Al-Mustansiriyah University

College of Engineering

HWY and Transportation Department

Structural Design of Highway

Third Stage

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2016

References:

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Types of pavements

- Hot Mix Asphalt (HMA) for asphalt pavements
- Portland cement concrete (PCC) pavements.

FUNCTIONS

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil,
- Structurally strong to withstand all types of stresses imposed upon it,
- Adequate coefficient of friction to prevent skidding of vehicles,
- Smooth surface to provide comfort to road users even at high speed,
- Impervious surface, so that sub-grade soil is well protected, and
- Long design life with low maintenance cost.

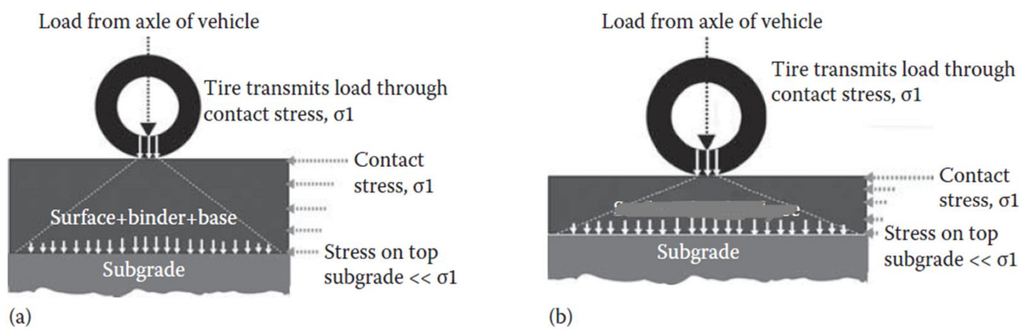


FIGURE 1.2 Function of the pavement is to decrease the tire contact stress on the subgrade to a tolerable level; flexible pavement (a), rigid pavement (b).

LOAD DISTRIBUTION CONCEPT

- The load (from a vehicle) is transferred to the pavement through load-bearing axles and pressurized tires. The resulting pressure or stress on the pavement, at any depth, is dependent on many factors, such as total load, the number of axles and tires, and the condition of the tires.
- The stress on the surface of the pavement gets distributed in an inverted V form from the surface downward. In other words, the stress intensity decreases along the depth of the pavement.

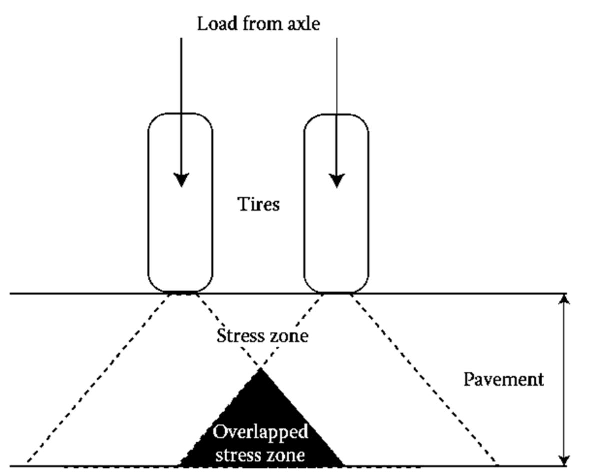


FIGURE 2.2 Concept of distribution of stress in the inverted V form.

STRUCTURAL COMPONENTS OF A FLEXIBLE PAVEMENT

Subgrade (Prepared Road Bed)

The subgrade is usually the natural material located along the horizontal alignment of the pavement and serves as the foundation of the pavement structure. It also may consist of a layer of selected borrow materials, well compacted to prescribed specifications. It may be necessary to treat the subgrade material to achieve certain strength properties required for the type of pavement being constructed.

Subbase Course

Located immediately above the subgrade, the subbase component consists of material of a superior quality to that which is generally used for subgrade construction. The requirements for subbase materials usually are given in terms of the gradation and strength. In cases where suitable subbase material is not readily available, the available material can be treated with other materials to achieve the necessary properties. This process of treating soils to improve their engineering properties is known as stabilization.

Base Course

The base course lies immediately above the subbase. It is placed immediately above the subgrade if a subbase course is not used. This course usually consists of granular materials such as crushed stone, crushed or uncrushed gravel, and sand. The specifications for base course materials usually include more strict requirements than those for subbase materials, particularly with respect to their gradation, and strength. Materials that do not have the required properties can be used as base materials if they are properly stabilized with Portland cement, asphalt, or lime.

Surface course

Consists of a mixture of mineral aggregates and asphalt, It should be capable of withstanding high tire pressures, resisting abrasive forces due to traffic, providing a skid resistant driving surface, and preventing the penetration of surface water into the underlying layers. The thickness of the wearing surface can vary from 3 in. to more than 6 in., depending on the expected traffic on the pavement. It was shown that the quality of the surface course of a flexible pavement depends on the mix design of the asphalt concrete used.

