Pavement Analysis and Design

TE-503 A/TE-503

Lecture-10
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AASHTO Method

The design procedure recommended by the American Association of State Highway and Transportation Officials (AASHTO) is based on the results of the extensive AASHO Road Test conducted in Ottawa, Illinois, in the late 1950s and early 1960s.

The AASHO Committee on Design first published an interim design guide in 1961. It was revised in 1972 and 1981. In 1984-85, the Subcommittee on Pavement Design and a team of consultants revised and expanded the guide under National Cooperative Highway Research Program (NCHRP) Project 20-7/24; they issued the guide in 1986. The guide was revised in 1993 with practically no change in the design method.

AASHTO Method

The empirical performance equations obtained from the AASHO Road Test are still being used as the basic models in the current guide, but were modified and extended to make them applicable to other regions in the nation. It should be kept in mind that the original equations were developed under a given climatic setting with a specific set of pavement materials and subgrade soils.

The climate at the test site is temperate with an average annual precipitation of about 34 in. The average depth of frost penetration is about 28 in. The subgrade soils consists of A-6 and A-7-6 that are poorly drained, with CBR values ranging from 2 to 4.

AASHTO Method-Design Variables

Time Constraints

To achieve the best use of available funds, the AASHTO design guide encourages the use of a longer analysis period for high-volume facilities, including at least one rehabilitation period. Thus, the analysis period should be equal to or greater than the performance period.

AASHTO Method-Design Variables-Performance Period

The performance period refers to the time that an initial pavement structure will last before it needs rehabilitation or the performance time between rehabilitation operations. It is equivalent to the time elapsed as a new, reconstructed or rehabilitated structure deteriorates from its initial serviceability to its terminal serviceability.

The designer must select the performance period within the minimum and maximum allowable bounds that are established by agency experience and policy. The selection of performance period can be affected by such factors as the functional classification of the pavement, the type and level of maintenance applied, the funds available for initial construction, life cycle costs and other engineering considerations.

AASHTO Method-Design Variables-Analysis Period

The analysis period is the period of time that any design strategy must cover. It may be identical to the selected performance period. However, realistic performance limitations may necessitate the consideration of staged construction or planned rehabilitation for the desired analysis period.

In the past, pavements were typically designed and analyzed for a 20-year performance period. It is now recommended that consideration be given to longer analysis periods because they can be better suited for the evaluation of alternative long-term strategies based on life cycle costs. Table 11.13 contains general guidelines for the length of the analysis period.

AASHTO Method-Design Variables-Analysis Period

TABLE 11.13 Guidelines for Length of Analysis Period

Highway conditions	Analysis period (years)
High-volume urban	30–50
High-volume rural	20–50
Low-volume paved	15–25
Low-volume aggregate surface	10–20

Source. After AASHTO (1986).

AASHTO Method-Design Variables-Traffic

The design procedures are based on cumulative expected 18-kip (80-kN) equivalent single-axle load (ESAL).

If a pavement is designed for the analysis period without any rehabilitation or resurfacing, all that is required is the total ESAL over the analysis period.

However, if stage construction is considered and rehabilitation or resurfacing is anticipated, a graph or equation of cumulative ESAL versus time is needed so that the ESAL traffic during any given stages can be obtained.

AASHTO Method-Design Variables-Reliability

Reliability is a means of incorporating some degree of certainty into the 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.

AASHTO Method-Design Variables-Reliability

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).

AASHTO Method-Design Variables-Reliability

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

$$R_{stage} = (R_{overall})^{1/n}$$

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%

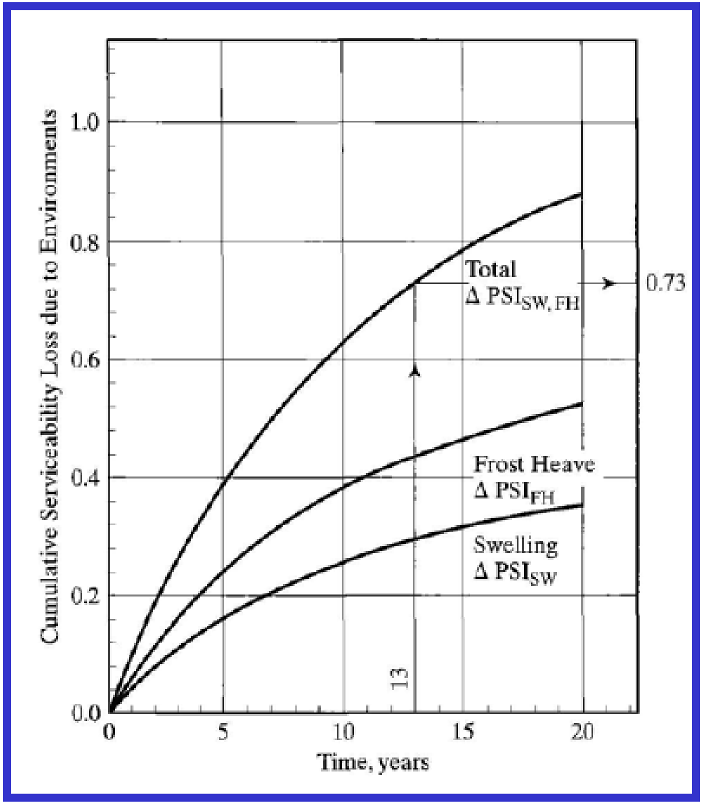
AASHTO Method-Design Variables-Environmental Effects

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 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.

Flexible Pavement Design

AASHTO Method-Design Variables-Environmental Effects



AASHTO Method-Design Variables-Environmental Effects

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 favour 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.

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.

AASHTO Method-Design Variables-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.

