1.42	1.44	1.46	1.46	1.45	1.45
44	1.56	1.56	1.64	1.71	1.75	1.77	1.78	1.78	1.78
46	1.84	1.83	1.94	2.04	2.10	2.14	2.15	2.16	2.16
48	2.16	2.12	2.26	2.41	2.51	2.56	2.58	2.59	2.60
50	2.53	2.45	2.61	2.82	2.96	3.03	3.07	3.09	3.10
52	2.95	2.82	3.01	3.27	3.47	3.58	3.53	3.68	3.68
54	3.43	3.23	3.43	3.77	4.03	4.18	4.27	4.31	4.33
56	3.98	3.70	3.90	4.31	4.55	4.85	4.98	5.04	5.07
58	4.59	4.22	4.42	4.90	5.34	5.62	5.78	5.86	5.90
60	5.28	4.80	4.99	5.54	6.08	5.45	8.56	6.78	5.84
62	6.06	5.46	5.61	6.23	6.89	7.36	7.54	7.80	7.88
64	6.92	6.18	5.29	5.98	7.76	8.38	8.72	8.93	9.04
66	7.89	6.98	7.05	7.78	8.70	9.44	9.91	10.18	10.33
68	8.96	7.88	7.87	8.66	9.71	10.61	11.20	11.55	11.76
70	10.2	8.9	8.8	9.6	10.8	11.9	12.6	13.1	13.3
72	11.6	10.0	9.8	10.6	12.0	13.2	14.1	14.7	15.0
74	12.9	11.2	10.9	11.7	13.2	14.7	15.8	16.5	16.9
76	14.5	12.5	12.1	12.9	14.5	16.2	17.5	18.4	18.9
78	16.2	13.9	13.4	14.2	15.9	17.8	19.4	20.5	21.1
80	18.2	15.5	14.8	15.6	17.4	19.6	21.4	22.7	23.5
82	20.2	17.2	16.4	17.2	19.1	21.4	23.5	25.1	26.1
84	22.5	19.1	18.1	18.8	20.8	23.4	25.8	27.8	28.8
86	25.0	21.2	19.9	20.6	22.6	25.5	28.2	30.4	31.8
88	27.6	23.4	21.9	22.5	24.6	27.7	30.7	33.2	35.0
90	30.5	25.8	24.1	24.6	26.8	30.0	33.4	36.3	38.3

DESIGN OF RIGID PAVEMENTS

Reinforcing Steel

Temperature Steel

Reinforcement / Contraction Joint: Design

- ✪ Bar mat or wire mesh: longitudinal and transverse steel wires welded at regular intervals
- ✪ Placed about 7 cm below the slab surface

Calculations:

$F = f \times W = f \times L/2 \times (1) \times h \times \gamma$

$F = p_c (1) \times h + p_s \times A_s$

$P_s = p_c \times E_s/E_c$

$n = E_s/E_c$

$F = p_c (h + nA_s)$

$f \times L/2 \times (1) \times h \times \gamma = p_c (h + nA_s)$

$$L = \frac{2 \times p_c (h + nAs)}{f \times h \times \gamma}$$ **L < 15 m**

f = coefficient of friction between the bottom of concrete pavement and subgrade

h = thickness of concrete pavement (m)

L = length of pavement between *contraction* joints (m)

γ = density of concrete (KN/m^3)

pc = maximum desired stress in the concrete in (kn/m^2)

If L > 15 m then the follwing must be considered:

1. Cross-sectional area of *longitudinal* steel should be at least equal to 0.1 % of the cross-sectional area of the slab
2. *Longitudinal* wires ≥ No. 2 gauge (ϕ 8), spaced at a maximum distance of 15 cm
3. Transverse wires ≥ No. 4 gauge (ϕ 12), spaced at a maximum distance of 30 cm

- Temperature steel does not prevent cracking of the slab, but it does control the crack widths because the steel acts as a tie holding the edges of the cracks together.
- This helps to maintain the shearing resistance of the pavement, thereby maintaining its capacity to carry traffic load, even though the flexural strength is not improved.

Dowel Bars

- ✪ Load-transfer mechanisms across joints
- ❑ Diameters = 25 – 37 mm
- ❑ Lengths = 0.6 – 1 m
- ❑ Spacing = 0.3 m c/c across the width of the slab
- ✪ At least <u>one end</u> of the bar should be *smooth* and *lubricated* to provide <u>free</u> expansion

Tie Bars

- Tie bars are used to tie two sections of the pavement together
- Deformed <u>or</u> contain hooks to facilitate the bonding of the two sections
- ❑ Diameter 19 mm
- ❑ Spacing 1 m

Expansion Joints

- Prevent buckling or "blow up"
- ❑ Across the full width of the slab
- ❑ Full depth
- ❑ Width = 19-25 mm wide
- ❖ Filled with a compressible filler material that permits the slab to expand
- ❖ Filler materials can be cork, rubber, bituminous materials, or bituminous fabrics
- ❖ The upper part must be sealed with suitable sealant
- ✪ The load-transfer mechanism is a <u>smooth dowel bar</u> that is *lubricated* <u>on one side</u>

Contraction Joints

- ✪ Placed transversely
- ✪ Install a load-transfer mechanism in the form of a dowel bar when there is doubt about the ability of the interlocking gains to transfer the load

Hinge Joints (Keyed) (Warping)