Properties of Highway Materials:

Soil

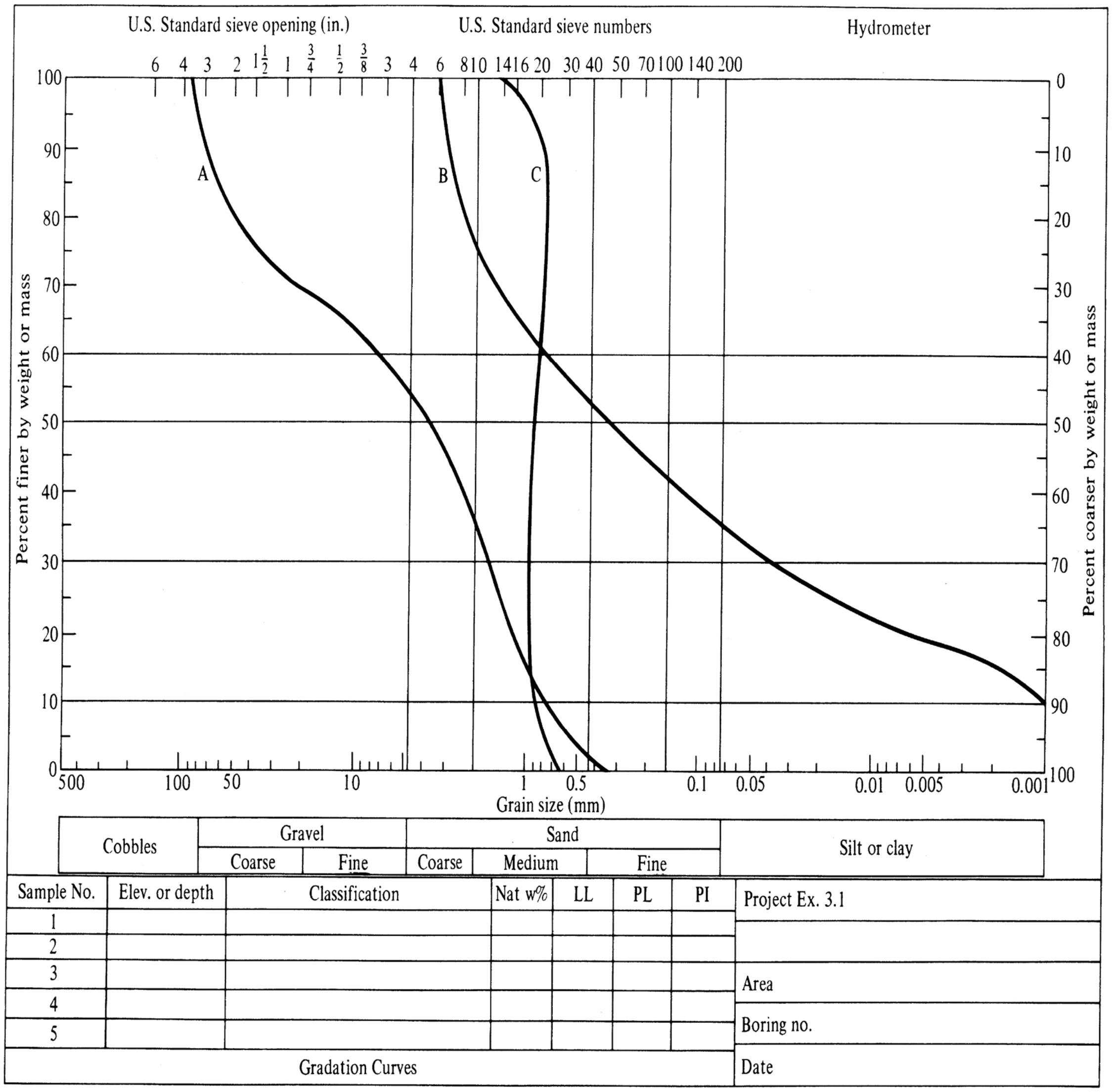
SOIL CHARACTERISTICS

The distribution of particle size in soils can be determined by conducting a sieve analysis (sometimes known as mechanical analysis) on a soil sample if the particles are sufficiently large. This is done by shaking a sample of air-dried soil through a set of sieves with progressively smaller openings. The smallest practical opening of these sieves is 0.075 mm; this sieve is designated No. 200. Other sieves include

No. 140 (0.106 mm), No. 100 (0.15 mm), No. 60 (0.25 mm), No. 40 (0.425 mm), No. 20 (0.85 mm), No. 10 (2.0 mm), No. 4 (4.75 mm).

- Gravel : > 2 mm
- sand size: 2.0-0.06 mm
- silt : 0.06-0.002
- clay: less than 0.002

For soils containing particle sizes smaller than the lower limit, the hydrometer analysis is used.



Atterberg Limits

Clay soils with very low moisture content will be in the form of solids. As the water content increases, however, the solid soil gradually becomes plastic—that is, the soil easily can be molded into different shapes without breaking up.

Continuous increase of the water content will eventually bring the soil to a state where it can flow as a viscous liquid. The stiffness or consistency of the soil at any time therefore depends on the state at which the soil is, which in turn depends on the amount of water present in the soil.

The water content levels at which the soil changes from one state to the other is the Atterberg limits. They are the shrinkage limit (SL), plastic limit (PL), and liquid limit (LL), as illustrated in Figure below.

Atterberg limits are important limits of engineering behavior, because they facilitate the comparison of the water content of the soil with those at which the soil changes from one state to another. They are used in the classification of fine-grained soils and are extremely useful, since they correlate with the engineering behaviors of such soils.