Flexible Pavement Design

AASHTO Method-Design Variables-Design Equation

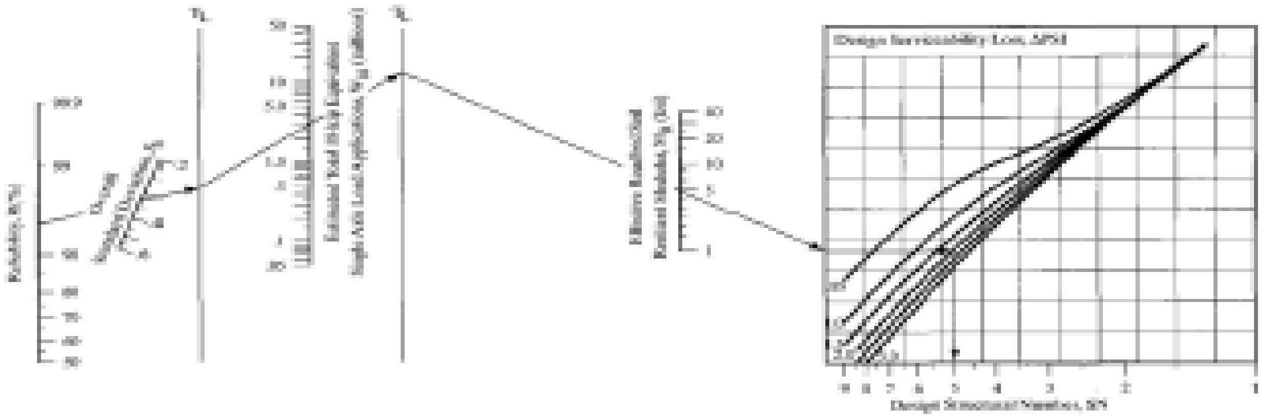
$$\log W_{18} = Z_R S_0 + 9.36 \log(\text{SN} + 1) - 0.20 + \frac{\log[\Delta \text{PSI}/(4.2 - 1.5)]}{0.4 + 1094/(\text{SN} + 1)^{5.19}} + 2.32 \log M_R - 8.07$$

$$\text{SN} = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

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

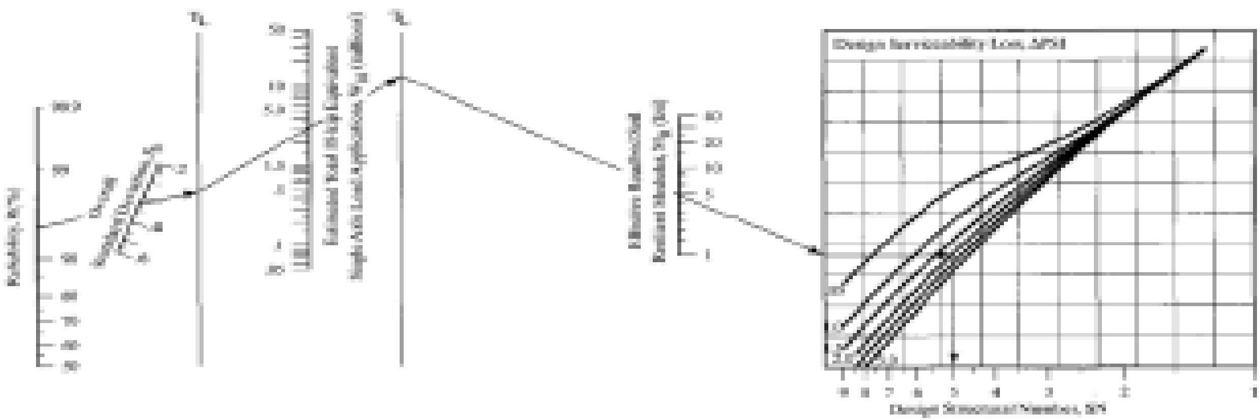
AASHTO Method-Design Variables-Design Nomograph



AASHTO Method-Design Variables-Design Nomograph-Numerical problem

Given W_{18} = 5 x 10^6, R = 95%, S_o = 0.35, M_R = 5000 psi and ΔPSI =1.9, determine SN using nomograph.

Solution: As shown by the arrows on nomograph, starting from R = 95%, a series of lines are drawn through S_o=0.35, W_{18}=5x10^6, M_R=5000 psi and ΔPSI=1.9 and finally intersect SN at 5.0. So SN=5.0.



AASHTO Method-Design Variables-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 is as under:

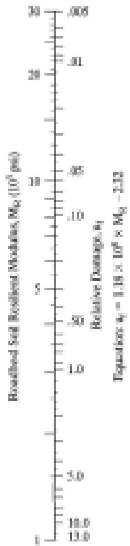
$$u_f = 1.18 \times 10^8 M_R^{-2.32}$$

Flexible Pavement Design

AASHTO Method-Design Variables-Computation of Effective Roadbed Soil Resilient Modulus

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.	15,800	0.02
Feb.	27,300	0.01
Mar.	38,700	0.00
Apr.	30,000	0.00
May	900	16.52
June	1,620	4.22
July	2,340	1.80
Aug.	3,060	0.97
Sept.	3,780	0.59
Oct.	4,500	0.39
Nov.	4,500	0.39
Dec.	4,500	0.39
Summation: Σu_f =		25.30

Average: $\bar{u}_f = \frac{\Sigma u_f}{n} = \underline{2.11}$



Effective Roadbed Soil Resilient Modulus, M_R (psi) = $\underline{2,200}$ (corresponds to $\bar{u}_f$)

AASHTO Method-Design Variables-Structural Number

Structural number is a function of layer thicknesses, layer coefficients and drainage coefficients and can be computed from:

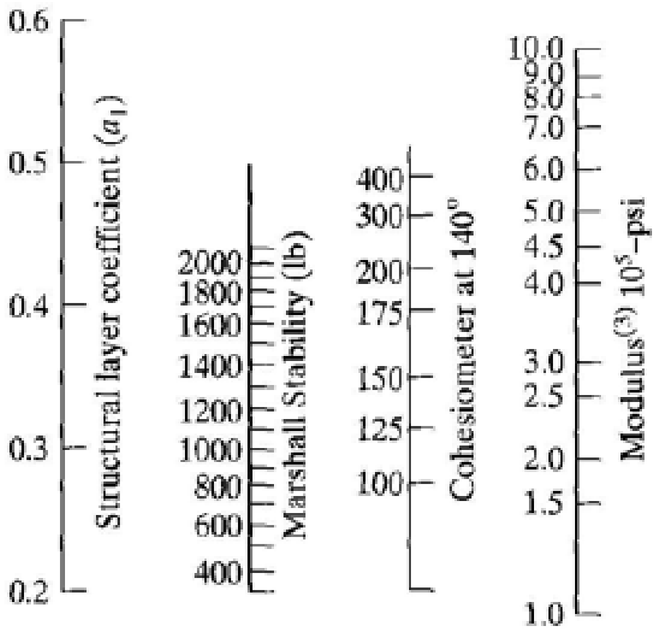
$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

Layer Coefficient

The layer coefficient a_i, is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. Layer coefficients can be determined from test roads or satellite sections, as was done in the AASHO Road Test, or from correlations with material properties, as shown in Figures.

Flexible Pavement Design

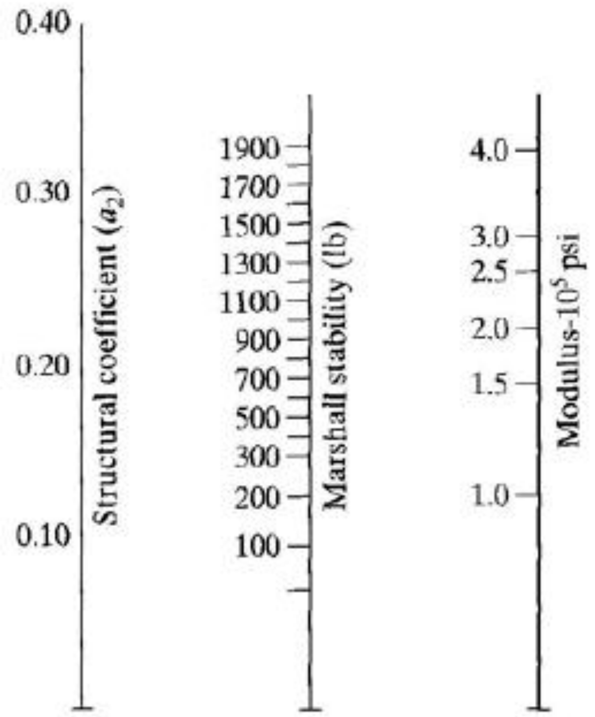
AASHTO Method-Design Variables-Structural Number-Layer coefficients



(a) Surface Course

Flexible Pavement Design

AASHTO Method-Design Variables-Structural Number-Layer coefficients



(b) Base Course

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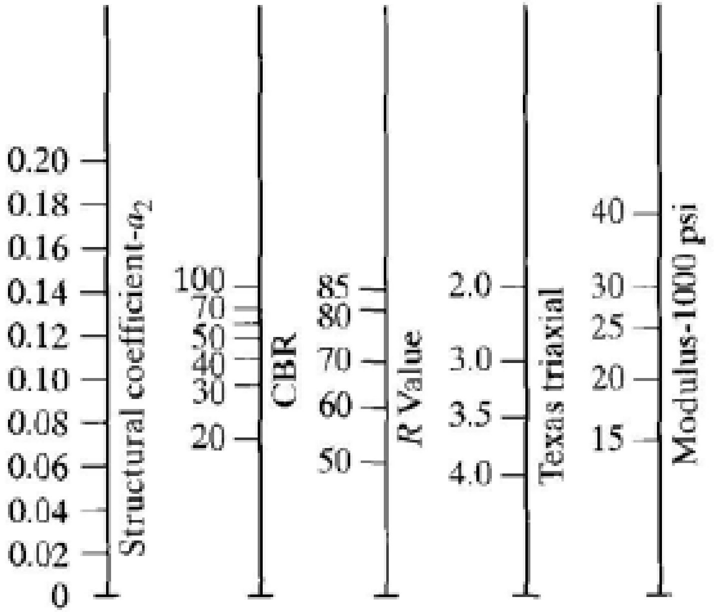
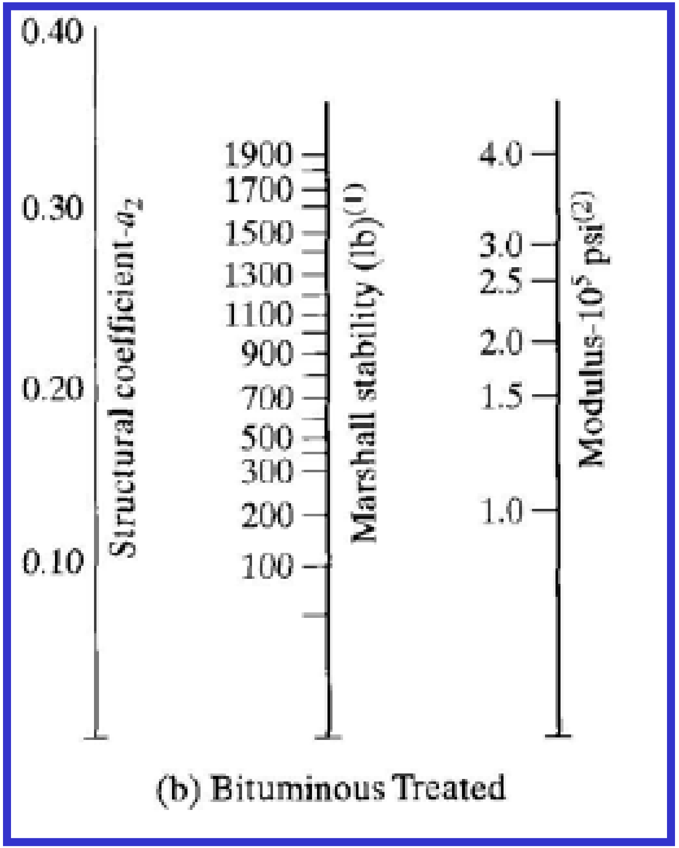
AASHTO Method-Design Variables-Structural Number-Layer coefficients