- ✪ Reduce cracking along the center line of highway pavements
- ✪ Permits movement due to difference in temperature between the surface and base during day/night

Construction Joints

- ✪ Placed transversely across the pavement width to provide suitable transition between concrete *laid* at <u>different times</u>
- ✪ An expansion joint can be used in lieu of a transverse construction joint

TYPES OF RIGID HIGHWAY PAVEMENTS

1. **Jointed *Plain* Concrete Pavement (JPCP)**

No temperature steel or dowels for load transfer

Steel tie bars often are used to provide a hinge effect at longitudinal joints and to prevent the opening of these joints

Low-volume highways

Joints are placed at relatively short distances (3 – 6 m)

Skewed → only one wheel of a vehicle passes through the joint at a time → smoother ride

2. **Simply Reinforced Concrete Pavement**

Have dowels for the transfer of traffic loads across joints,

Spaced at 10 – 30 m

Temperature steel is used

Tie bars also are used commonly at longitudinal joints

3. **Continuously Reinforced Concrete Pavement (CRCP)**

No transverse joints, except construction joints or expansion joints

High percentage of steel

Minimum = 0.6 % of the cross section of the slab

Contain tie bars across the longitudinal joints

4. ~~**Pre-stressed Concrete Pavements**~~

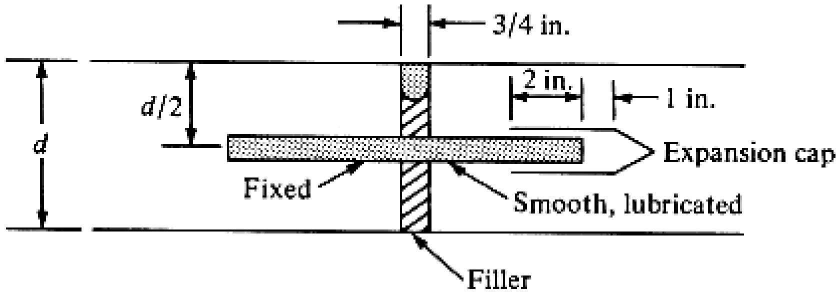


Figure 20.1 Typical Expansion Joint

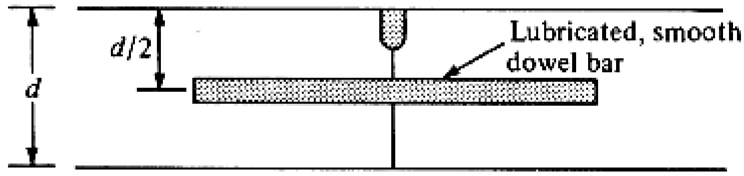


Figure 20.2 Typical Contraction Joint

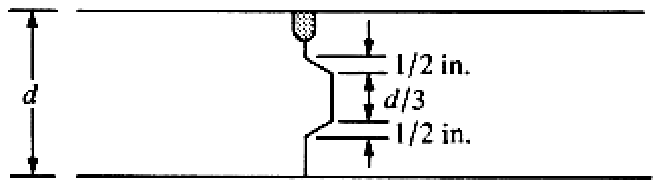


Figure 20.3 Typical Hinge Joint (Keyed Joint)

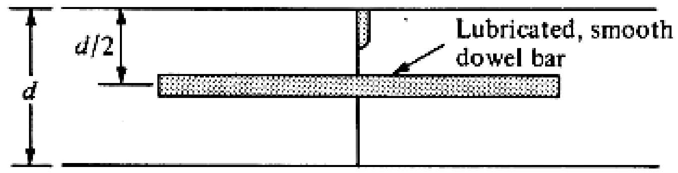


Figure 20.4 Typical Butt Joint

PUMPING OF RIGID PAVEMENTS

Discharge of water and subgrade (or subbase) material through joints, cracks, and along the pavement edges.

It primarily is caused by the repeated deflection of the pavement slab in the presence of accumulated water beneath it.

fine-grained → form a slurry

Pumping → faulting of the joints → formation of transverse cracks or the breaking of the corners of the slab.

Joint faulting and cracking is therefore progressive, since formation of a crack facilitates the pumping action.

<u>Visual manifestations of pumping include:</u>

Discharge of water from cracks and joints

Spalling near the centerline of the pavement and a transverse crack or joint

Mud boils at the edge of the pavement

Pavement surface *discoloration* (caused by the subgrade soil)

Breaking of pavement at the *corners*

Preventing Pumping

reduction or elimination of expansion joints,

replace soils with granular or sandy soils (7 to 15 cm) or to improve them by stabilization

THICKNESS DESIGN OF RIGID PAVEMENTS (AASHTO Design Method)

<u>**Design Considerations**</u>

Pavement Performance*:* *Pi* = 4.5 p_t = select

Subbase Strength

- ⊕ Table 20.4

<u>AASHTO suggests that:</u>

- ❖ A through E can be used *within* the upper 4 in (10 cm) layer of the subbase
- ❖ Type F can be used *below* the uppermost 4 in (10 cm) layer