Shrinkage Limit (SL)

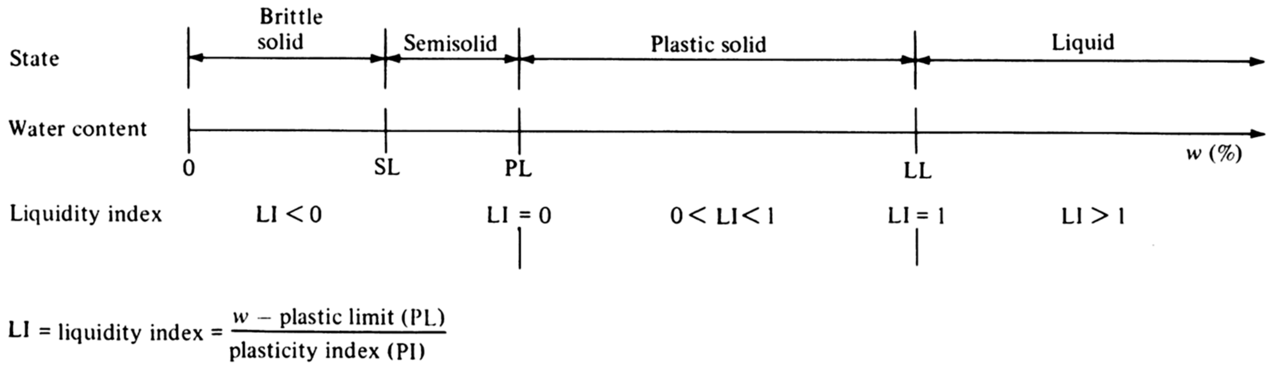
When a saturated soil is slowly dried, the volume shrinks, but the soil continues to contain moisture. Continuous drying of the soil, however, will lead to moisture content at which further drying will not result in additional shrinkage. The volume of the soil will stay constant, and further drying will be accompanied by air entering the voids. The moisture content at which this occurs is the shrinkage limit, or SL, of the soil.

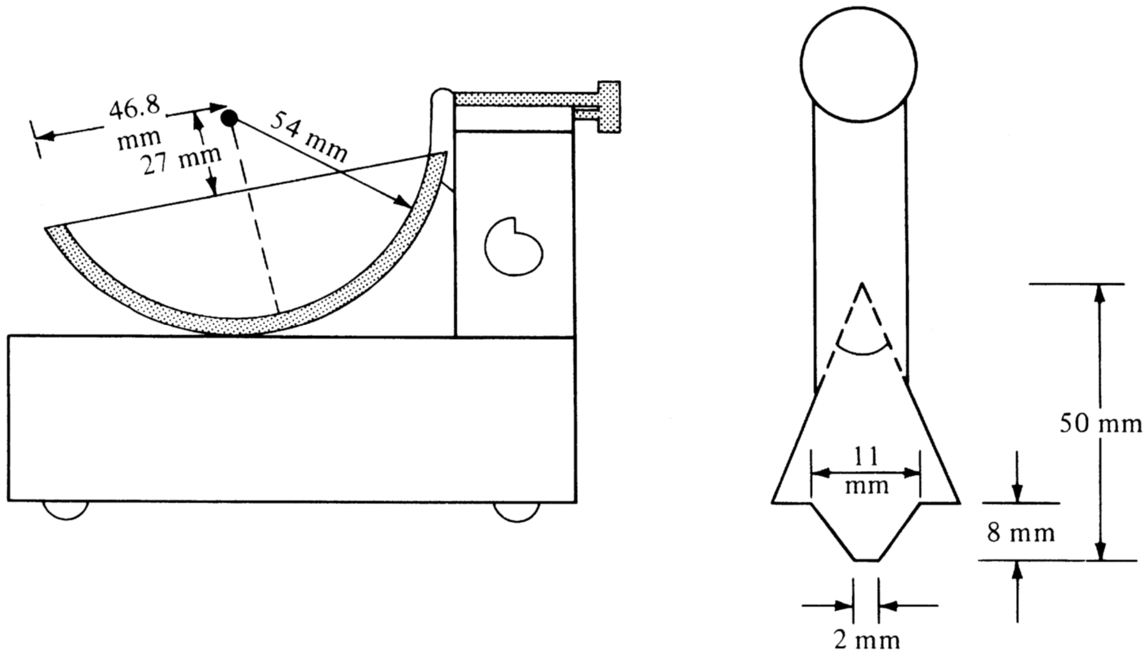
Plastic Limit (PL)

The plastic limit, or PL, is defined as the moisture content at which the soil crumbles when it is rolled down to a diameter of one-eighth of an inch. The moisture content is higher than the PL if the soil can be rolled down to diameters less than one-eighth of an inch, and the moisture content is lower than the PL if the soil crumbles before it can be rolled to one-eighth of an inch diameter.

Liquid Limit (LL)

The liquid limit, or LL, is defined as the moisture content at which the soil will flow and close a groove of one-half inch within it after the standard LL equipment has been dropped 25 times. The equipment used for LL determination is shown in Figure below.





This device was developed by Casagrande, who worked to standardize the Atterberg limits tests. It is difficult in practice to obtain the exact moisture content at which the groove will close at exactly 25 blows. The test is therefore conducted for different moisture contents and the number of blows required to close the groove for each moisture content recorded. A graph of moisture content versus the logarithm of the number of blows (usually a straight line known as the flow curve) is then drawn. The moisture content at which the flow curve crosses 25 blows is the LL.

The range of moisture content over which the soil is in the plastic state is the difference between the LL and the PL and is known as the plasticity index (PI).

$$PI = LL - PL$$

where

PI = plasticity index
LL = liquid limit
PL = plastic limit

CLASSIFICATION OF SOILS FOR HIGHWAY USE

The most commonly used classification system for highway purposes is

- The American Association of State Highway and Transportation Officials (AASHTO) Classification System.
- The Unified Soil Classification System (USCS)

AASHTO Soil Classification System

The system has been described by AASHTO as a means for determining the relative quality of soils for use in embankments, subgrades, subbases, and bases.