(a) Untreated

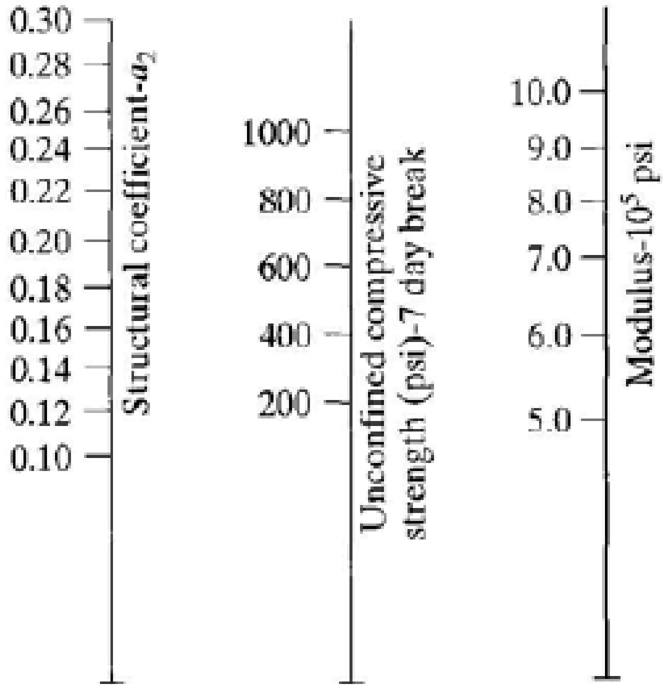
Flexible Pavement Design

AASHTO Method-Design Variables-Structural Number-Layer coefficients



Flexible Pavement Design

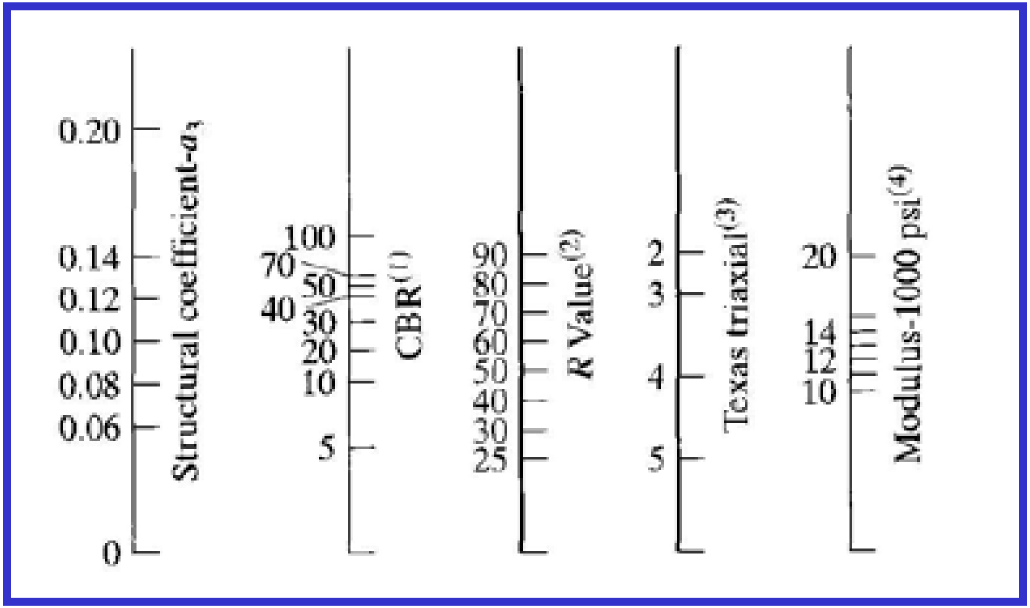
AASHTO Method-Design Variables-Structural Number-Layer coefficients



(c) Cement Treated

Flexible Pavement Design

AASHTO Method-Design Variables-Structural Number-Layer coefficients



AASHTO Method-Design Variables-Structural Number-Layer Coefficient

It is recommended that the layer coefficient be based on the resilient modulus, which is a more fundamental material property. The procedure for determining the resilient modulus of a particular material varies with its type. Except for the higher stiffness materials, such as HMA and stabilized bases, that may be tested by the repeated load indirect tensile test (ASTM D-4123), all materials should be tested by the resilient modulus test methods (AASHTO T274).

In following the AASHTO design guide, the notation M_R, as used herein, refers only to roadbed soils, whereas E_1, E_2 and E_3 apply to the HMA, base and subbase, respectively.

