E.7 AASHTO 剛性鋪面厚度設計法(1993 年版)

12.3 AASHTO METHOD

The design guide for rigid pavements was developed at the same time as that for flexible pavements and was published in the same manual. The design is based on the empirical equations obtained from the AASHO Road Test with further modifications based on theory and experience. In this section, only the thickness design is presented. The design of steel reinforcements and tiebars is similar to that discussed in Section 4.3.2 and is not presented here.

12.3.1 Design Equations

The basic equations developed from the AASHO Road Test for rigid pavements are in the same form as those for flexible pavements but with different values for the regression constants. The equations were then modified to include many variables originally not considered in the AASHO Road Test.

Original Equations

Similar to flexible pavements, the regression equations are

$$G_t = \beta(\log W_t - \log \rho) \tag{11.29}$$

$$\beta = 100 + \frac{3.63(L_1 + L_2)^{5.20}}{(D + 1)^{8.46} L_2^{3.52}} \tag{12.10}$$

$$\log \rho = 5.85 + 7.35 \log(D + 1) - 4.62 \log (L_1 + L_2) + 3.28 \log L_2 \tag{12.11}$$

in which $G_t = \log [(4.5 - p_t)/(4.5 - 1.5)]$, where 4.5 is the initial serviceability for rigid pavement at the AASHO Road Test, which is different from the 4.2 for flexible pavements, and p_t is the serviceability at time t;

D = slab thickness in inches, which replaces SN for flexible pavements.

Using an equivalent 18-kip (80-kN) single-axle load with $L_1 = 18$ and $L_2 = 1$ and combining Eqs. 11.29, 12.10, and 12.11 yields

$$\log W_{t18} = 7.35 \log(D + 1) - 0.06 + \frac{\log[(4.5 - p_t)/(4.5 - 1.5)]}{1 + 1.624 \times 10^7/(D + 1)^{8.46}} \tag{12.12}$$

in which W_{t18} is the number of 18-kip (80-kN) single-axle load applications to time t. Equation 12.12 is applicable only to the pavements in the AASHO Road Test with the following conditions: modulus of elasticity of concrete $E_c = 4.2 \times 10^6$ psi (29 GPa), modulus of rupture of concrete $S_c = 690$ psi (4.8 MPa), modulus of subgrade reaction $k = 60$ pci (16 MN/m³), load transfer coefficient $J = 3.2$, and drainage coefficient $C_d = 1.0$.

Modified Equations

To account for conditions other than those that existed in the road test, it is necessary to modify Eq. 12.12 using experience and theory. After comparing stresses calculated from strain measurements on the Road Test pavements with theoretical solutions, the Spangler equation for corner loading (Spangler, 1942) was selected for its simplicity by AASHTO (1972) to extend Eq. 12.12 to other conditions. The Spangler equation is given as

$$\sigma = \frac{JP}{D^2}\left(1 - \frac{a_1}{\ell}\right) \tag{12.13}$$

in which σ is the maximum tensile stress in concrete in psi, J is the load transfer coefficient, P is the wheel load in lb, a_1 is the distance from corner of slab to center of load, and ℓ is the radius of relative stiffness defined by Eq. 4.10 and rewritten as

$$\ell = \left[\frac{ZD^3}{12(1 - \nu^2)}\right]^{0.25} \tag{12.14}$$

in which $Z = E_c/k$ and ν is Poisson ratio of concrete. Assuming that $a_1 = 10$ in. (254 mm) and $\nu = 0.2$, and substituting Eq. 12.14 into 12.13 gives

$$\sigma = \frac{JP}{D^2}\left(1 - \frac{18.42}{Z^{0.25}D^{0.75}}\right) \tag{12.15}$$

Stresses were calculated for different combinations of Road Test variables using Eq. 12.15. The ratio between the calculated stresses and the modulus of rupture, σ/S_c, was subsequently compared with axle load applications. These comparisons indicated that for any given load and terminal serviceability level p_t, the following relationship similar to the general fatigue equation exists:

$$\log W_t = a - (4.22 - 0.32p_t)\log\frac{\sigma}{S_c} \tag{12.16}$$

Assuming the same form of equation for other pavements with W_t', σ', and S_c' yields

$$\log W_t' = a - (4.22 - 0.32p_t)\log\frac{\sigma'}{S_c'} \tag{12.17}$$

Combining Eqs. 12.16 and 12.17 and using the equivalent 18-kip (80-kN) single-axle load gives

$$\log W_{t18}' = \log W_{t18} + (4.22 - 0.32p_t)\log\left(\frac{S_c'}{S_c}\frac{\sigma}{\sigma'}\right) \tag{12.18}$$

From Eq. 12.15

$$\frac{\sigma}{\sigma'} = \frac{J[1 - 18.42/(Z^{0.25}D^{0.75})]}{J'[1 - 18.42/(Z'^{0.25}D^{0.75})]} \tag{12.19}$$

Combining Eqs. 12.12, 12.18, and 12.19 results in

$$\log W_{t18}' = 7.35\log(D + 1) - 0.06 + \frac{\log[(4.5 - p_t)/(4.5 - 1.5)]}{1 + 1.624 \times 10^7/(D + 1)^{8.46}}$$
$$+ (4.22 - 0.32p_t)\log\left[\left(\frac{S_c'J}{S_cJ'}\right)\left(\frac{D^{0.75} - 18.42/Z^{0.25}}{D^{0.75} - 18.42/Z'^{0.25}}\right)\right] \tag{12.20}$$

Letting $Z = E_c/k = 4.2 \times 10^6/60 = 70{,}000$, $S_c = 690$, and $J = 3.2$; adding a drainage coefficient C_d and a reliability term Z_RS_o; replacing the term $(4.5 - p_t)$ by ΔPSI; and removing the primes for simplicity; the final design equation for rigid pavements becomes

$$\log W_{18} = Z_RS_o + 7.35\log(D + 1) - 0.06 + \frac{\log[\Delta PSI/(4.5 - 1.5)]}{1 + 1.624 \times 10^7/(D + 1)^{8.46}}$$
$$+ (4.22 - 0.32p_t)\log\left\{\frac{S_cC_d(D^{0.75} - 1.132)}{215.63J[D^{0.75} - 18.42/(E_c/k)^{0.25}]}\right\} \tag{12.21}$$

Figure 12.17 is a nomograph for solving Eq. 12.21. Note that p_t does not appear in the nomograph because it was assumed that $p_t = 4.5 - \Delta$PSI. The DNPS86 computer program can also be used to solve Eq. 12.21 and perform the design procedure.

