Original Equations The following are the basic equations developed from the AASHO Road Test for flexible pavements (HRB, 1962):

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

$$\beta = 0.40 + \frac{0.081\,(L_1 + L_2)^{3.23}}{(\mathrm{SN} + 1)^{5.19} L_2^{3.23}} \tag{11.30}$$

$$\log \rho = 5.93 + 9.36 \log(\mathrm{SN} + 1) - 4.79 \log(L_1 + L_2) + 4.33 \log L_2 \tag{11.31}$$

Here,

- G_t = logarithm of the ratio of loss in serviceability at time t to the potential loss taken at a point where $p_t = 1.5$, or $G_t = \log[(4.2 - p_t)/(4.2 - 1.5)]$, noting that 4.2 is the initial serviceability for flexible pavements;
- β = a function of design and load variables, as shown by Eq. 11.30, that influences the shape of ρ versus W_t curve;
- ρ = a function of design and load variables, as shown by Eq. 11.31, that denotes the expected number of load applications to a p_t of 1.5, as can be seen from Eq. 11.29, where $\rho = W_t$ when $p_t = 1.5$;
- W_t = axle load application at end of time t;
- p_t = serviceability at end of time t;
- L_1 = load on one single axle or a set of tandem axles, in kip;
- L_2 = axle code—1 for single axle, 2 for tandem axle;
- SN = structural number of pavement, which was computed as

$$\mathrm{SN} = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{11.32}$$

in which a_1, a_2, and a_3 are layer coefficients for the surface, base, and subbase, respectively, and D_1, D_2, and D_3 are the thicknesses of the surface, base, and subbase, respectively.

The procedure is greatly simplified if an equivalent 18-kip (80-kN) single axle load is used. By combining Eqs. 11.29, 11.30, and 11.31 and setting $L_1 = 18$ and $L_2 = 1$, we obtain the equation

$$\log W_{t18} = 9.36 \log(\mathrm{SN} + 1) - 0.20 + \frac{\log[(4.2 - p_t)/(4.2 - 1.5)]}{0.4 + 1094/(\mathrm{SN} + 1)^{5.19}} \tag{11.33}$$

in which W_{t18} is the number of 18-kip (80-kN) single-axle load applications to time t and p_t is the terminal serviceability index. Equation 11.33 is applicable only to the flexible pavements in the AASHO Road Test with an effective subgrade resilient modulus of 3000 psi (20.7 MPa).

Modified Equations For other subgrade and environmental conditions, Eq. 11.33 is modified to