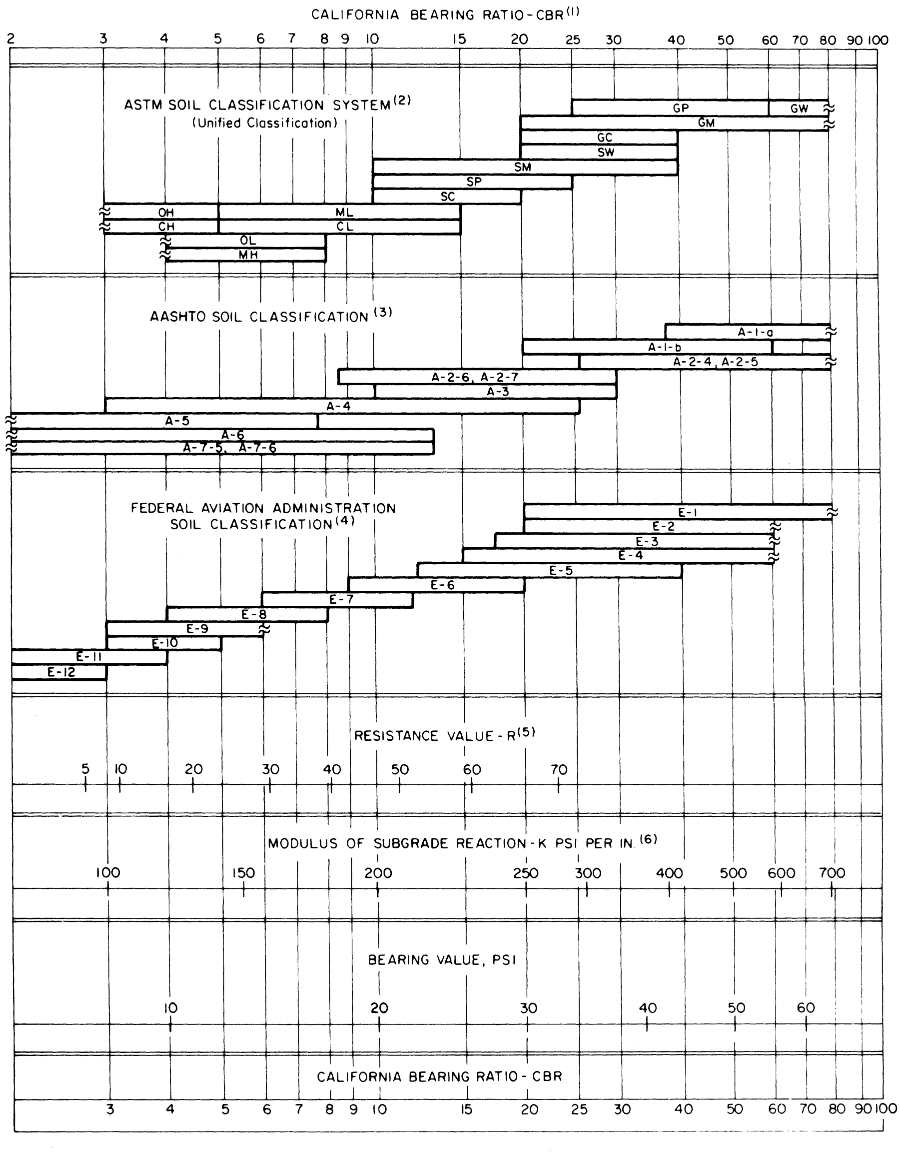
- ⊕ When A, B, and F are used & subjected to *frost action* → *minimize* % of *fines*
- ⊕ Thickness ≥ 6 in 10 cm) and should be extended 1 to 3 ft outside the edge of pavement

Subgrade Strength

Modulus of subgrade reaction *k*, which is defined as the load in lb/in^2 on a loaded area, divided by the deformation in inches.

Values of *k* can be obtained by conducting a plate-bearing test in accordance with the using a 30 in 75 cm) diameter plate.

- ⊕ Figure 20.8



(1) For the basic idea, see O. J. Porter, "Foundations for Flexible Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, pages 100-136.
(2) ASTM Designation D2487.
(3) "Classification of Highway Subgrade Materials," Highway Research Board *Proceedings of the Twenty-fifth Annual Meeting*, 1945, Vol. 25, pages 376-392.
(4) *Airport Paving*, U.S. Department of Commerce, Federal Aviation Agency, May 1948, pages 11-16. Estimated using values given in FAA *Design Manual for Airport Pavements*.(Formerly used FAA Classification; Unified Classification now used.)
(5) C. E.Warnes, "Correlation Between *R* Value and *k* Value," unpublished report, Portland Cement Association, Rocky Mountain-Northwest Region, October 1971 (best-fit correlation with correction for saturation).
(6) See T. A. Middlebrooks and G. E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, page 152.