Example 12.6

Given k = 72 pci (19.5 MN/m^3), $E_c = 5 \times 10^6$ psi (34.5 GPa), S_c = 650 psi (4.5 MPa), J = 3.2, C_d = 1.0, ΔPSI = 4.2 − 2.5 = 1.7, R = 95%, S_o = 0.29, and $W_t = 5.1 \times 10^6$, determine thickness D from Figure 12.17.

Solution: The required thickness D can be determined by the following steps:

1. Starting from Figure 12.17*a* with k = 72 pci (19.5 MN/m^3), a series of lines, as indicated by the arrows, are drawn through $E_c = 5 \times 10^6$ psi (34.5 GPa), S_c = 650 (4.5 MPa), J = 3.2, and C_d = 1.0 until a scale of 74 is obtained at the match line.
2. Starting at 74 on the match line in Figure 12.17*b*, a line is drawn through ΔPSI = 1.7 until it intersects the vertical axis.
3. From the scale with R = 95%, a line is drawn through S_o = 0.29 and then through $W_{18} = 5.1 \times 10^6$ until it intersects the horizontal axis.
4. A horizontal line is drawn from the last point in steps 2 and a vertical line from that in step 3. The intersection of these two lines gives a D of 9.75 in. (246 mm), which is rounded to 10 in. (254 mm).

Example 12.7

Same as Example 12.6 except that D is given as 9.75 in. (246 mm). Determine W_{18} by using Eq. 12.21.

Solution: For R = 95%, from Table 11.15, Z_R = −1.645. From Eq. 12.21, $\log W_{18} = -1.645 \times 0.29 + 7.35 \log(9.75 + 1) - 0.06 + \log(1.7/2.7)/[1 + 1.624 \times 10^7/(9.75 + 1)^{8.46}] + (4.22 - 0.32 \times 2.5) \log\{[(650 \times 1.0)/(215.63 \times 3.2)][(9.75)^{0.75} - 1.132]/[(9.75)^{0.75} - 18.42/(5 \times 10^6/72)^{0.25}]\} = -0.477 + 7.581 - 0.06 - 0.195 - 0.088 = 6.761$, or $W_{18} = 5.8 \times 10^6$, which checks well with the 5.2×10^6 obtained from the chart.

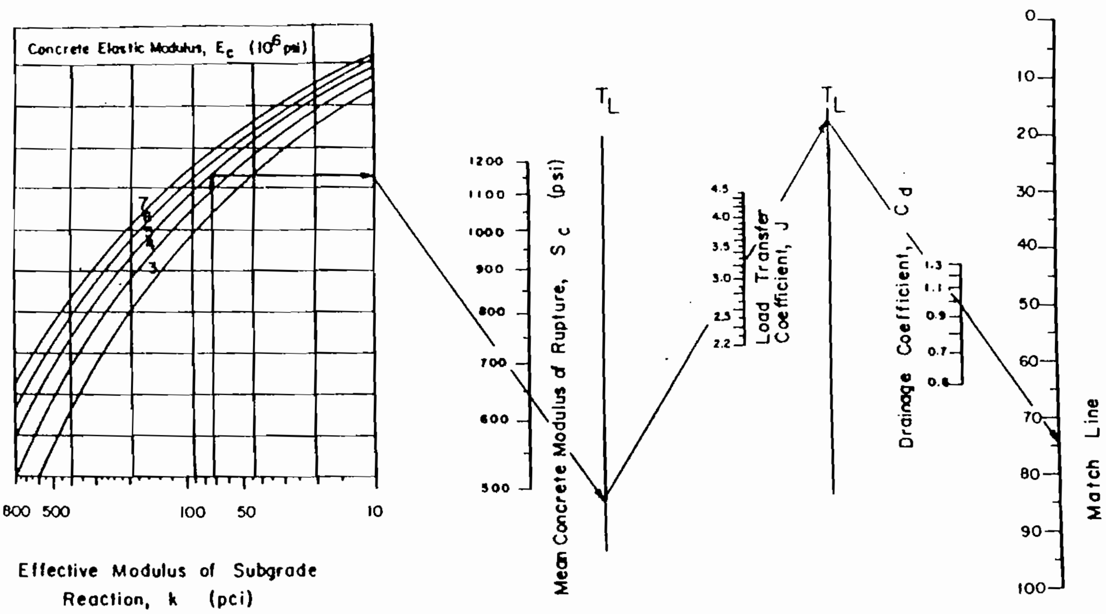

Figure 12.17(a) Design chart for rigid pavements based on mean values (1 in. = 25.4 mm, 1 psi = 6.9 kPa, 1 pci = 271.3 kN/m^3). (From the *AASHTO Guide for Design of Pavement Structures*. Copyright 1986. American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