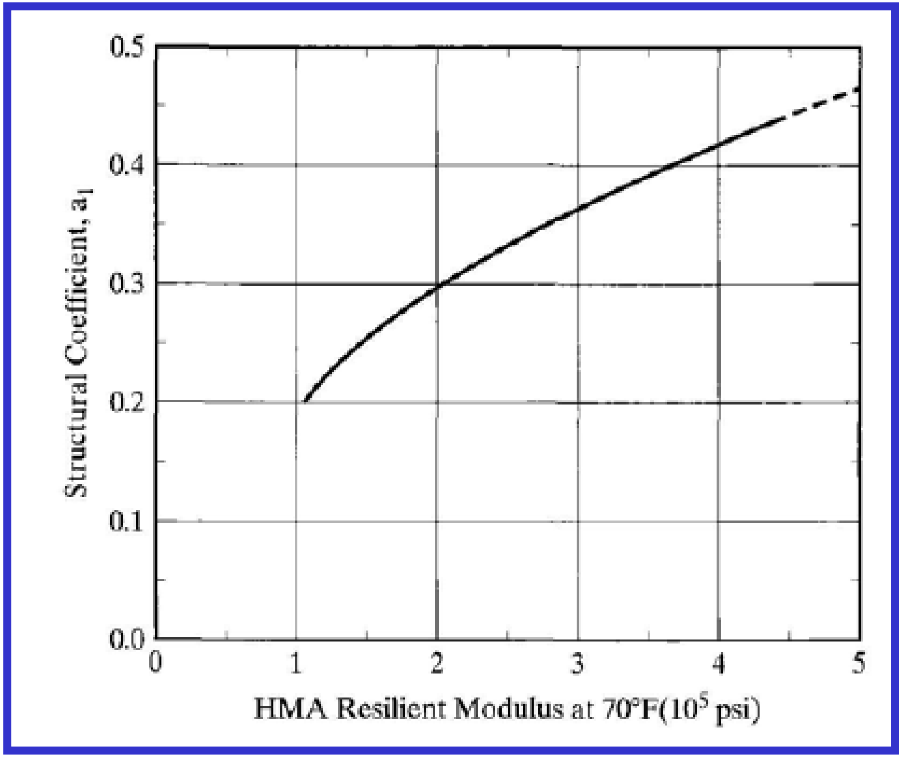
AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Asphalt-Concrete Surface Course

Figure is a chart relating the layer coefficient of a dense-graded HMA to its resilient modulus at 70°F (21°C). Caution should be used in selecting layer coefficients with modulus values greater than 450,000 psi (3.1 GPa), because the use of these larger moduli is accompanied by increased susceptiblity to thermal and fatigue cracking. The layer coefficient a_1 for the dense-graded HMA used in the AASHO Road Tests is 0.44, which corresponds to a resilient modulus of 450,000 psi (3.1 GPa).

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Asphalt-Concrete Surface Course



AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Untreated and Stabilized Base Courses

Figures (Already shown) show the charts that can be used to estimate the layer coefficient a_2 for untreated, bituminous-treated, and cement-treated base courses.

In lieu of Figures, the following equation can also be used to estimate a_2 for an untreated base course from its resilient modulus E_2:

$$a_2 = 0.249(\log E_2)-0.977$$

The layer coefficient a_2 for the granular base material used in the AASHO Road Test is 0.14, which corresponds to a base resilient modulus of 30,000 psi (207 GPa).

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Untreated and Stabilized Base Courses

The resilient modulus of untreated granular materials depends on the stress state θ, as under:

$$E_2 = K_1 \theta^{K2}$$

Typical values of K_1 for base materials range from 3000 to 8000; those of K_2 range from 0.5 to 0.7.

Values of K_1 and K_2 for each specific base material should be determined using AASHTO Method T274. In the absence of this information, the values shown in Table 11.16 can be used.

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Untreated and Stabilized Base Courses

TABLE 11.16 Typical Values of K_1 and K_2 for Untreated Base Materials

Moisture condition	K_1	K_2
Dry	6000–10,000	0.5–0.7
Damp	4000–6000	0.5–0.7
Wet	2000–4000	0.5–0.7

Source. After AASHTO (1986).

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Untreated and Stabilized Base Courses

The resilient modulus of the base course is a function not only of K_1 and K_2 , but also of the stress state θ. Values for the stress state within the base course vary with the roadbed soil resilient modulus and with the thickness of the surface layer. Typical values of θ are shown in Table 11.17. Given K_1, K_2 and θ, E_2 can be determined from the above equation.

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Untreated and Stabilized Base Courses

TABLE 11.17 Typical Values of Stress State θ for Base Course

Asphalt concrete thickness (in.)	Roadbed soil resilient modulus (psi)		
	3000	7500	15,000
Less than 2	20	25	30
2–4	10	15	20
4–6	5	10	15
Greater than 6	5	5	5

Note. Unit of θ is in psi, 1 in. = 25.4 mm, 1 psi = 6.9 kPa.
Source. After AASHTO (1986).

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Granular Subbase Course

Figure (Already shown) provides the chart that may be used to estimate layer coefficient a_3 of granular subbase courses. The relationship between a_3 and E_3 can be expressed as:

$$a_3 = 0.227(\log E_3)-0.839$$

The layer coefficient a_3 for the granular subbase in the AASHO Road Test is 0.11, which corresponds to a resilient modulus of 15,000 psi (104 MPa).

As with granular base courses, values of K_1 and K_2 for granular subbase courses can be determined from the resilient modulus test (AASHTO T274) or estimated from Table 11.18.

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Granular Subbase Course

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).

AASHTO Method-Design Variables-Structural Number-Layer Coefficient

Granular Subbase Course

Values of K_1, K_2 θ and E_3 for the subbase in the AASHO Road Test are shown in Table 11.19.

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

AASHTO Method-Design Variables-Structural Number-Drainage Coefficient

Depending on the quality of drainage and the availability of moisture, drainage coefficients m_2 and m_3 should be applied to granular bases and subbases to modify the layer coefficients. At the AASHTO Road Test site, these drainage coefficients are all equal to 1.

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.

Flexible Pavement Design

AASHTO Method-Design Variables-Structural Number-Drainage Coefficient

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).

AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

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 above equation, will be greater than the required SN.