Soils are classified into seven groups, A-1 through A-7, with several subgroups, as shown in Table 17.1. The classification of a given soil is based on its particle size distribution, LL, and PI. Soils are evaluated within each group by using an empirical formula to determine the group index (GI) of the soils, given as

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10) \qquad (17.18)$$

where

GI = group index
F = percent of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve
LL = liquid limit expressed in whole number
PI = plasticity index expressed in whole number

The GI is determined to the nearest whole number. A value of zero should be recorded when a negative value is obtained for the GI. Also, in determining the GI for A-2-6 and A-2-7 subgroups, the LL part of Eq. 17.18 is not used—that is, only the second term of the equation is used.

Under the AASHTO system, granular soils fall into classes A-1 to A-3. A-1 soils consist of well-graded granular materials, A-2 soils contain significant amounts of silts and clays, and A-3 soils are clean but poorly graded sands.

Classifying soils under the AASHTO system will consist of first determining the particle size distribution and Atterberg limits of the soil and then reading Table 17.1 from left to right to find the correct group. The correct group is the first one from the left that fits the particle size distribution and Atterberg limits and should be expressed in terms of group designation and the GI. Examples are A-2-6(4) and A-6(10).

In general, the suitability of a soil deposit for use in highway construction can be summarized as follows.

1. Soils classified as A-1-a, A-1-b, A-2-4, A-2-5, and A-3 can be used satisfactorily as subgrade or subbase material if properly drained. In addition, such soils must be properly compacted and covered with an adequate thickness of pavement (base and/or surface cover) for the surface load to be carried.

2. Materials classified as A-2-6, A-2-7, A-4, A-5, A-6, A-7-5, and A-7-6 will require a layer of subbase material if used as subgrade.

3. When soils are properly drained and compacted, their value as subgrade material decreases as the GI increases. For example, a soil with a GI of zero (an indication of a good subgrade material) will be better as a subgrade material than one with a GI of 20 (an indication of a poor subgrade material).

Group Index (GI)	Subgrade Rating
0	Excellent
0–1	Good
2–4	Fair
5–9	Poor
10–20	Very poor

Table 17.1 AASHTO Classification of Soils and Soil Aggregate Mixtures

General Classification	*Granular Materials (35% or Less Passing No. 200)*							*Silt-Clay Materials (More than 35% Passing No. 200)*			
	A-1			*A-2*							*A-7*
Group Classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*	*A-4*	*A-5*	*A-6*	*A-7-5, A-7-6*
Sieve analysis											
Percent passing											
No. 10	−50 max.	–	–	–	–	–	–	–	–	–	
No. 40	30 max.	50 max.	51 min.	–	–	–	–	–	–	–	–
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40:											
Liquid limit	–		–	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

*Plasticity index of A-7-5 subgroup ≤ LL − 30. Plasticity index of A-7-6 subgroup > LL − 30.

SOURCE: Adapted from *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 27th ed., Washington, D.C., The American Association of State Highway and Transportation Officials, copyright 2007. Used with permission.

Example 17.3 Classifying a Soil Sample Using the AASHTO Method

The following data were obtained for a soil sample.

Mechanical Analysis

Sieve No.	*Percent Finer*	*Plasticity Tests:*
4	97	LL = 48%
10	93	PL = 26%
40	88	
100	78	
200	70	

Using the AASHTO method for classifying soils, determine the classification of the soil and state whether this material is suitable in its natural state for use as a subbase material.

Solution:

- Since more than 35% of the material passes the No. 200 sieve, the soil is either A-4, A-5, A-6, or A-7.
- LL > 40%, and therefore the soil cannot be in group A-4 or A-6. Thus, it is either A-5 or A-7.
- The PI is 22% (48 – 26), which is greater than 10%, thus eliminating group A-5. The soil is A-7-5 or A-7-6.
- (LL − 30) = 18 < PI (22%). Therefore the soil is A-7-6, since the plasticity index of A-7-5 soil subgroup is less than (LL − 30). The GI is given as:

$$(70 - 35)[0.2 + 0.005(48 - 40)] + 0.01(70 - 15)(22 - 10) = 8.4 + 6.6 = 15$$

The soil is A-7-6 (15) and is therefore unsuitable as a subbase material in its natural state.

Field Compaction Equipment

Compaction equipment used in the field can be divided to the following types:

- **Rollers** are used for field compaction and apply either a vibrating force or an impact force on the soil. The type of roller used for any particular job depends on the type of soil to be compacted.
- **A smooth wheel or drum roller** applies contact pressure of up to 55 lb/in.2 over 100 percent of the soil area in contact with the wheel. This type of roller is generally used for finish rolling of subgrade material and can be used for all types of soil material except rocky soils.
- **The rubber-tired roller** is another type of contact roller, consisting of a heavily loaded wagon with rows of 3 to 6 tires placed close to each other.

The pressure in the tires may be up to 100 lb/in^2. They are used for both granular and cohesive materials.