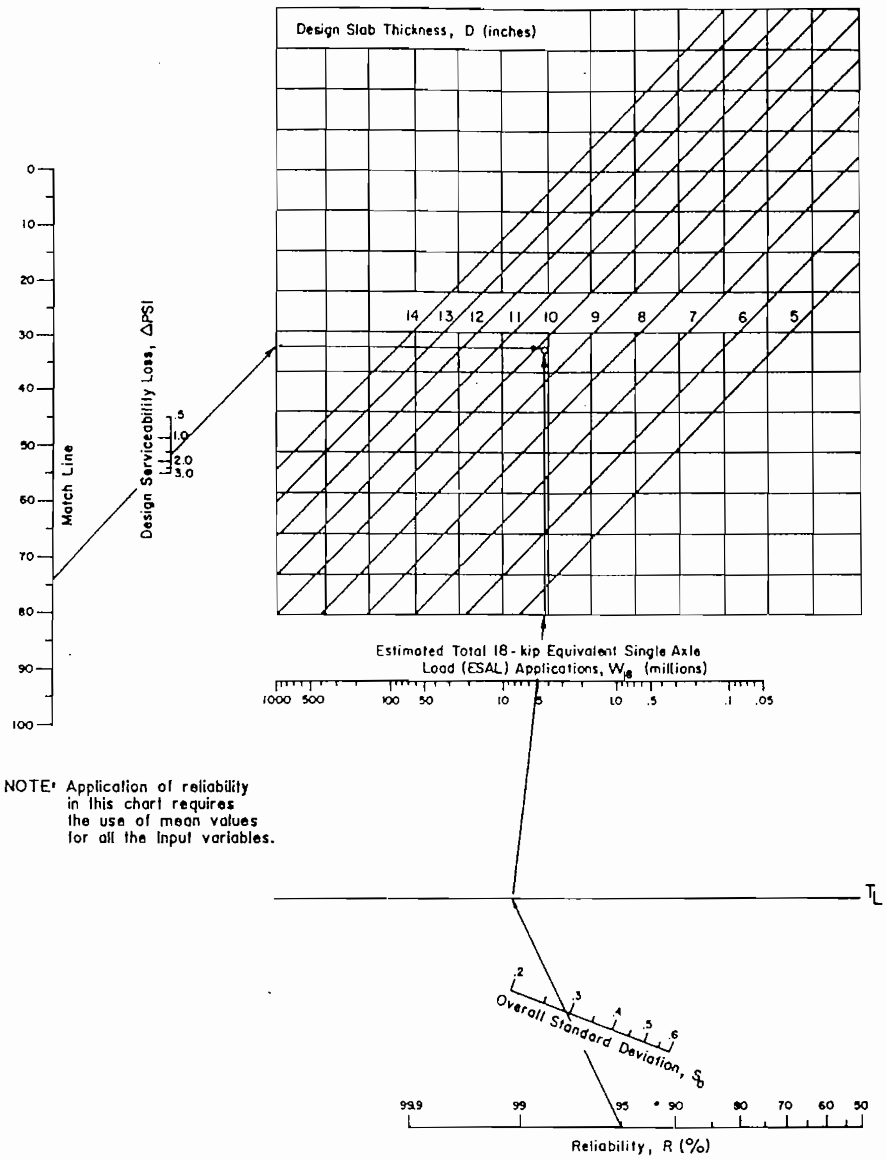

Figure 12.17(b) (cont.)

12.3.2 Modulus of Subgrade Reaction

The property of roadbed soil to be used for rigid pavement design is the modulus of subgrade reaction k, rather than the resilient modulus M_R. It is therefore necessary to convert M_R to k. Similar to M_R, the values of k also vary with the season of the year and the relative damage caused by the change of k needs to be evaluated.

Correlation with Resilient Modulus

As discussed in Section 5.1.1, there is no unique correlation between the modulus of subgrade reaction for liquid foundation and the resilient modulus for solid foundation. Any relationship between k and M_R is arbitrary and depends on whether stresses or deflections are to be compared or whether the loads are applied at the interior, edge, or corner of the slab.

Without Subbase

If the slab is placed directly on the subgrade without a subbase, AASHTO suggested the use of the following theoretical relationship based on an analysis of plate bearing test:

$$k = \frac{M_R}{19.4} \tag{12.22}$$

in which k is in pci and M_R is in psi. This equation gives a k value that is too large. For example, the resilient moduli equivalent to a k value of 100 pci (27.1 MN/m^3) are about 4000 psi (27.6 kPa) for an 8-in. (203-mm) slab and 4720 psi (32.6 MPa) for a 10-in. (254-mm) slab, based on the edge stress as shown in Table 5.13, while those based on the corner deflection are 6400 and 7560 psi (44.2 and 52.2 MPa), as shown in Table 5.14. For a k value of 100 pci (27.1 MPa), the resilient modulus obtained from Eq. 12.22 is only 1940 psi (13.4 MPa).

Equation 12.22 was based on the definition of k using a 30-in. (762-mm) plate. The deflection w_0 of a plate on a solid foundation can be determined by Eq. 2.10. The modulus of subgrade reaction, which is defined as the ratio between an applied pressure q and the deflection w_0, can be expressed as

$$k = \frac{q}{w_0} = \frac{2M_R}{\pi(1 - \nu^2)a} \tag{12.23}$$

in which ν is Poisson ratio of the foundation and a is the radius of the plate. If $\nu = 0.45$ and $a = 15$ in. (381 mm), then Eq. 12.23 becomes

$$k = \frac{M_R}{18.8} \tag{12.24}$$

Equation 12.24 is a more exact solution compared to Eq. 12.22, which is an approximate solution using the average surface deflection under a flexible loaded area as w_0 (AASHTO, 1985). Equations 12.22 and 12.24 give a k value that is too large because k is inversely proportional to a, as indicated by Eq. 12.23. To correlate k with M_R, a very large plate should be used. The use of a 30-in. (762-mm) plate is arbitrary and is the only practical way to obtain a given value of k because tests with larger plates will be more expensive and difficult to perform. Therefore, the use of Eq. 12.22 or 12.24 based on a 30-in. (762-mm) plate to compute k from M_R is misleading and will result in stresses and deflections that are too small compared with those based on M_R.