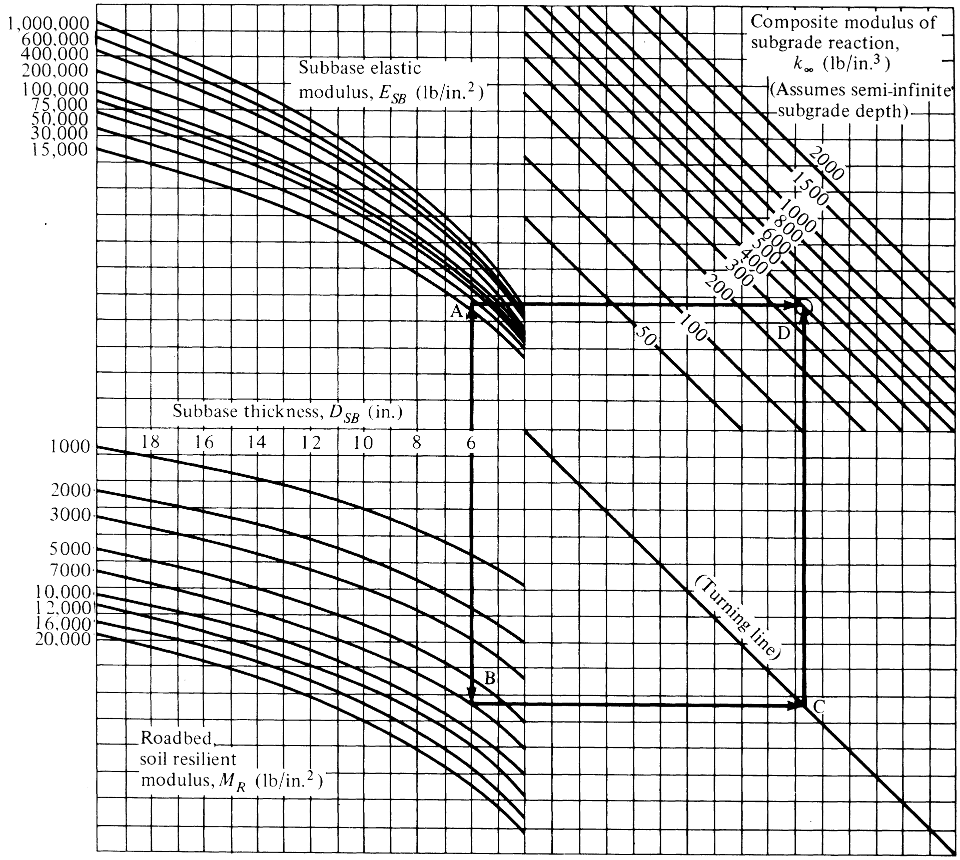
Figure 20.8 Approximate Interrelationship of Soil Classification and Bearing Values

SOURCE: Robert G. Packard, *Thickness Design for Concrete Highway and Street Pavements,* American Concrete Pavement Association, Skokie, IL, 1984.

Table 20.4 Recommended Particle Size Distributions for Different Types of Subbase Materials

	Types of Subbase					
Sieve Designation	*Type A*	*Type B*	*Type C (Cement Treated)*	*Type D (Lime Treated)*	*Type E (Bituminous Treated)*	*Type F (Granular)*
Sieve analysis percent passing						
2 in.	100	100	—	—	—	—
1 in.	—	75–95	100	100	100	100
⅜ in.	30–65	40–75	50–85	60–100	—	—
No. 4	25–55	30–60	35–65	50–85	55–100	70–100
No. 10	15–40	20–45	25–50	40–70	40–100	55–100
No. 40	8–20	15–30	15–30	25–45	20–50	30–70
No. 200	2–8	5–20	5–15	5–20	6–20	8–25
(The minus No. 200 material should be held to a practical minimum.)						
Compressive strength lb/in^2 at 28 days			400–750	100		
Stability						
Hveem Stabilometer					20 min	
Hubbard field					1000 min	
Marshall stability					500 min	
Marshall flow					20 max	
Soil constants						
Liquid limit	25 max	25 max				25 max
Plasticity index[a]	N.P.	6 max	10 max[b]		6 max[b]	6 max

[a]As performed on samples prepared in accordance with AASHTO Designation T87.
[b]These values apply to the mineral aggregate prior to mixing with the stabilizing agent.
SOURCE: Adapted with permission from *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, American Association of State Highway and Transportation Officials, Washington, D.C., 2007.



Example:

D_{SB} = 6 in.

E_{SB} = 20,000 lb/in.^{2}

M_R = 7,000 lb/in.^{2}

Solution: k_∞ = 400 lb/in.^{3}

Figure 20.9 Chart for Estimating Composite Modulus of Subgrade Reaction, K_∞, Assuming a Semi-Infinite Subgrade Depth*

*For practical purposes, a semi-infinite depth is considered to be greater than 10 ft below the surface of the subgrade.

SOURCE: Redrawn from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

Traffic (Discussed)

Concrete Properties

Flexural strength (modulus of rupture) at 28 days

$E_c = 5700\sqrt{f_c'}$

Drainage (*Cd*) : Table 20.9

Reliability

$$\log_{10} W_{18} = Z_R S_o + 7.35\log_{10}(D+1) - 0.06 + \frac{\log_{10}[\Delta PSI/(4.5-1.5)]}{1+[(1.624\times 10^7)/(D+1)^{8.46}]} + (4.22 - 0.32P_t)\log_{10}\left\{\frac{S_c' C_d}{215.63J}\left(\frac{D^{.75}-1.132}{D^{.75}-[18.42/(E_c/k)^{.25}]}\right)\right\} \quad (20.21)$$

where

Z_R = standard normal variant corresponding to the selected level of reliability

S_o = overall standard deviation (see Chapter 19)

W_{18} = predicted number of 18 kip ESAL applications that can be carried by the pavement structure after construction

D = thickness of concrete pavement to the nearest half-inch

ΔPSI = design serviceability loss = $p_i - p_t$

p_i = initial serviceability index

p_t = terminal serviceability index

E_c = elastic modulus of the concrete to be used in construction (lb/in^2)