- **Sheep foot roller**, this roller has a drum wheel that can be filled with water. The drum wheel has several protrusions, which may be round or rectangular in shape. Contact pressures ranging from 200 to 1000 lb/in^2 can be obtained from sheepsfoot rollers, depending on the size of the drum and whether or not it is filled with water. The sheepsfoot roller is used mainly for cohesive soils.
- **Tamping foot rollers** are similar to sheepsfoot rollers in that they also have protrusions that are used to obtain high contact pressures, ranging from 200 to 1200 lb/in.2. The feet of the tamping foot rollers are specially hinged to obtain a kneading action while compacting the soil. As with sheepsfoot rollers, tamping foot rollers compact from the bottom of the soil layer. Tamping foot rollers are used mainly

(a) Smooth wheel roller

(b) Rubber-tired roller

16 Typical Sheepsfoot Roller

FROST ACTION IN SOILS

When the ambient temperature falls below freezing for several days, it is quite likely that the water in soil pores will freeze. Since the volume of water increases by about 10 percent when it freezes, the first problem is the increase in volume of the soil. The second problem is that the freezing can cause ice crystals and lenses that are several centimeters thick to form in the soil. These two problems can result in heaving of the subgrade (frost heave), which may result in significant structural damage to the pavement.

SWELL POTENTIAL

Soils consisting of minerals such as montmorillonite would expand significantly in contact with water. The presence of such soils in the subgrade would lead to differential movement, surface roughness, and cracking in pavements. The presence of swelling soil can affect the performance of the pavement and deteriorate its serviceability.

SOIL STABILIZATION

Soil stabilization is the treatment of natural soil to improve its engineering properties. Soil stabilization methods can be divided into two categories, namely, mechanical and chemical.

Mechanical stabilization is the blending of different grades of soils to obtain a required grade.

Chemical stabilization is the blending of the natural soil with chemical agents. Several blending agents have been used to obtain different effects. The most commonly used agents are Portland cement, asphalt binders, and lime.

Cement Stabilization

Cement stabilization of soils usually involves the addition of 5 to 14 percent Portland cement by volume of the compacted mixture to the soil being stabilized. This type of stabilization is used mainly to obtain the required engineering properties of soils that are to be used as base course materials. Although the best results have been obtained when well-graded granular materials were stabilized with cement, the Portland Cement Association has indicated that nearly all types of soil can be stabilized with cement.

The procedure for stabilizing soils with cement involves:

• Pulverizing the soil

• Mixing the required quantity of cement with the pulverized soil

• Compacting the soil cement mixture

• Curing the compacted layer

Asphalt Stabilization

Asphalt stabilization is carried out to achieve one or both of the following:

• Waterproofing of natural materials

• Binding of natural materials

Waterproofing the natural material through asphalt stabilization aids in maintaining the water content at a required level by providing a membrane that impedes the penetration of water, thereby reducing the effect of any surface water that may enter the soil when it is used as a base course. In addition, surface water is prevented from seeping into the subgrade, which protects the subgrade from failing due to increase in moisture content. Several types of soil can be stabilized with asphalt, although it is generally required that less than 25 percent of the material passes the No. 200 sieve. This is necessary because the smaller soil particles tend to have extremely large surface areas per unit volume and require a large amount of bituminous material for the soil surfaces to be adequately coated. It is also necessary to use soils that have a plasticity index (PI) of less than 10, because difficulty may be encountered in mixing soils with a high PI.

Lime Stabilization

Lime stabilization is one of the oldest processes of improving the engineering properties of soils and can be used for stabilizing both base and subbase materials.

In general, the oxides and hydroxides of calcium and magnesium are considered as lime, clayey materials are most suitable for lime stabilization, but these materials should also have PI values less than 10 for the lime stabilization to be most effective.

SPECIAL TESTS FOR PAVEMENT DESIGN

1- California Bearing Ratio (CBR) Test

This test is commonly known as the CBR test and involves the determination of the load-deformation curve of the soil in the laboratory using the standard CBR testing. The test is conducted on samples of soil compacted to required standards and immersed in water for four days, during which time the samples are loaded with a surcharge that simulate the estimated weight of pavement material the soil will support. The objective of the test is to determine the relative strength of a soil with respect to crushed rock, which is considered an excellent coarse base material. This is obtained by conducting a penetration test on the samples still carrying the simulated load and using a standard CBR equipment. The CBR is defined as the penetration resistance of a subgrade soil relative to a standard crushed rock.

$$\text{CBR} = \frac{\text{(unit load for 0.1 piston penetration in test specimen) (lb/in}^2\text{.)}}{\text{(unit load for 0.1 piston penetration in standard crushed rock) (lb/in}^2\text{.)}} \qquad (17.24)$$

The unit load for 0.1 piston in standard crushed rock is usually taken as 1000 lb/in^2, which gives the CBR as

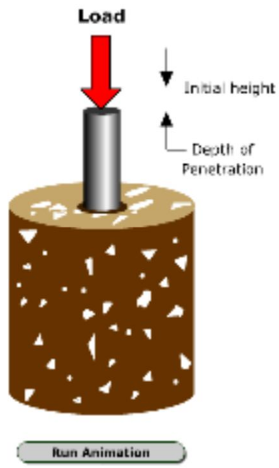
$$\text{CBR} = \frac{\text{(unit load for 0.1 piston penetration in test sample)}}{1000} \times 100 \qquad (17.25)$$

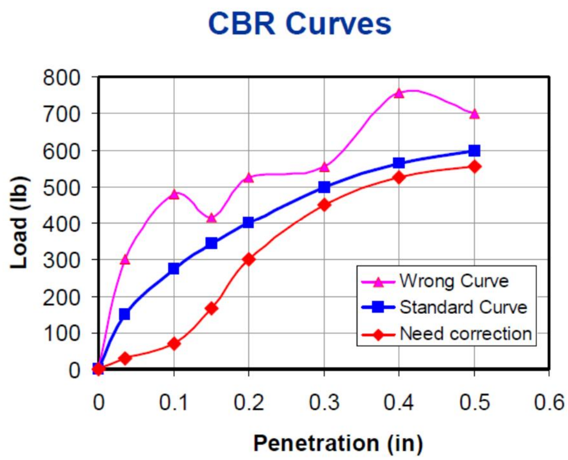
Load a piston (area = 3 in^2) at a constant rate (0.05 in/min)

• Record Load every 0.1 in penetration

• Total penetration not to exceed 0.5 in.