With Subbase

If a subbase exists between the slab and the subgrade, the composite modulus of subgrade reaction can be determined from Figure 12.18. The modulus is based on a subgrade of infinite depth and is denoted by k_∞. The chart was developed using the same method as for a homogeneous half-space except that the 30-in. (762-mm) plate is applied on a two-layer system. Therefore, the k values obtained from the chart are too large and do not represent what actually occurs in the field.

Example 12.8

Given a subbase thickness D_{SB} of 6 in. (152 mm), a subbase resilient modulus E_{SB} of 20,000 psi (138 MPa), and a roadbed soil resilient modulus M_R of 7000 psi (48 MPa), determine the composite modulus of subgrade reaction k_∞.

Solution: The composite modulus of subgrade reaction can be determined as follows:

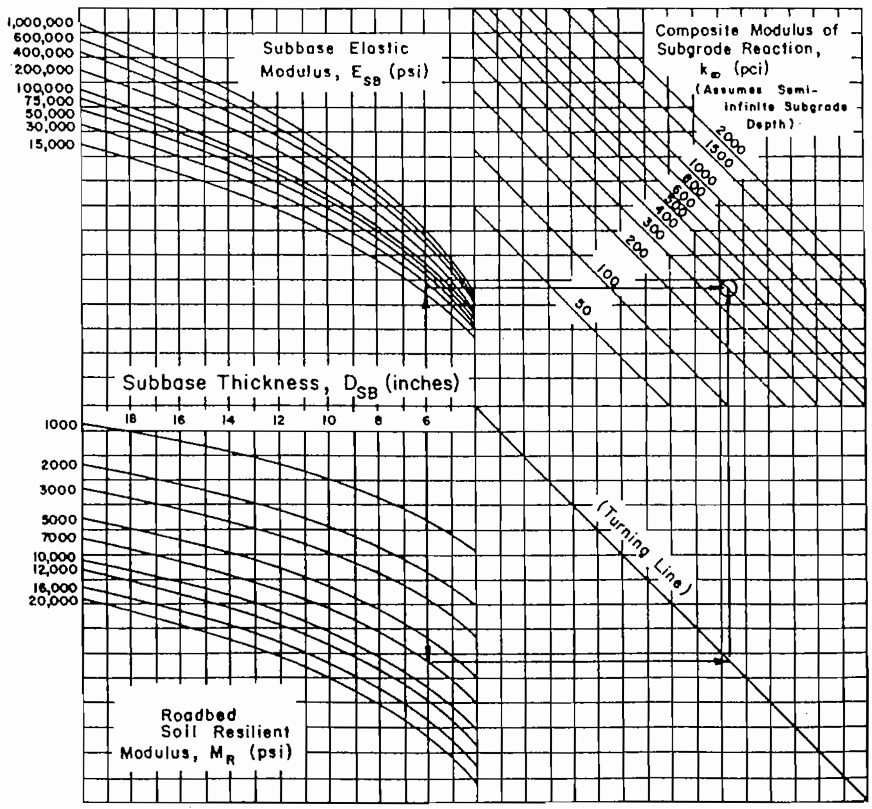

Figure 12.18 Chart for estimating modulus of subgrade reaction (1 in. = 25.4 mm, 1 psi = 6.9 kPa, 1 pci = 271.3 kN/m³). (From the *AASHTO Guide for Design of Pavement Structures*. Copyright 1986. American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

1. In Figure 12.18, a vertical line is drawn upward from the horizontal scale with a subbase thickness E_{SB} of 6 in. (152 mm) until it reaches a point with a subbase modulus E_{SB} of 20,000 psi (138 MPa).

2. The same line is drawn downward until it intersects the curve with a roadbed soil resilient modulus M_R of 7000 psi (48 MPa), and then the line is turned horizontally until it intersects the turning line.

3. A horizontal line is drawn from the point in step 1 and a vertical line from the point on the turning line in step 2. The intersection of these two lines gives a k_∞ of 400 pci (108 MN/m³).

Rigid Foundation at Shallow Depth

Equation 12.22 and Figure 12.18 are based on a subgrade of infinite depth. If a rigid foundation lies below the subgrade and the subgrade depth to rigid foundation D_{SG} is smaller than 10 ft (3 m), then the modulus of subgrade reaction must be modified by the chart shown in Figure 12.19. The chart can be applied to slabs either with or without a subbase.