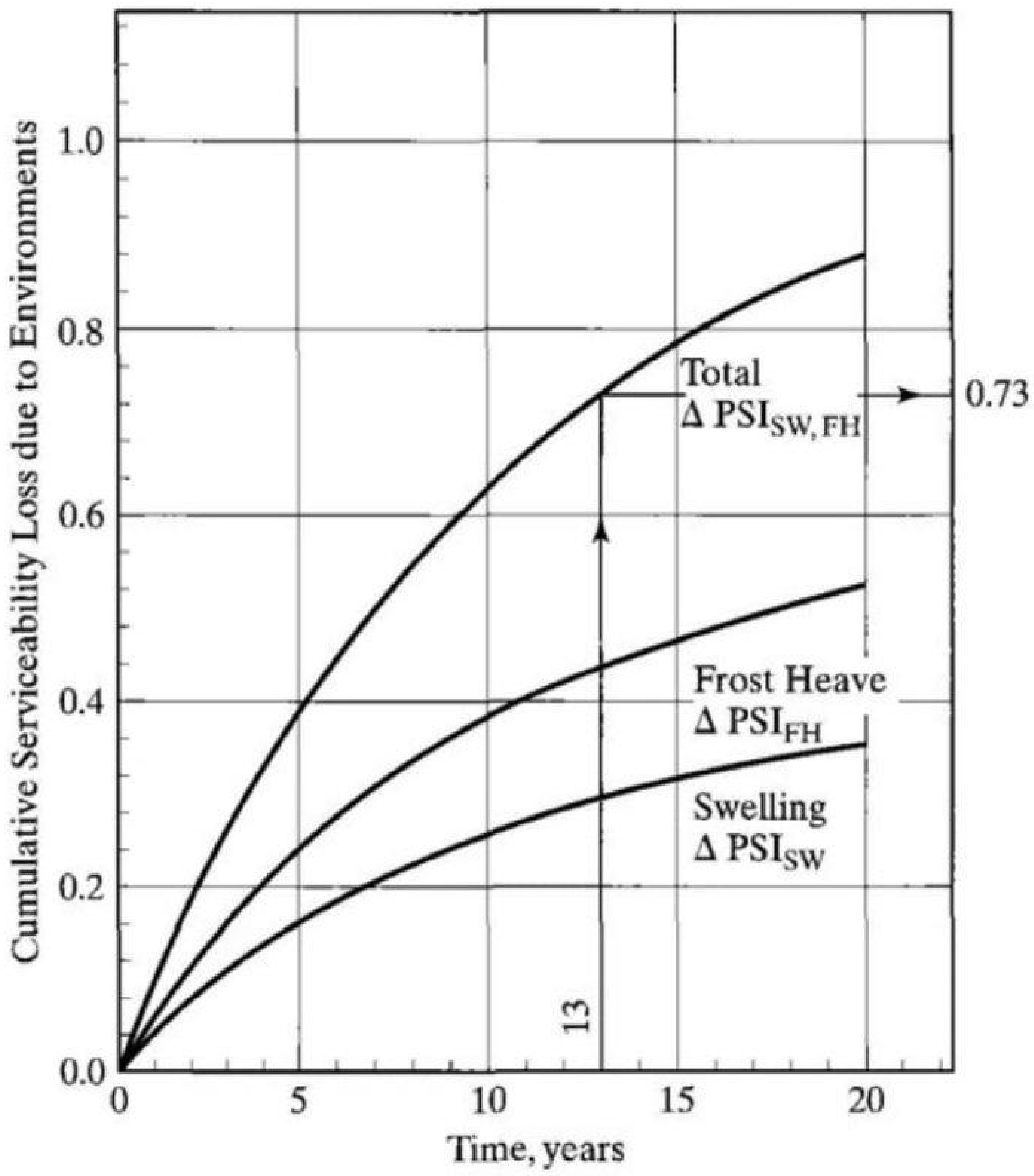

FIGURE 11.23

Environmental serviceability loss versus time for a specific location. (From the *AASHTO Guide for Design of Pavement Structures*. Copyright 1986. American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

The serviceability loss due to roadbed swelling depends on the swell rate constant, the potential vertical rise, and the swell probability; that due to frost heave depends on the frost heave rate, the maximum potential serviceability loss, and the frost heave probability. Methods for evaluating these losses are described in Appendix G of the AASHTO design guide.

Serviceability Initial and terminal serviceability indexes must be established to compute the change in serviceability, ΔPSI, to be used in the design equations. The initial serviceability index is a function of pavement type and construction quality. Typical values from the AASHO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavements. The terminal serviceability index is the lowest index that will be tolerated before rehabilitation, resurfacing, and reconstruction become necessary. An index of 2.5 or higher is suggested for design of major highways and 2.0 for highways with lower traffic. For relatively minor highways where economics dictate a minimum initial capital outlay, it is suggested that this be accomplished by reducing the design period or total traffic volume, rather than by designing a terminal serviceability index less than 2.0.

11.3.2 Design Equations

The original equations were based purely on the results of the AASHO Road Test but were modified later by theory and experience to take care of subgrade and climatic conditions other than those encountered in the Road Test.

TABLE 11.15 Standard Normal Deviates for Various Levels of Reliability

Reliability (%)	Standard normal deviate (Z_R)	Reliability (%)	Standard normal deviate (Z_R)
50	0.000	93	−1.476
60	−0.253	94	−1.555
70	−0.524	95	−1.645
75	−0.674	96	−1.751
80	−0.841	97	−1.881
85	−1.037	98	−2.054
90	−1.282	99	−2.327
91	−1.340	99.9	−3.090
92	−1.405	99.99	−3.750

Example 11.10:

Given $W_{18} = 5 \times 10^6$, $R = 95\%$, $S_0 = 0.35$, $M_R = 5000$ psi (34.5 MPa), and $\Delta PSI = 1.9$, determine SN from Figure 11.25.