S_c' = modulus of rupture of the concrete to be used in construction (lb/in^2)

J = load transfer coefficient = 3.2 (assumed)

C_d = drainage coefficient

Pavement Type (no tied shoulders)	*J*
JCP/JRCP w/ load transfer devices	3.2
JCP/JRCP w/out load transfer devices	3.8-4.4
CRCP	2.9

Table 2.6. Recommended Load Transfer Coefficient for Various Pavement Types and Design Conditions

Shoulder	Asphalt		Tied P.C.C.	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				
1. Plain jointed and jointed reinforced	3.2	3.8–4.4	2.5–3.1	3.6–4.2
2. CRCP	2.9–3.2	N/A	2.3–2.9	N/A

Table 20.9 Recommended Values for Drainage Coefficient, C_d, for Rigid Pavements

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	*Less Than 1%*	*1–5%*	*5–25%*	*Greater Than 25%*
Excellent	1.2–1.20	1.20–1.15	1.15–1.10	1.10
Good	1.20–1.15	1.15–1.10	1.10–1.00	1.00
Fair	1.15–1.10	1.10–1.00	1.00–0.90	0.90
Poor	1.10–1.00	1.00–0.90	0.90–0.80	0.80
Very poor	1.00–0.90	0.90–0.80	0.80–0.70	0.70

SOURCE: Adapted from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

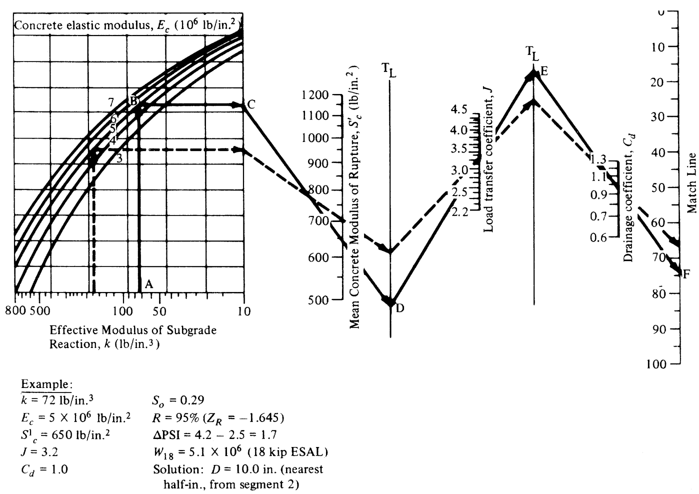


Figure 20.13 Design Chart for Rigid Pavements Based on Using Values for Each Input Variable (Segment 1)

SOURCE: Redrawn from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

Example 20.3 Designing a Rigid Pavement Using the AASHTO Method

The use of the charts is demonstrated with the example given in Figure 20.13. In this case, input values for Segment 1 of the chart (Figure 20.13) are

Effective modulus of subgrade reaction, $k = 72 \text{ lb/in}^3$
Mean concrete modulus of rupture, $S'_c = 650 \text{ lb/in}^2$
Load transfer coefficient, $J = 3.2$
Drainage coefficient, $C_d = 1.0$

These values are used to determine a value on the match line as shown in Figure 20.13 (solid line ABCDEF). Input parameters for Segment 2 (Figure 20.14) of the chart are

Match line value determined in segment 1 (74)
Design serviceability loss, $\Delta PSI = 4.5 - 2.5 = 2.0$
Reliability, $R\% = 95\%$ ($Z_R = 1.645$)
Overall standard deviation, $S_o = 0.29$
Cumulative 18 kip ESAL = (5×10^6)

Solution: The required thickness of the concrete slab is then obtained, as shown in Figure 20.14, as 10 in. (nearest half-inch).