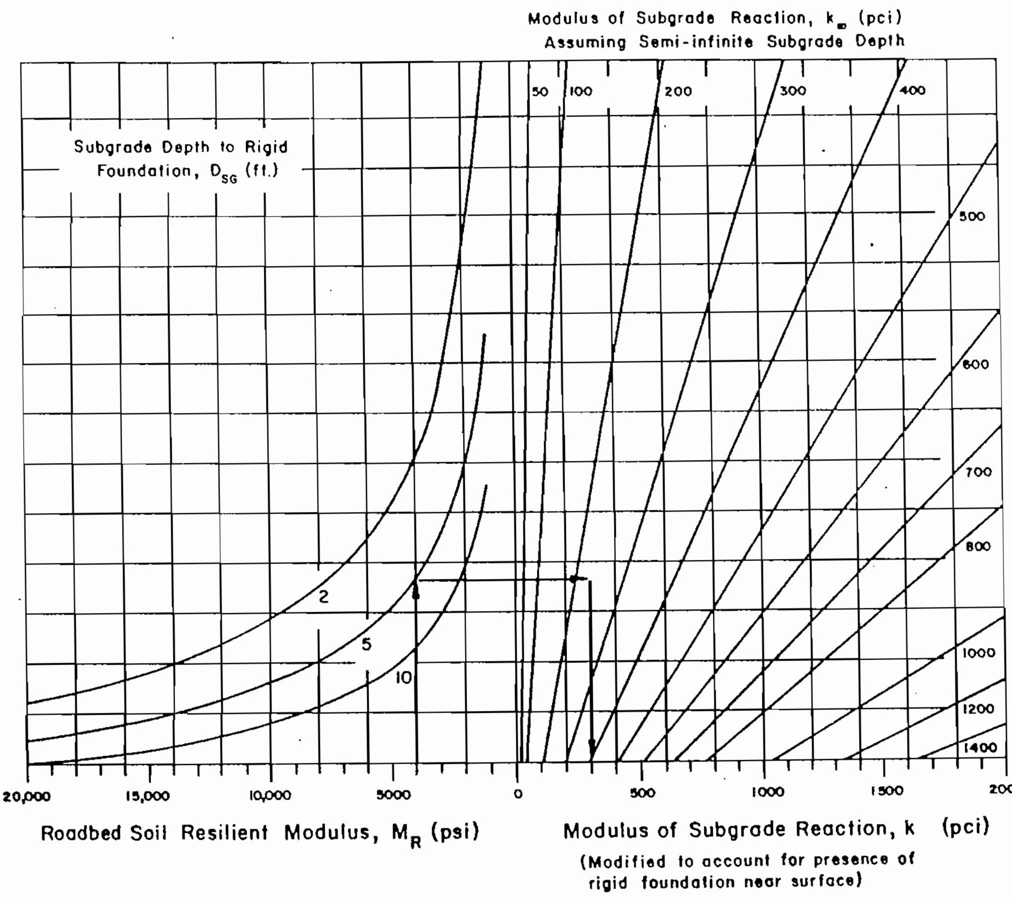

Figure 12.19 Chart for modifying modulus of subgrade reaction due to rigid foundation near surface (1 ft = 0.305 m, 1 psi = 6.9 kPa, 1 pci = 271.3 kN/m³). (From the *AASHTO Guide for Design of Pavement Structures*. Copyright 1986. American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

Example 12.9

Given M_R = 4000 psi (27.6 MPa), D_{SG} = 5 ft (1.52 m), and k_∞ = 230 pci (62.4 MN/m^3), determine k.

Solution: In Figure 12.19, a vertical line is drawn from the horizontal scale with a M_R of 4000 psi (27.6 MPa) until it intersects the curve with a D_{SG} of 5 ft. The line is turned horizontally until it reaches a point with a k_∞ of 230 pci (62.4 MN/m^3) and then vertically until a k of 300 pci (81.4 MN/m^3) is obtained.

Effective Modulus of Subgrade Reaction

The effective modulus of subgrade reaction is an equivalent modulus that would result in the same damage if seasonal modulus values were used throughout the year. The equation for evaluating the relative damage and the method for computing the effective k are discussed below.

Relative Damage

From Eq. 12.21, the effect of k on W_{18} can be expressed as

$$\log W_{18} = \log C - \log\left[\left(D^{0.75} - \frac{18.42k^{0.25}}{E_c^{0.25}}\right)^{(4.22-0.32p_t)}\right]$$

or

$$W_{18} = \frac{C}{(D^{0.75} - 18.42k^{0.25}/E_c^{0.25})^{(4.22-0.32p_t)}} \tag{12.25}$$

in which C is the sum of all terms except for those related to k. Because of the relatively small variation in E_c and p_t, Eq. 12.25 can be simplified by assuming that E_c = 5 × 10^6 psi (34.5 GPa) and p_t = 2.5:

$$W_{18} = \frac{C}{(D^{0.75} - 0.39k^{0.25})^{3.42}} \tag{12.26}$$

If W_T is the predicted total traffic, then the damage ratio can be expressed as

$$D_r = \frac{W_T}{C}(D^{0.75} - 0.39k^{0.25})^{3.42} \tag{12.27}$$

If W_T is distributed uniformly over n periods, then the cumulative damage ratio is

$$D_r = \frac{W_T}{C}\frac{1}{n}\sum_{i=1}^{n}(D^{0.75} - 0.39k_i^{0.25})^{3.42} \tag{12.28}$$

Equating Eq. 12.27 to Eq. 12.28 gives

$$(D^{0.75} - 0.39k^{0.25})^{3.42} = \frac{1}{n}\sum_{i=1}^{n}(D^{0.75} - 0.39k_i^{0.25})^{3.42} \tag{12.29}$$

Equation 12.29 can be used to determine the effective modulus of subgrade reaction k in terms of seasonal moduli k_i. The relative damage to rigid pavements u_r is defined as