• Draw Load-Penetration Curve.





CBR Calculation

$$CBR = 100\left(\frac{\text{Load or Stress of Soil}}{\text{Load or Stress of Standard Rocks}}\right)$$

Loads and Stresses Corresponding to 0.1 and 0.2 inches Penetration for the Standard Rocks

Penetration	0.1" (2.5 mm)	0.2" (5.0 mm)
Load of Standard Rocks (lb)	3000	4500
Load of Standard Rocks (kN)	13.24	19.96
Stress of Standard Rocks (KPa)	6895	10342
Stress of Standard Rocks (psi)	1000	1500

Calculate CBR at 0.1 in (2.5 mm) and 0.2 in (5.0 mm) deformation then use the Maximum value as the design CBR.

2- Resistance Value (R-Value) ASTM D2844

The Resistance Value (R-value) is a test value, which measures the ability of a soil to resist lateral flow due to vertically applied load.

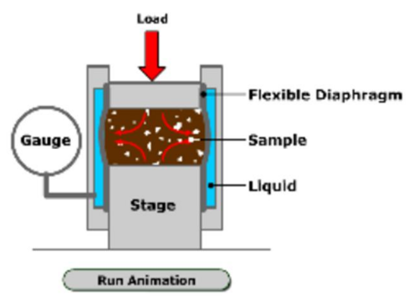
This test Developed by California Division of Highways in 1940s,

- Measures frictional resistance of granular material to deformation

- Uses the Hveem Stabilometer
- Tests material in a saturated condition (worst case scenario)

At the completion of the expansion test, the specimen is put into a flexible sleeve and placed in the stabilometer as shown in the figure. Vertical pressure is applied gradually on the specimen at a speed of 0.05 in./min until a pressure of 160 lb/in.2 is attained. The corresponding horizontal pressure is immediately recorded.

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_v}{P_h} - 1\right) + 1} \qquad (17.26)$$



where

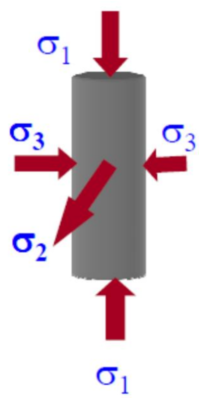
R = resistance value
P_v = vertical pressure (160 lb/in.^{2})
P_h = horizontal pressure at P_v of 160 lb/in.^{2} (lb/in.^{2})
D = number of turns of displacement pump

3- Resilient Modulus (MR)

The Resilient Modulus (MR) is a measure of subgrade material stiffness. A material's resilient modulus is actually an estimate of its modulus of elasticity (E). While the modulus of elasticity is stress divided by strain for a slowly applied load, resilient modulus is stress divided by strain for rapidly applied loads – like those experienced by pavements.
MR is ability of material to absorb energy within the elastic range.
Resilient modulus is determined using the triaxial test. The test applies a repeated axial cyclic stress of fixed magnitude, load duration and cycle duration 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. It is essentially a cyclic version of a triaxial compression test; the cyclic load application is thought to more accurately simulate actual traffic loading.

Resilient modulus test can be conducted on all types of pavement materials ranging from cohesive to stabilized materials. The test is conducted in a triaxial device equipped for repetitive load conditions.

- Measures "stiffness" of the material under repeated load.
- Determines the load carrying capacity of the material.
- Used for HMA as well as unbound materials
- Uses a repeated load triaxial test.
- Used in most modern methods of pavement design.



$$M_R = \frac{\text{Deviator stress}}{\text{Recoverable strain}} = \frac{\sigma_1 - \sigma_3}{\varepsilon_r}$$

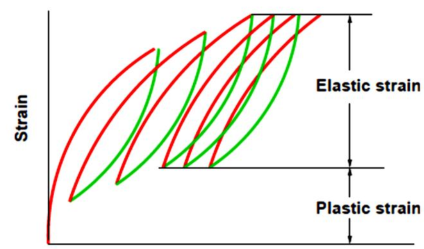


Figure 27:2: Recoverable strain under repeated loads

$$M_R = 1500(CBR)$$

Origin: Heukelom and Klomp (1962)
Limitation: Fine-grained non-expansive soils with soaked CBR ≤ 10

$$M_R = 2555(CBR)^{0.64}$$

Origin: NCHRP 1-37A – Mechanistic Design Guide
Limitation: No Limitation

Units for Both Models: CBR → % EX: 80% use 80

$$\text{R} - \text{Value} = \frac{1500(CBR) - 1155}{555}$$

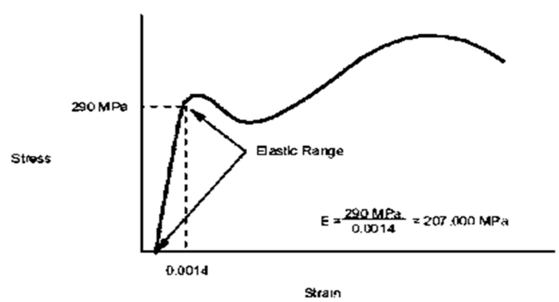
Origin: HDOT
Limitation: Fine-grained non-expansive soils with soaked CBR ≤ 8

$$M_R = 1000 + 555(\text{R Value})$$

Origin: 1993 AASHTO Guide
Limitation: Fine-grained non-expansive soils with R ≤ 20

Elastic modulus is sometimes called Young's modulus, An elastic modulus (E) can be determined for any solid material and represents a constant ratio of stress and strain (a stiffness): E = stress/ strain

A material is elastic if it is able to return to its original shape or size immediately after being stretched or squeezed. The modulus of elasticity for a material is basically the slope of its stress-strain plot within the elastic range (as shown in Figure 1).