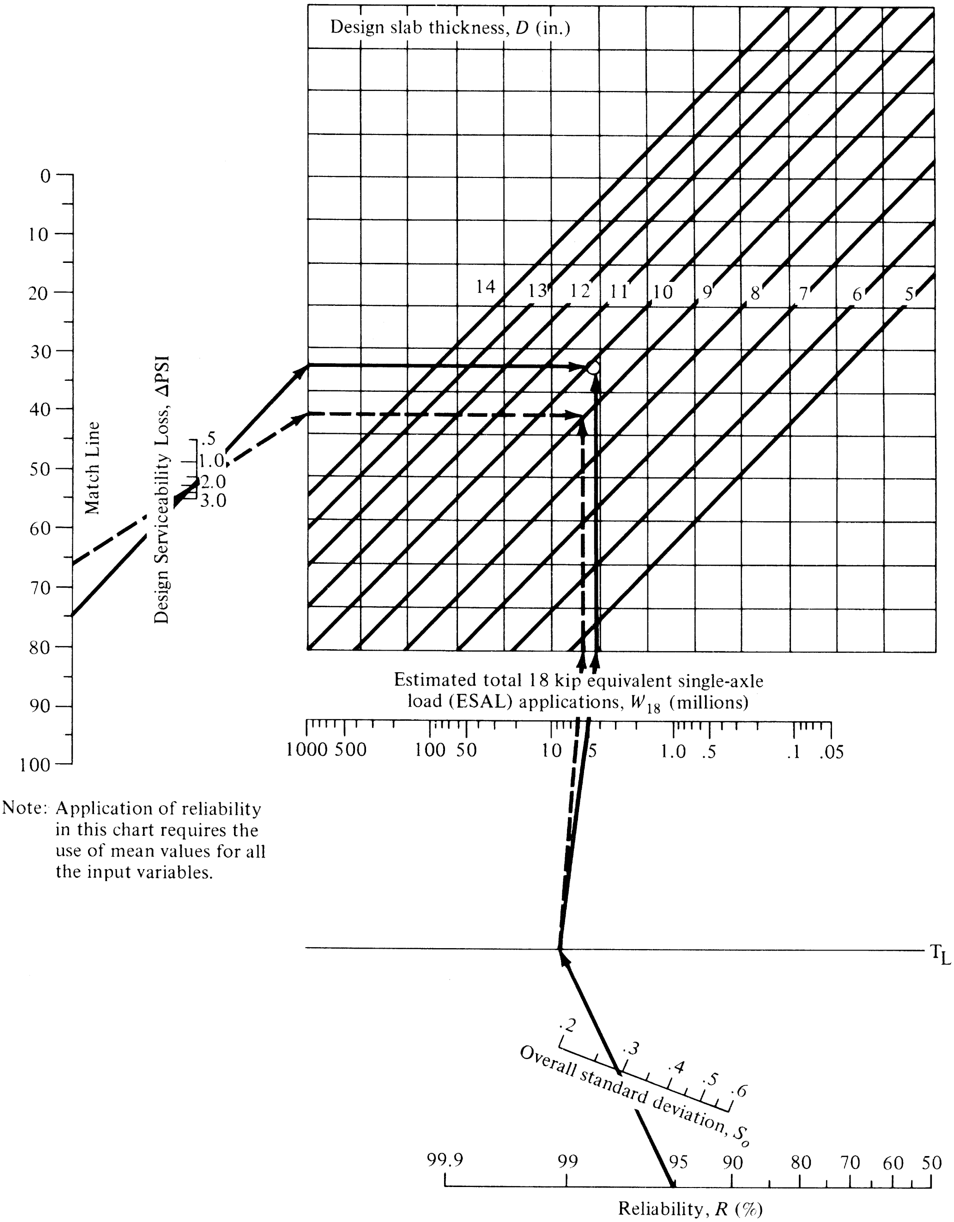
Note that when the thickness obtained from solving Eq. 20.21 analytically or by use of Figures 20.13 and 20.14 is significantly different from that originally assumed to determine the effective subgrade modulus and to select the ESAL factors, the whole procedure has to be repeated until the assumed and designed values are approximately the same, emphasizing the importance of using a computer program to facilitate the necessary iteration.

Example 20.4 Evaluating the Adequacy of a Rigid Pavement Using the AASHTO Method

Using the data and effective subgrade modulus obtained in Example 20.2, determine whether the 9 in. pavement design of Example 20.2 will be adequate on a rural expressway for a 20-year analysis period and the following design criteria

$P_i = 4.5$
$P_t = 2.5$
ESAL on design lane during first year of operation = 0.2×10^6
Traffic growth rate = 4%
Concrete elastic modulus, $E_c = 5 \times 10^6$ lb/in^2
Mean concrete modulus of rupture = 700 lb/in^2
Drainage conditions are such that $C_d = 1.0$
$R = 0.95$ ($Z_R = 1.645$)
$S_o = 0.30$ (for rigid pavements $S_o = 0.3 - 0.4$)
Growth factor = 29.78 (from Table 19.6)
$k = 170$ (from Example 20.2)
Assume $D = 9$ in. (from Example 20.2)
ESAL over design period = $0.2 \times 10^6 = 29.78 = 6 \times 10^6$

Solution: The depth of concrete required is obtained from Figures 20.13 and 20.14. The dashed lines represent the solution, and a depth of 9 in. is obtained. The pavement is therefore adequate.



Note: Application of reliability in this chart requires the use of mean values for all the input variables.

Figure 20.14 Design Chart for Rigid Pavements Based on Using Mean Values for Each Input Variable (Segment 2)

SOURCE: Redrawn from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

Hot-Mix, Cold-Laid Asphalt Mixture

[AC (200-300)] + [MC-30 : 0.75%] + wetting agent + Water (2-3%) + 77 ^{o}C + mixing

Suitable for:

- small jobs
- patching high-type pavements

Cold-Mix, Cold-Laid Asphalt Mixture

Using:

- Emulsified asphalts
- Low-viscosity cutback asphalts

Can be used: *immediately* or *later*

✪ MS-1 and MS-2: pavement bases and surfaces for open graded aggregates

✪ CS-1 and CSH-1 : well graded aggregates

Seal Coats

Single applications of asphalt material that may or may **NOT** contain aggregates

✪ Used in maintenance

Types:

- fog seals
- slurry seals
- aggregate seals

Fog Seal

Thin application of emulsified asphalt, usually with **NO** aggregates

✪ SS-1, SS-1H, CSS-1, and CSS-1H

✪ sprayed at a rate of 0.1 to 0.2 gal/yd^2

Used to:

- Reduce the infiltration of air and water into the pavement
- Prevent the progressive separation of aggregate particles from the surface (raveling)
- Bring the surface of the pavement to its original state

Slurry Seal

- ✪ SS-1+ fine aggregate + mineral filler+ water
- ✪ average thickness = 1.5 - 3 mm

Used as:

- ✓ low-cost maintenance material for pavements carrying light traffic
- ✓ fill cracks ≥ 6 mm
- ✓ provide a fine-textured surface
  - ☒ existing cracks will appear through the slurry seal in a short time

Aggregate Seals

Spraying asphalt → immediately covering it with aggregates → rolling the aggregates into the asphalt

- ✪ Use: the softer grades of paving asphalt and the heavier grades of liquid asphalts
- ✪ Purpose: Aggregate seals can be used to restore the surface of old pavements

Prime Coats

Obtained by spraying asphalt binder materials onto non-asphalt base courses.