$$u_r = (D^{0.75} - 0.39k^{0.25})^{3.42} \tag{12.30}$$

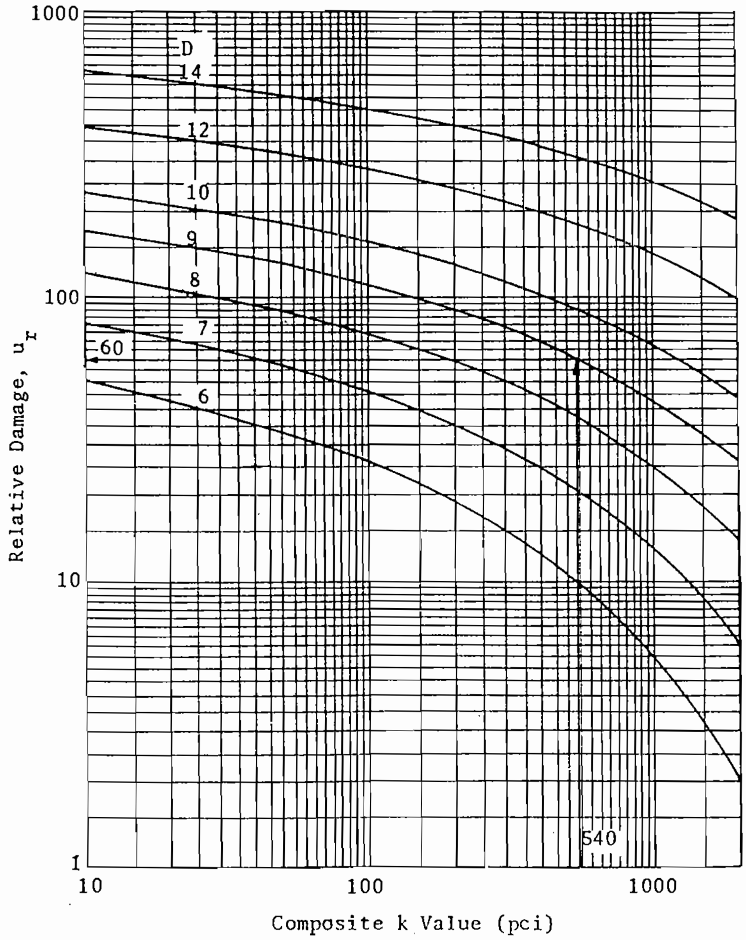

Figure 12.20 Chart for estimating relative damage to rigid pavements (1 in. = 25.4 mm, 1 pci = 271.3 kN/m^3). (From the *AASHTO Guide for Design of Pavement Structures*. Copyright 1986. American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

Figure 12.20 is a chart for solving Eq. 12.30

Example 12.10

Given D = 9 in. (229 mm) and k = 540 pci (147 MN/m^3), determine u_r by Eq. 12.30 and compare the result with Figure 12.20.

Solution: From Eq. 12.30, $u_r = [(9)^{0.75} - 0.39(540)^{0.25}]^{3.42} = 60.3$, which checks with a relative damage of 60 obtained from the chart.

Computation

Table 12.17 shows the computation of the effective modulus of subgrade reaction for a slab thickness of 9 in. (229 mm). It is assumed that the slab is placed directly on the subgrade having the monthly resilient moduli shown in the table.

TABLE 12.17 COMPUTATION OF EFFECTIVE MODULUS OF SUBGRADE REACTION

Month (1)	Roadbed modulus M_R (psi) (2)	k Value (pci) (3)	Relative damage u_r (%) (4)
Jan	4500	232	85.7
Feb	27,300	1407	34.2
Mar	50,000	2577	20.5
Apr	1350	70	121.4
May	2140	110	108.1
Jun	2930	151	98.7
Jul	3710	191	91.6
Aug	4500	232	85.7
Sep	4500	232	85.7
Oct	4500	232	85.7
Nov	4500	232	85.7
Dec	4500	232	85.7

Average $\bar{u}_r = \frac{\Sigma u_r}{n} = 82.4$ $\Sigma u_r = 988.7$

Effective modulus of subgrade reaction, $k = 263$ pci

Note. 1 psi = 6.9 kPa, 1 pci = 271.3 kN/m^3.

Explanation of Columns in Table 12.17

1. Each year is divided into 12 months, each with different subgrade moduli.
2. The roadbed resilient moduli are the same as those used in the DAMA program for a MAAT of 60°F (15.5°C) and a normal modulus of 4500 psi (31 MPa), as shown in Table 11.10. The maximum modulus is 50,000 psi (345 MPa) and occurs in March when the subgrade is frozen.
3. The k values are obtained from Eq. 12.22. Because no corrections for rigid foundation are needed, these k values can be used for computing relative damage.
4. The relative damage can be obtained from Eq. 12.30 or Figure 12.20. The sum of relative damage is 988.7 and the average over the 12 months is 82.4, which is equivalent to an effective modulus of 263 pci (71.4 MN/M^3).

It is interesting to note the significant difference in behaviors between flexible and rigid pavements under freeze–thaw conditions. For the flexible pavement analyzed in Figure 11.26, the damage caused by the spring breakup in May constitutes about 65% of the total damage, and the effective roadbed soil modulus is 2200 psi (15 MPa) versus the normal value of 4500 psi (31 MPa). For the rigid pavement analyzed in Table 12.17, the damage caused by the spring breakup in April constitutes only 12% of the total damage, and the effective modulus of subgrade reaction is 263 pci (71.4 MN/m^3) versus the normal value of 232 pci (62.9

MN/m³). The fact that the effective modulus is even greater than the normal modulus substantiates the claim by PCA (1984) that the normal summer or fall *k* values can be used for design purposes to avoid the tedious method of considering seasonal variations.

Loss of Subgrade Support

To account for the potential loss of support by foundation erosion or differential vertical soil movements, the effective modulus of subgrade reaction must be reduced by a factor, LS. Figure 12.21 shows a chart for correcting the effective modulus of subgrade reaction due to the loss of foundation support. For example, if the effective modulus of subgrade reaction for full contact, LS = 0, is 540 pci (147 MN/m³), the effective modulus of subgrade reaction for partial contact with LS = 1 is 170 pci (46 MN/m³).