It is important to remember that a measure of a material's modulus of elasticity is not a measure of strength. Strength is the stress needed to break or rupture a material, whereas elasticity is a measure of how well a material returns to its original shape and size.

4- Plate Loading Test

- Measure supporting power of subgrades, subases, bases and a complete pavement.
- Field test.

- Data from the test are applicable for design of both flexible and rigid pavements.
- Results might need some corrections.
- The test site is prepared and loose material is removed so that the 75 cm diameter plate rests horizontally in full contact with the soil sub-grade. The plate is seated accurately and then a seating load equivalent to a pressure of 0.07 kg/cm^2 (320 kg for 75 cm diameter plate) is applied and released after a few seconds. The settlement dial gauge is now set corresponding to zero load.
- A load is applied by means of jack, sufficient to cause an average settlement of about 0.25 cm. When there is no perceptible increase in settlement or when the rate of settlement is less than 0.025 mm per minute (in the case of soils with high moisture content or in clayey soils) the load dial reading and the settlement dial readings are noted.
- Deflection of the plate is measured by means of deflection dials; placed usually at one-third points of the plate near its outer edge.
- To minimize bending, a series of stacked plates should be used.
- Average of three or four settlement dial readings is taken as the settlement of the plate corresponding to the applied load. Load is then increased till the average settlement increase to a further amount of about 0.25 mm, and the load and average settlement readings are noted as before. The procedure is repeated till the settlement is about 1.75 mm or more.

Plate Loading Test

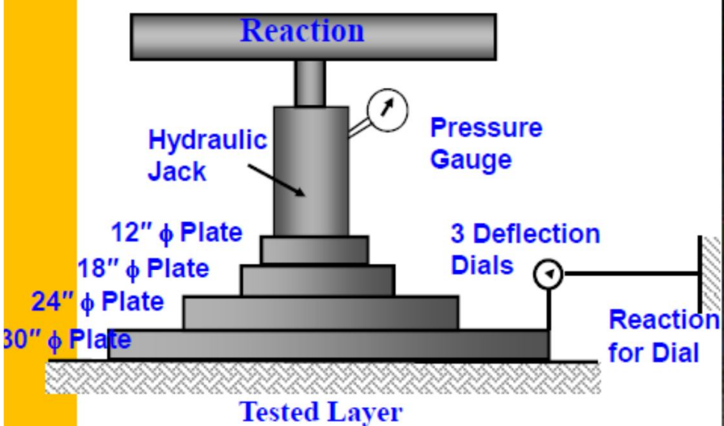


Plate Loading Test Schematic

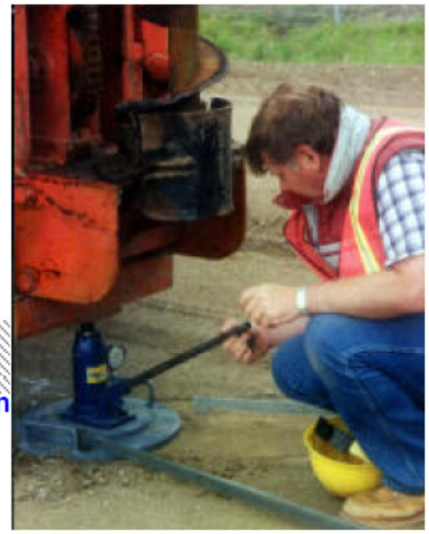
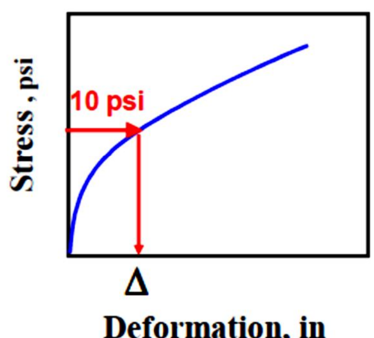


Plate Loading Test

Required for rigid pavement design.

$$K = \frac{P}{\Delta}$$



K = modulus of subgrade reaction
P = unit load on the plate (stress) (psi)
Δ = deflection of the plate (in)

- *For design use stress P = 10 psi (68.95 kN/m²)*

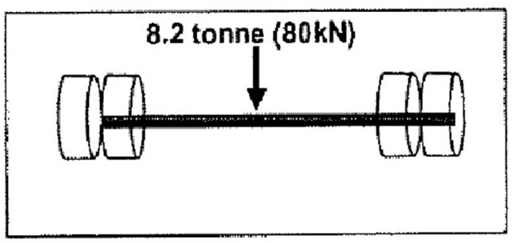
The 1993 AASHTO Guide offers the following relationship between k-values from a plate bearing test and resilient modulus (MR)

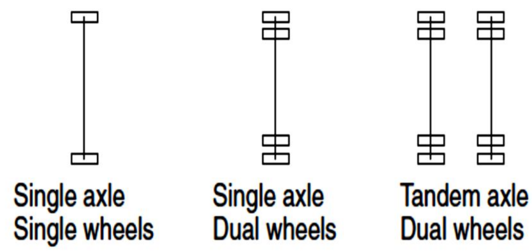
K= MR/19.4

Design Traffic Loading and contact area

Design Traffic Loading

The Standard Axle loading is defined as an axle with dual tyres loaded to 80kN (8.2 tonne).