Note that above equation 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 to avoid the possibility of producing an impractical design.

AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

From a cost-effective viewpoint, if the ratio of costs for HMA and granular base is less than the corresponding ratio of layer coefficients times the drainage coefficient, then the optimum economical design is to use a minimum base thickness by increasing the HMA thickness.

Minimum Thickness

It is generally impractical and uneconomical to use layers of material that are less than some minimum thickness. Furthermore, traffic considerations may dictate the use of a certain minimum thickness for stability. Table 11.21 shows the minimum thicknesses of asphalt surface and aggregate base. Because such minimums depend somewhat on local practices and conditions, they may be changed if needed.

Flexible Pavement Design

AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

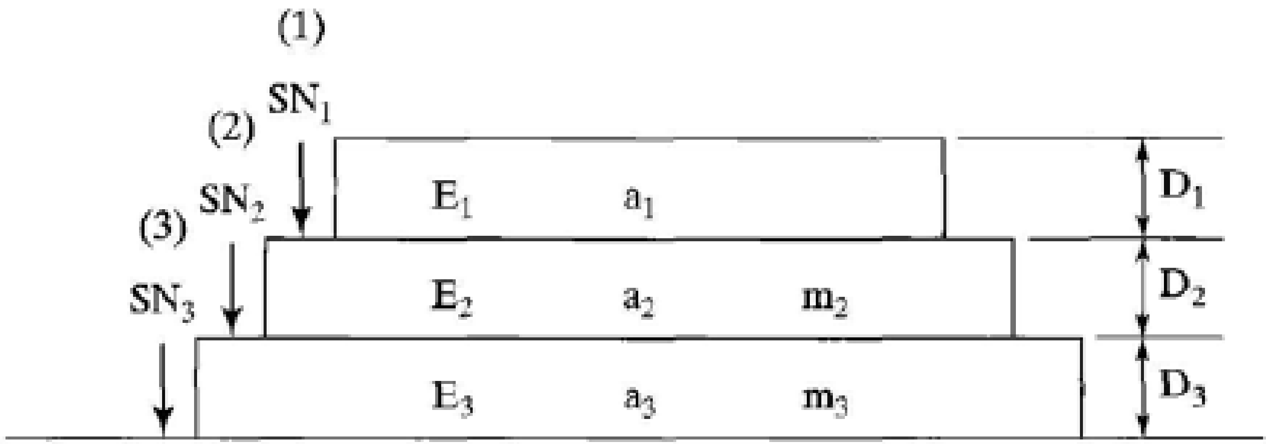
TABLE 11.21 Minimum Thickness for Asphalt Surface and Aggregate Base

Traffic (ESAL)	Asphalt concrete	Aggregate base
Less than 50,000	1.0	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

Note. Minimum thickness is in in.; 1 in. = 25.4 mm.
Source. After AASHTO (1986).

AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

General Procedure



AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

General Procedure

The procedure for thickness design is usually started from the top, as shown in Figure and described as follows:

1. **Using E_2 as M_R, determine from nomograph the structural number SN_1 required to protect the base and compute the thickness of layer 1 from**

$$D_1 \geq \frac{SN_1}{a_1}$$

2. **Using E_3 as M_R, determine from nomograph the structural number SN_2 required to protect the subbase and compute the thickness of layer 2 from**

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2}$$

AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

General Procedure

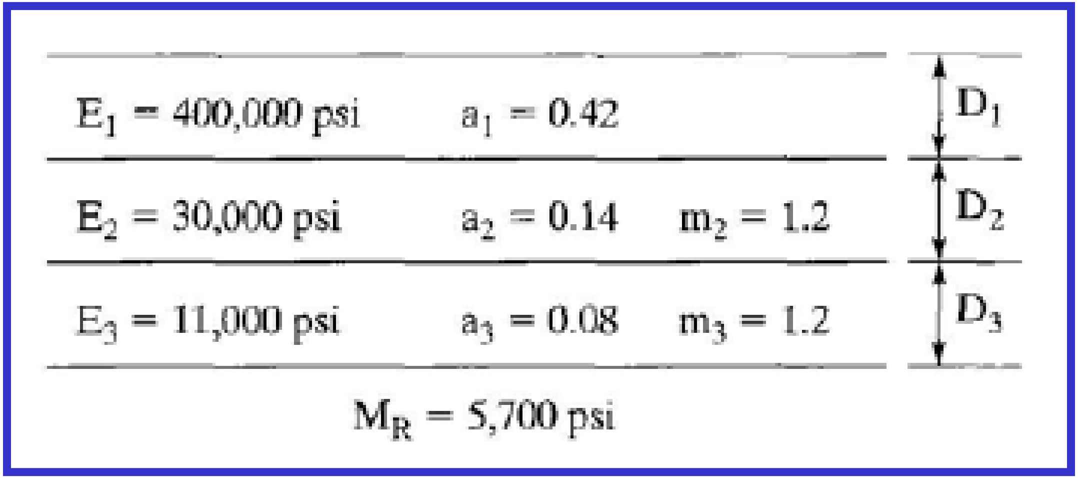
3. Based on the roadbed soil resilient modulus M_R, determine from nomograph the total structural number SN_3 required and compute the thickness of layer 3 from

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 D_2 m_2}{a_3 m_3}$$

AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

Numerical problem

Figure is a pavement system with the resilient moduli, layer coefficients, and drainage coefficients as shown. If predicted ESAL = 18.6×10^6, R = 95%, S_o = 0.35 and ΔPSI = 2.1, select thicknesses D_1, D_2 and D_3.



AASHTO Method-Design Variables-Structural Number-Selection of Layer Thickness

Numerical problem

Solution: With $M_R = E_2 = 30{,}000$ psi (207 MPa), from Figure 11.25, $SN_1 = 3.2$; from Eq. 11.47, $D_1 \geq 3.2/0.42 = 7.6$ in. (193 mm); use $D_1 = 8$ in. (203 mm).