Figure 12.21 was developed by computing the maximum principal stress under a single-axle load for four different contact conditions with LS = 0, 1, 2, and 3. The best case is LS = 0 when the slab and foundation are assumed to be in full contact. The worst case is LS = 3 when an area of slab, 9 ft (2.7 m) long and 7.25 ft (2.2 m) wide along the pavement edge, is assumed not to be in contact with the subgrade. The area assumed not to be in contact for LS = 2 is smaller than that for LS = 3 but greater than that for LS = 1. Since the result of AASHO Road Test indicates that the stresses produced in a concrete pavement are proportional to the number of load applications it can carry, the equivalent *k* value for partial

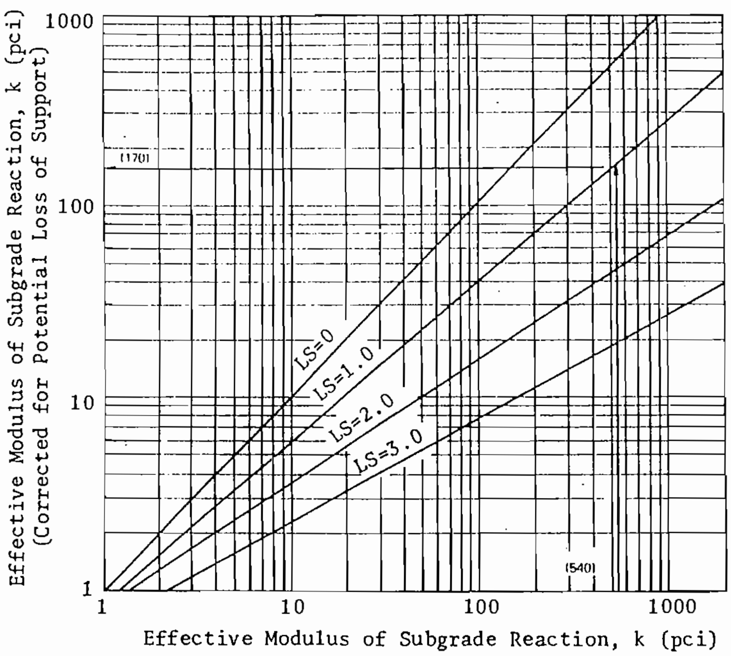

Figure 12.21 Correction of effective modulus of subgrade reaction due to loss of foundation contact (1 pci = 271.3 kN/m³). (After McCullough and Elkins (1979).)

TABLE 12.18 TYPICAL RANGES OF LS FACTORS FOR VARIOUS TYPES OF MATERIALS

Type of material	Loss of support (LS)
Cement-treated granular base ($E = 1 \times 10^6$ to 2×10^6 psi)	0.0 to 1.0
Cement aggregate mixtures (E = 500,000 to 1×10^6 psi)	0.0 to 1.0
Asphalt-treated bases (E = 350,000 to 1×10^6 psi)	0.0 to 1.0
Bituminous-stabilized mixture (E = 40,000 to 300,000 psi)	0.0 to 1.0
Lime-stabilized materials (E = 20,000 to 70,000 psi)	1.0 to 3.0
Unbound granular materials (E = 15,000 to 45,000 psi)	1.0 to 3.0
Fine-grained or natural subgrade materials (E = 3000 to 40,000 psi)	2.0 to 3.0

Note. E in this table refers to the general symbol of the resilient modulus.
Source. After AASHTO (1986).

contact can be obatined by varying the $k$values until the maximum principal stress for full contact is equal to that for partial contact.

Table 12.18 provides some suggested ranges of LS for different types of subbase and subgrade materials. In the selection of LS factor, consideration should be given to differential vertical soil movements that may result in voids beneath the pavement. Even though a nonerosive subbase is used, LS values of 2.0 to 3.0 may still be used for active swelling clays or excessive frost heave.

12.3.3 Design Variables

The design variables presented in Sections 11.3.1 and 11.3.6 for flexible pavements, such as time constraints, traffic, reliability, environmental effects, serviceability, stage construction, and the analysis of swelling and frost heave, are the same as those for rigid pavements and are not repeated here. Since the effective modulus of subgrade reaction is discussed in Section 12.3.2, only the elastic modulus of concrete E_c, the concrete modulus of rupture S_c, the load transfer coefficient J, and the drainage coefficient C_d are presented in this section.

Elastic Modulus of Concrete

The elastic modulus of concrete can be determined according to the procedure described in ASTM C469 or correlated with the compressive strength. The following is a correlation recommended by the American Concrete Institute:

$$E_c = 57{,}000\,(f'_c)^{0.5} \tag{12.31}$$

in which E_c is the concrete elastic modulus in psi and f'_c is the concrete compressive strength in psi as determined by AASHTO T22, T140, or ASTM C39.

Concrete Modulus of Rupture

The modulus of rupture required by the design procedure is the mean value determined after 28 days using third-point loading, as specified in AASHTO T97 or ASTM C78. If center-point loading is used, a correlation should be made between the two tests.

Load Transfer Coefficient

The load transfer coefficient J is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer a load across joints and cracks. The use of load transfer devices and tied concrete shoulders increases the amount of load transfer and decreases the load transfer coefficient. Table 12.19 shows the recommended load transfer coefficients for various pavement types and design conditions. The AASHO Road Test conditions represent a J value of 3.2, since all joints were doweled and there were no tied concrete shoulders.

Drainage Coefficient

The drainage coefficient C_d has the same effect as the load transfer coefficient J. As indicated by Eq. 12.21, an increase in C_d is equivalent to a decrease in J, both causing an increase in W_{18}. Table 12.20 provides the recommended C_d values based on the quality of drainage and the percentage of time during which the pavement structure would normally be exposed to moisture levels approaching saturation. Similar to flexible pavements, the percentage of time is dependent on the average yearly rainfall and the prevailing drainage conditions.