Solution: As shown by the arrows in Figure 11.25, starting from $R = 95\%$, a series of lines are drawn through $S_0 = 0.35$, $W_{18} = 5 \times 10^6$, $M_R = 5000$ psi (34.5 MPa), and $\Delta PSI = 1.9$ and finally intersect SN at 5.0, so SN = 5.0.

The chart is most convenient for determining SN, because the solution of SN by Eq. 11.37 is cumbersome and requires a trial and error process. If W_{18} is the unknown to be determined, the use of Eq. 11.37 is more accurate.

Example 11.11:

Given $R = 95\%$, SN = 5, $S_0 = 0.35$, $M_R = 5000$ psi (34.5 MPa), $\Delta PSI = 1.9$, determine W_{18} by Eq. 11.37.

Solution: For $R = 95\%$, from Table 11.15, $Z_R = -1.645$. From Eq. 11.37, $\log W_{18} = -1.645 \times 0.35 + 9.36 \log(5 + 1) - 0.2 + \log(1.9/2.7)/[0.4 + 1094/(6)^{5.19}] + 2.32 \log(5000) - 8.07 = 6.714$, or $W_{18} = 5.18 \times 10^6$, which checks with 5×10^6 in the previous example.

11.3.3 Effective Roadbed Soil Resilient Modulus

The effective roadbed soil resilient modulus M_R is an equivalent modulus that would result in the same damage if seasonal modulus values were actually used. The equation for evaluating the relative damage to flexible pavements u_f and the method for computing M_R are discussed next.

Relative Damage From Eq. 11.37, the effect of M_R on W_{18} can be expressed as

$$\log W_{18} = \log C - \log(1.18 \times 10^8 M_R^{-2.32}) \tag{11.38}$$

TABLE 11.14 Suggested Levels of Reliability for Various Functional Classifications

Functional classification	Recommended level of reliability	
	Urban	Rural
Interstate and other freeways	85–99.9	80–99.9
Principal arterials	80–99	75–95
Collectors	80–95	75–95
Local	50–80	50–80

Note. Results based on a survey of AASHTO Pavement Design Task Force.
Source. After AASHTO (1986).

design process to ensure that the various design alternatives will last the analysis period. The level of reliability to be used for design should increase as the volume of traffic, difficulty of diverting traffic, and public expectation of availability increase. Table 11.14 presents recommended levels of reliability for various functional classifications.

Application of the reliability concept requires the selection of a standard deviation that is representative of local conditions. It is suggested that standard deviations of 0.49 be used for flexible pavements and 0.39 for rigid pavements. These correspond to variances of 0.2401 and 0.1521, which are nearly the same as those shown in Table 10.12.

When stage construction is considered, the reliability of each stage must be compounded to achieve the overall reliability; that is,

$$R_{\text{stage}} = (R_{\text{overall}})^{1/n} \tag{11.28}$$

in which n is the number of stages being considered. For example, if two stages are contemplated and the desired level of overall reliability is 95%, the reliability of each stage must be $(0.95)^{1/2}$, or 97.5%.

Environmental Effects The AASHO design equations were based on the results of traffic tests over a two-year period. The long-term effects of temperature and moisture on the reduction of serviceability were not included. If problems of swell clay and frost heave are significant in a given region and have not been properly corrected, the loss of serviceability over the analysis period should be estimated and added to that due to cumulative traffic loads. Figure 11.23 shows the serviceability loss versus time curves for a specific location. The environmental loss is a summation of losses from both swelling and frost heave. The chart may be used to estimate the serviceability loss at any intermediate period, for example, a loss of 0.73 at the end of 13 years. Of course, if only swelling or frost heave is considered, there will be only one curve on the graph. The shape of these curves indicates that the serviceability loss due to environment increases at a decreasing rate. This may favor the use of stage construction because most of the loss will occur during the first stage and can be corrected with little additional loss in later stages.