With $M_R = E_3 = 11{,}000$ psi (76 MPa), from Figure 11.25, $SN_2 = 4.5$; from Eq. 11.48, $D_2 \geq (4.5 - 0.42 \times 8)/(0.14 \times 1.2) = 6.8$ in. (173 mm); use $D_2 = 7$ in. (178 mm). Note that a surface thickness of 8 in. (203 mm) and a base thickness of 7 in. (178 mm) meet the minimum thicknesses shown in Table 11.21.

With $M_R = 5700$ psi (39.3 MPa), from Figure 11.25, $SN_3 = 5.6$; from Eq. 11.49, $D_3 \geq (5.6 - 0.42 \times 8 - 0.14 \times 7 \times 1.20)/(0.08 \times 1.2) = 11.1$ in. (282 mm); use $D_3 = 11.5$ in. (292 mm).

$$D_1 \geq \frac{SN_1}{a_1}$$

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2}$$

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 D_2 m_2}{a_3 m_3}$$

AASHTO Method-Stage Construction

If the maximum performance period is less than the analysis period, any initial structure selected will require an overlay to last out the analysis period.

The thickest recommended initial structure is that corresponding to the maximum performance period.

Thinner initial structures, selected for the purpose of life cycle cost analyses, will result in shorter performance periods and require thicker overlays to last out the same analysis period.

The design of the initial structure for stage construction works the same as that for new construction, except that the reliability must be compounded over all stages.

AASHTO Method-Stage Construction

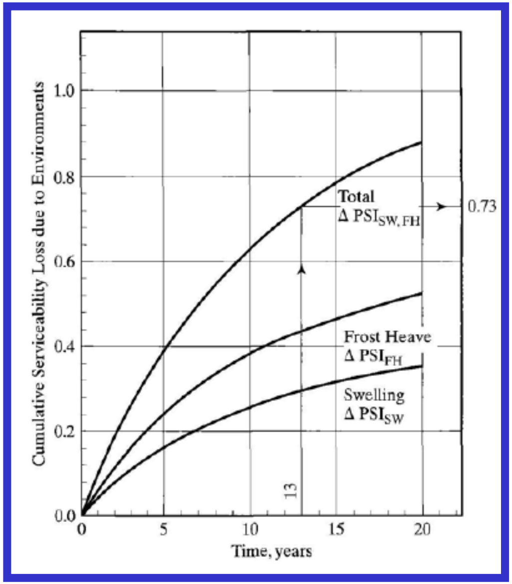
If the loss of serviceability is caused by traffic loads alone, the length of the performance period, which is related to W_{18}, for a given serviceability loss can be determined from nomograph.

However, if the serviceability loss is caused by both traffic loads and the environmental effects of roadbed swelling and frost heave, the performance period for a given terminal serviceability can be determined only by an iterative process, as illustrated by the following example.

Flexible Pavement Design
AASHTO Method-Stage Construction-Numerical problem

Given the following design inputs, determine the length of the performance period required: structural number SN = 5.0, reliability R = 95%, standard deviation S_0 = 0.35, initial serviceability p_0 = 4.3, terminal serviceability p_t = 2.5, effective roadbed soil resilient modulus M_R = 5000 psi (35 MPa), ΔPSI due to both swelling and frost heave as shown in Figure 11.23, and traffic versus time relationship as

$$Y = 77.9 \log\left(\frac{W_{18}}{10 \times 10^6} + 1\right)$$



Flexible Pavement Design

AASHTO Method-Stage Construction-Numerical problem

$$Y = 77.9 \log\left(\frac{W_{18}}{10 \times 10^6} + 1\right)$$

Solution: First, assume $Y = 13$ years. From Figure 11.23, ΔPSI due to environmental effects = 0.73; ΔPSI due to traffic = 4.3 − 2.5 − 0.73 = 1.07. From Eq. 11.37 or Figure 11.25, $W_{18} = 1.6 \times 10^6$. From Eq. 11.51, $Y = 5.1$ years, which is much smaller than the 13 years assumed.

$$\boxed{\begin{aligned}\log W_{18} = {} & Z_R S_0 + 9.36 \log(\text{SN} + 1) - 0.20 + \frac{\log[\Delta \text{PSI}/(4.2 - 1.5)]}{0.4 + 1094/(\text{SN} + 1)^{5.19}} \\ & + 2.32 \log M_R - 8.07\end{aligned}}$$

Next assume Y as the average of 13 and 5.1 years, or 9.0 years. From Figure 11.23, ΔPSI due to environmental effects = 0.59; ΔPSI due to traffic = 4.3 − 2.5 − 0.59 = 1.21. From Eq. 11.37 or Figure 11.25, $W_{18} = 2.1 \times 10^6$. From Eq. 11.51, $Y = 6.5$ years.

Finally, assume $Y = (9 + 6.5)/2 = 7.7$ years. From Figure 11.23, ΔPSI due to environmental effects = 0.52; ΔPSI due to traffic = 4.3 − 2.5 − 0.52 = 1.28. From Eq. 11.37 or Figure 11.25, $W_{18} = 2.4 \times 10^6$. From Eq. 11.51, $Y = 7.3$, which is nearly equal to the 7.7 years assumed. When the difference between the assumed and calculated values is smaller than 1 year, no more iterations are needed and the average of the two values can be used as the performance period. Therefore, the performance period = (7.7 + 7.3)/2 = 7.5 years.

Shoulder Design-Prediction of Traffic

The factors affecting shoulder design are similar to those affecting mainline pavement design. The major difference is the amount of traffic. Traffic volume on shoulders is lower than on mainline pavements and more difficult to predict.