12.3.4 Comparison with PCA Method

It is difficult to compare the results between AASHTO and PCA methods because the AASHTO method is based on reliability, using mean values for all variables, while the PCA method does not consider reliability, but incorporates load safety factors and more conservative material properties. The AASHTO method is based on the equivalent 18-kip (80-kN) single-axle load applications and does not distinguish the type of distress, while the PCA method considers both fatigue cracking and foundation erosion using actual single- and tandem-axle loads. In view of the fact that fatigue cracking is more critical under single-axle loads and foundation erosion is more critical under tandem-axle loads, it is unreasonable to use ESAL for rigid pavement design because the conversion of a tandem-axle load

TABLE 12.19 RECOMMENDED LOAD TRANSFER COEFFICIENT FOR VARIOUS PAVEMENT TYPES AND DESIGN CONDITIONS

Type of shoulder	Asphalt		Tied PCC	
Load transfer devices	Yes	No	Yes	No
JPCP and JRCP	3.2	3.8–4.4	2.5–3.1	3.6–4.2
CRCP	2.9–3.2	N/A	2.3–2.9	N/A

Source. After AASHTO (1986).

TABLE 12.20 RECOMMENDED VALUES OF DRAINAGE COEFFICIENTS C_d FOR RIGID PAVEMENTS

Quality of drainage		Percentage of time pavement structure is exposed to moisture levels approaching saturation			
Rating	Water removed within	Less than 1%	1–5%	5–25%	Greater than 25%
Excellent	2 hours	1.25–1.20	1.20–1.15	1.15–1.10	1.10
Good	1 day	1.20–1.15	1.15–1.10	1.10–1.00	1.00
Fair	1 week	1.15–1.10	1.10–1.00	1.00–0.90	0.90
Poor	1 month	1.10–1.00	1.00–0.90	0.90–0.80	0.80
Very poor	Never drain	1.00–0.90	0.90–0.80	0.80–0.70	0.70

Source. After AASHTO (1986).

to an equivalent single-axle load actually changes the failure mode from the erosion at the joint to the fatigue at midslab. In the following comparison, it is assumed that the pavement is subjected to only one type of axle load, that is, the same 18-kip (80-kN) single-axle load, so the predominant mode of distress is fatigue cracking.

In applying the AASHTO method, the following parameter values are assumed: reliability R = 95%, standard deviation S_o = 0.35, serviceability loss $\Delta PSI = 2.0$ (p_t = 2.5), drainage coefficient C_d = 1.0, load transfer coefficient J = 3.2 without concrete shoulders and 2.5 with concrete shoulders, concrete modulus of rupture S_c = 650 psi (4.5 MPa), and modulus of subgrade reaction k = 100 pci (27.1 MN/m^3). In the PCA method, a load safety factor of 1.2 and the same S_c of 650 psi (4.5 MPa) and k of 100 pci (27.1 MN/m^3) are assumed. The PCA design chart has already taken into account the variation of S_c and the increase of S_c with time, so only the input of S_c at 28 days is required, which is the same as the S_c specified by AASHTO. The same k values are used in both methods because the normal summer or fall modulus of subgrade reaction specified by PCA is not too much different from the effective modulus of subgrade reaction specified by AASHTO, as illustrated by the example in Table 12.17. Table 12.21 shows a comparison of thickness design between the AASHTO and PCA methods.

In Table 12.21, comparisons are made for pavements both with and without tied concrete shoulders. The allowable ESAL for five different slab thicknesses are shown. The AASHTO ESAL was computed by the AASHTO design equation (Eq. 12.21), and the PCA ESAL was obtained from the PCA design tables and charts based on the fatigue criterion (Table 12.6 or 12.7 and Figure 12.12) using a single-axle load of 21.6 kip (96 kN), which is the 18-kip (80-kN) load multiplied by a load safety factor of 1.2. It was found that when the joints are doweled, the use of the erosion criterion is not as critical as the use of the fatigue criterion. The thickness by PCA, as shown in Table 12.21, is the slab thickness determined by the PCA method based on the AASHTO ESAL.

As can be seen from Table 12.21, a large difference in ESAL exists between the two methods. For slabs with a thickness smaller than 8 in. (203 mm) and no concrete shoulder or slabs with a thickness smaller than 7 in. (178 mm) and a tied

TABLE 12.21 COMPARISON OF THICKNESS BETWEEN AASHTO AND PCA METHODS

Slab thickness (in.)	No concrete shoulder			With concrete shoulder		
	AASHTO ESAL	PCA ESAL	Thickness by PCA (in.)	AASHTO ESAL	PCA ESAL	Thickness by PCA (in.)
5	1.2×10^5	<100	7.5	2.8×10^5	350	6.6
6	3.2×10^5	500	7.9	7.4×10^5	4.0×10^4	6.9
7	7.4×10^5	3.0×10^4	8.1	1.7×10^6	1.0×10^6	7.1
8	1.6×10^6	5.0×10^5	8.2	3.7×10^6	$>10^7$	7.2
9	3.3×10^6	$>10^7$	8.3	7.7×10^6	$>10^7$	7.3

Note. 1 in. = 25.4 mm.

concrete shoulder, the ESAL determined by the AASHTO method is one or more orders of magnitude greater than that obtained by the PCA method. Therefore, the use of AASHTO equation for thin pavements, such as concrete shoulders, is less conservative. For example, with an ESAL of 2.8×10^5 and a tied concrete shoulder, a thickness of 5 in. (122 mm) is sufficient by the AASHTO equation but the thickness required by the PCA method is 6.6 in. (168 mm).

It appears that the ESAL obtained by the PCA method is more reasonable, at least for thinner pavements. Consider the case of a 5-in. (127-mm) slab without a concrete shoulder. A run of KENSLABS shows that the critical edge stress under an 18-kip (80-kN) single-axle load is 641 psi (4.4 MPa), which is nearly equal to the concrete modulus of rupture. Thus, it is not possible that the pavement can withstand 120,000 applications of an 18-kip (80-kN) single-axle load as computed by the AASHTO equation.