Used to:

- Provide a waterproof surface on the base
- Fill capillary voids in the base
- Facilitate the bonding of loose mineral particles
- Facilitate the adhesion of the surface treatment to the base
- ✪ MC-30 : dense flexible base
- ✪ MC-70 : more granular-type base materials

Rate:

- MC-30: 0.2 - 0.35 gal/yd^2
- MC-70: 0.3 - 0.6 gal/yd^2
- ❖ Within 24 hours

Tack Coats

Thin layer of asphalt material sprayed over an old pavement to facilitate the bonding of the old pavement and a new course which is to be placed over the old pavement.

- RC-70
- SS-1, SS-1H, CSS-1, and CSS-1H + equal amount of water

<u>Rate:</u>

0.05 - 0.15 gal/yd^2

- Hours in hot weather
- 24 hrs in cooler weather

Surface Treatments

Asphalt material and suitable aggregates on a properly constructed flexible base course to provide a suitable wearing surface for traffic.

Used to:

- Protect the base course
- Eliminate the problem of dust on the wearing surface

<u>Single course</u>: thicknesses = 12 – 19 mm

- Asphalt = 0.13 - 0.42 gal/yd^2 depending on the gradation of the aggregates
- Aggregates = 0.11 to 0.50 ft^3/yd^2

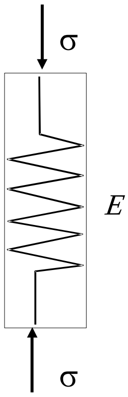
<u>Multiple course</u>: thicknesses = 22 - to 50 mm

Rheological Models

One-dimensional linear viscoelastic models for bitumen

Mechanical (rheological) models

viscoelastic = "viscous" + "elastic"
a bit like a fluid and a bit like a solid



The Linear Elastic Spring

$$\varepsilon = \frac{1}{E}\sigma$$

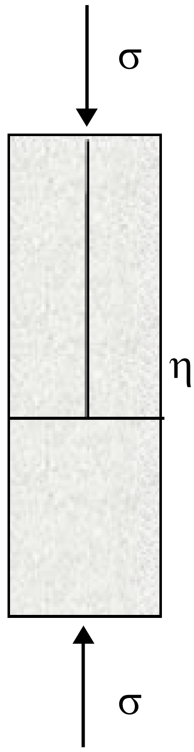
E= Spring stiffness

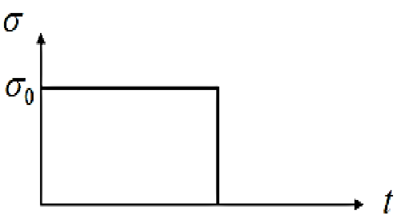
The Linear Viscous Dash-pot

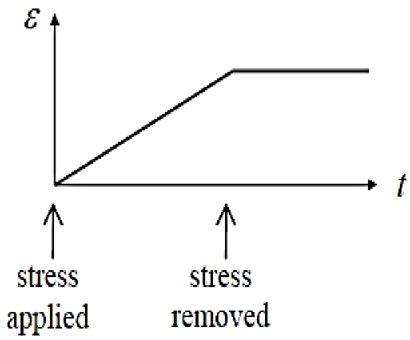
$$\dot{\varepsilon} = \frac{1}{\eta}\sigma$$

η = the **viscosity** of the material

$$\varepsilon = \frac{\sigma_o}{\eta} t$$

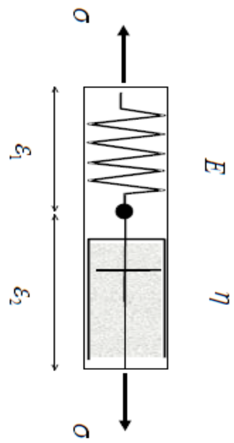






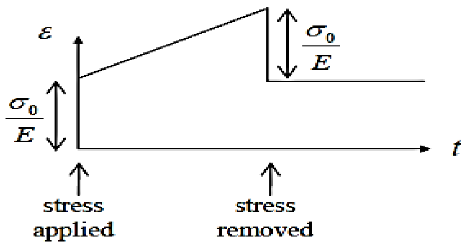
The Maxwell Model

$$\varepsilon_1 = \frac{1}{E}\sigma, \qquad \dot{\varepsilon}_2 = \frac{1}{\eta}\sigma, \qquad \varepsilon = \varepsilon_1 + \varepsilon_2$$



$$\boxed{\sigma + \frac{\eta}{E}\dot{\sigma} = \eta\dot{\varepsilon}} \qquad \textbf{Maxwell Model}$$