Three types of traffic may be considered in shoulder design: encroaching traffic, parking traffic and regular traffic. Regular traffic is considered only if the use of shoulder as an additional lane for peak hour or detoured traffic is anticipated. If there is no regular traffic, the sum of encroaching and parking traffic is used to design the inner edge of shoulder adjacent to the mainline pavement, and parking traffic is used to design the outer edge of shoulder. If a uniform thickness is used for shoulders, only the inner edge need be considered.

Shoulder Design-Prediction of Traffic

However, this may not be true for rigid pavement shoulders, because the stresses and deflections at the outer edge are much greater than those at the inner edge and may cause more damage even though the traffic volume is smaller.

Encroaching Traffic

When there is a paved shoulder and no lateral obstruction within the shoulder area, trucks using the outer traffic lane tend to encroach on the shoulder as much as 12 in. and sometimes even more. In California (1972), shoulder sections are designed for 1% of the mainline traffic in the adjacent lane with a minimum traffic index of 5, which corresponds to approximately 10^4 equivalent 18-kip single-axle load applications.

Shoulder Design-Prediction of Traffic-Encroaching Traffic

Results of a study in Georgia (Emery, 1975) showed that the use of 1% is low for some traffic flow conditions. In the absence of additional data, Barksdale and Hicks (1979) recommended that, for free flow traffic conditions in rural areas of the south, the inner edge of the shoulder should be designed for at least 2 to 2.5% of the truck traffic on the outer lane. Because local conditions vary significantly, the best way to determine the percentage of encroaching traffic is to make an actual survey on a segment of highway with paved shoulders, which has the same traffic, geometric, and topographic conditions as the design case in question.

Shoulder Design-Prediction of Traffic-Encroaching Traffic

For the study performed in Georgia, trucks were selected at random and followed by observers for 10 miles. Those trucks not completing the full 10-mile trip were dropped from the analysis. The longitudinal distance for each encroachment is estimated from the prevailing speed and the time during which the truck encroaches on the shoulder. The percent encroaching traffic, which is the ratio of load applications on the shoulder to those on the adjacent lane, can be computed by:

$$P_e = \frac{N_e L_e}{N_o L_o} \times 100$$

Shoulder Design-Prediction of Traffic-Encroaching Traffic

$$P_e = \frac{N_e L_e}{N_o L_o} \times 100$$

in which P_e is the percent encroaching traffic,
N_e is the total number of encroachments per day,
L_e is the average length of each encroachment,
N_o is the number of load applications per day on the outside lane, and
L_o is the length of observed distance, such as 10 miles used in the Georgia study.
Note that $N_e L_e$ is the total length of encroachment for all trucks and $N_e L_e / N_o$ is the length of encroachment per truck, so P_e can also be defined as the length of encroachment per truck within an observed distance L_o.

Field observations indicate that P_e usually varies from 1 to 8% of the traffic volume on the adjacent lane. The percentage of parking traffic should be added to P_e, because any truck must encroach to park on the shoulder.

Shoulder Design-Prediction of Traffic-Encroaching Traffic-Numerical problem

Given the number of trucks on the outside lane per day N_o = *2239*, the number of shoulder encroachments in the 10 mile stretch N_e = *7389*, and the average distance of each encroachment L_e=*384ft*, determine the percent encroachment.

$$P_e = \frac{N_e\, L_e}{N_o\, L_o} \times 100$$

$$P_e = [7389 \times 384 / 2239 \times 52800] \times 100 = 2.4\%$$

Shoulder Design-Prediction of Traffic- Parking Traffic

Parking traffic is the number of load applications for trucks that park on the shoulder for emergencies or other purposes. This information can also be estimated for the design section by using traffic counts on an existing pavement with similar traffic and design characteristics. Parking traffic usually varies greatly along a given route, depending on geometric and interchange conditions. Because most trucks park near interchange ramps, it may be necessary to identify separate design sections for areas where parking is likely and for areas where minimum parking is expected. The parking survey should last at least one day and cover the early morning hours, during which more parking usually takes place.

Shoulder Design-Prediction of Traffic- Parking Traffic

Similar to percent encroaching traffic, the percent parking traffic can be computed by:

$$P_p = \frac{N_p L_p}{N_o L_o} \times 100$$

in which P_p is the percent parking traffic, which is the ratio between parking traffic and the traffic on the outside lane:

N_p is the number of parked trucks per day;

L_p is the average distance the trucks drive on the shoulder during a typical stop, which can be determined from a field survey; N_o is the number of trucks traveling on the outside lane per day; and

L_o is the length of segment for the parking survey.

Field observations indicate that P_p may range from 0.0005 to 0.02 percent. Note that the percent parking traffic is much smaller than the percent encroaching traffic and can usually be neglected.

Shoulder Design-Prediction of Traffic- Parking Traffic-Numerical problem

The average number of trucks that might park on a 2-mile stretch of shoulder during one day could range from 1 to 25. If the number of trucks on the outside lane is 2951 per day and a truck drives an average of 200 ft during each parking maneuver, determine the range of percent parking traffic.

$$P_p = \frac{N_p L_p}{N_o L_o} \times 100$$

For $N_p = 1$

$P_p = [1 \times 200 / 2951 \times 2 \times 5280] \times 100 = 0.00064\%$

For $N_p = 25$

$P_p = 25 \times 0.00064 = 0.016\%$

Range of P_p is 0.00064 to 0.016%.

Shoulder Design-Prediction of Traffic- Regular Traffic

If it is anticipated that the shoulder will be used by regular traffic at any stage of its design life, this additional traffic should be added to the encroaching and parking traffic to form the total shoulder design traffic. The ultimate design is to consider the shoulder as an extra lane with the same traffic and cross section as that of the mainline outer lane.