TABLE 7.4 Comparison of CBR, R Value, and Resilient Modulus

Soil description	CBR test		R value test		Triaxial test
	CBR	M_R (psi) by eq. 7.6	R	M_R (psi) by eq. 7.7	M_R (psi)
Sand	31	46,500	60	34,500	16,900
Silt	20	30,000	59	33,900	11,200
Sandy loam	25	37,500	21	12,800	11,600
Silt-clay loam	25	37,500	21	12,800	17,600
Silty clay	7.6	11,400	18	11,000	8200
Heavy clay	5.2	7800	<5	<3900	14,700

Note. 1 psi = 6.9 kPa.
Source: After AI (1982).

Heukelom and Klomp (1962) show that

$$M_R = 1500\,(\mathrm{CBR}) \tag{7.6}$$

in which M_R is the resilient modulus in psi. The coefficient, 1500, could vary from 750 to 3000, with a factor of 2. Available data indicate that Eq. 7.6 provides better results at values of CBR less than about 20. In other words, the correlation appears to be more reasonable for fine-grained soils and fine sands than for granular materials.

The Asphalt Institute (1982) proposed the following correlation between M_R and the R value:

$$M_R = 1155 + 555R \tag{7.7}$$

Laboratory data obtained from six different soil samples were used by the Asphalt Institute (1982) to illustrate the relationships, as shown in Table 7.4. The R values were obtained at an exudation pressure of 240 psi (1.7 MPa). The CBR samples were compacted at optimum moisture content to maximum density and soaked before testing. The repeated load triaxial tests were performed at optimum conditions using a deviator stress of 6 psi (41 kPa) and a confining pressure of 2 psi (14 kPa).

It can be seen from Table 7.4 that the equations for estimating M_R from CBR and R values have a very limited range. The resilient moduli estimated from CBR values of 5.2 and 7.6 and R values of 18 and 21 generally conform to the guidelines for accuracy within a factor of 2. Estimates from CBR values of 25 or higher and R values above 60 would appear to overestimate M_R by Eqs. 7.6 and 7.7.

It should be noted that the M_R of granular materials increases with the increase in confining pressure, and that of fine-grained soils decreases with the increase in deviator stress. Therefore, a large variety of correlations might be obtained, depending on the confining pressure or the deviator stress to be used in the resilient modulus test.

Hot Mix Asphalt Figure 7.13 shows the relationships of the layer coefficient, Marshall stability, cohesiometer values, and resilient modulus.

TABLE 11.18 Typical Values of K_1 and K_2 for Granular Subbase Materials

Moisture condition	K_1	K_2
Dry	6000–8000	0.4–0.6
Damp	4000–6000	0.4–0.6
Wet	1500–4000	0.4–0.6

Source. After AASHTO (1986).

TABLE 11.19 Values of Resilient Modulus for AASHO Road Test Subbase Materials

			Stress state θ (psi)		
Moisture condition	K_1	K_2	5	7.5	10
Damp	5400	0.6	14,183	18,090	21,497
Wet	4600	0.6	12,082	15,410	18,312

Note. Resilient modulus is in psi; 1 psi = 6.9 kPa.
Source. After Finn *et al.* (1986).

TABLE 11.20 Recommended Drainage Coefficients for Untreated Bases and Subbases in Flexible 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	1.40–1.35	1.35–1.30	1.30–1.20	1.20
Good	1 day	1.35–1.25	1.25–1.15	1.15–1.00	1.00
Fair	1 week	1.25–1.15	1.15–1.05	1.00–0.80	0.80
Poor	1 month	1.15–1.05	1.05–0.80	0.80–0.60	0.60
Very poor	Never drain	1.05–0.95	0.95–0.75	0.75–0.40	0.40

Source. After AASHTO (1986).

Table 11.20 shows the recommended drainage coefficients for untreated base and subbase materials in flexible pavements. The quality of drainage is measured by the length of time for water to be removed from bases and subbases and depends primarily on their permeability. The percentage of time during which the pavement structure is exposed to moisture levels approaching saturation depends on the average yearly rainfall and the prevailing drainage conditions.

11.3.5 Selection of Layer Thicknesses

Once the design structural number SN for an initial pavement structure is determined, it is necessary to select a set of thicknesses so that the provided SN, as computed by Eq. 11.35, will be greater than the required SN. Note that Eq. 11.35 does not have a single unique solution. Many combinations of layer thicknesses are acceptable, so their cost effectiveness along with the construction and maintenance constraints must be considered