$$\varepsilon(t) = \sigma_o\left(\frac{1}{\eta}t + \frac{1}{E}\right)$$



The Kelvin Model

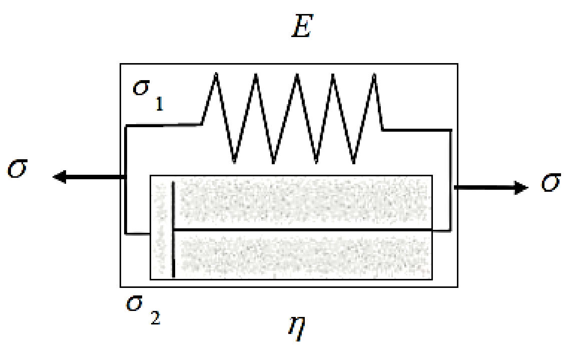
$$\varepsilon = \frac{1}{E}\sigma_1, \quad \dot{\varepsilon} = \frac{1}{\eta}\sigma_2, \quad \sigma = \sigma_1 + \sigma_2 \qquad ($$

where σ_1 is the stress in the spring and σ_2 is the dash-pot stress. Eliminating σ_1 leaves the constitutive law

$$\boxed{\sigma = E\varepsilon + \eta\dot{\varepsilon}} \qquad \textbf{Kelvin (Voigt) Model} \qquad ($$

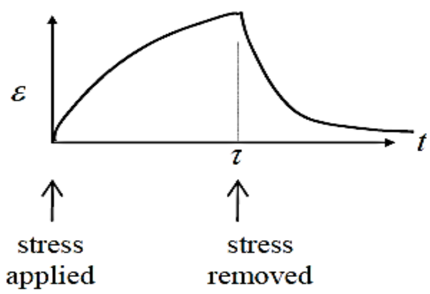
Loading

$$\varepsilon(t) = \frac{\sigma_o}{E}\left(1 - e^{-(E/\eta)t}\right)$$



Unloading at time = τ

$$\varepsilon(t) = \frac{\sigma_o}{E}e^{-(E/\eta)t}\left(e^{(E/\eta)\tau} - 1\right), \quad t > \tau$$



Three – Element Models

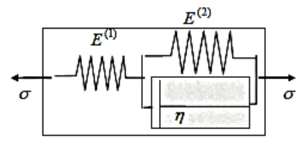
(a) $$\sigma + \frac{\eta}{E_1 + E_2}\dot{\sigma} = \frac{E_1 E_2}{E_1 + E_2}\varepsilon + \frac{\eta E_1}{E_1 + E_2}\dot{\varepsilon}$$

(b) $$\sigma + \frac{\eta}{E_2}\dot{\sigma} = E_1 \varepsilon + \frac{\eta(E_1 + E_2)}{E_2}\dot{\varepsilon}$$

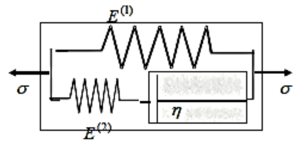
(c) $$\sigma + \frac{\eta_2}{E}\dot{\sigma} = (\eta_1 + \eta_2)\dot{\varepsilon} + \frac{\eta_1 \eta_2}{E}\ddot{\varepsilon}$$

(d) $$\sigma + \frac{\eta_1 + \eta_2}{E}\dot{\sigma} = \eta_1 \dot{\varepsilon} + \frac{\eta_1 \eta_2}{E}\ddot{\varepsilon}$$

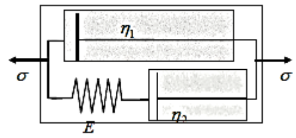
(a)



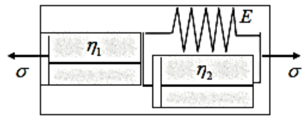
(b)



(c)

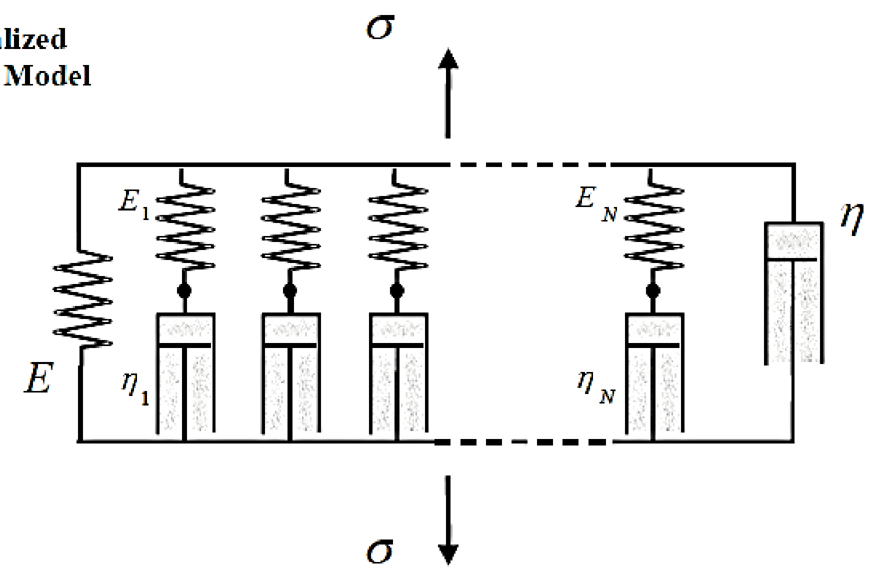


(d)



Generalized Models

Generalized Maxwell Model



Generalized Kelvin Chain