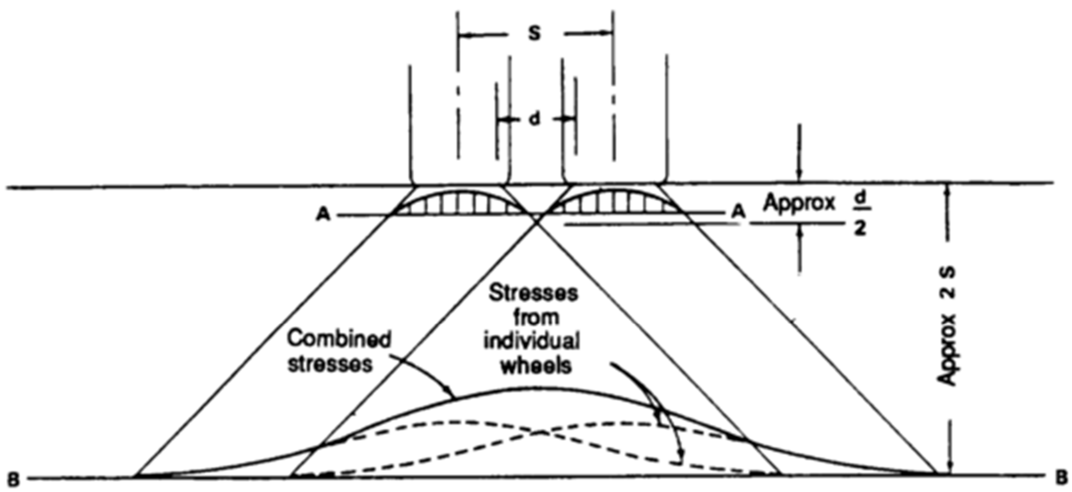


Figure 5-3. Distribution of pressures produced by multiple-wheel assemblies

- From zero up to pavement depth of d/2 , the dual wheel (4 ton) produce the same deflection on a single wheel load (2 ton)
- For depth of (2S) or greater, over lapped stress of dual wheels (4 ton) shall cause equivalent deflection of single wheel load with (4 ton)

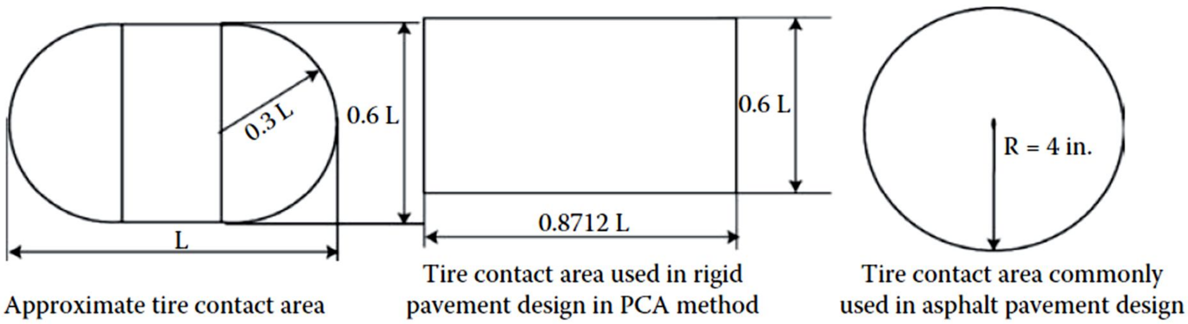


FIGURE 5.6 Assumed tire contact areas.

$$A_c = \pi(0.3L)^2 + (0.4L)(0.6L) = 0.5227L^2.$$

THEORETICAL CONSIDERATIONS FOR STRUCTURAL DESIGN

Before proceeding to structural design, it is necessary to review some of the basic mechanistic techniques that enable us to compute stresses and strains due to loads.

BOUSSINESQ'S METHOD

This method provides a way of determination of stresses, strains, and deflections of homogeneous, isotropic, linear elastic, and semi-infinite space under a point load. For flexible pavement design, layers are often simplified as homogeneous, isotropic, linear elastic materials. Boussinesq in 1885 was able to compute the vertical stress at any depth due to a static point load applied on a single-layer system

Vertical stress:

$$\sigma_z = \frac{-3Pz^3}{2\pi R^5}$$

Radial stress:

$$\sigma_r = -\frac{P}{2\pi}\left[\frac{3r^2z}{\left(r^2+z^2\right)^{5/2}} - \frac{1-2\mu}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

Tangential stress:

$$\sigma_t = -\frac{P}{2\pi}(1-2\mu)\left[\frac{z}{\left(r^2+z^2\right)^{3/2}} - \frac{1}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

Shear stress:

$$\tau_{rz} = \frac{P}{2\pi}\left[\frac{3rz^2}{\left(r^2+z^2\right)^{5/2}}\right]$$

Vertical deformation below the surface:

$$u_{zr} = \frac{P(1+\mu)}{2\pi E}\left[\frac{2(1-\mu)}{R} + \frac{z^2}{R^3}\right]$$

Surface (i.e., at $z=0$) vertical deflection:

$$u_r = \frac{(1-\mu^2)P}{\pi E R}$$

where
r is the radial distance from the point load
z is the depth

Example:

4500 Ib point load is applied on the surface of a soil layer with a modulus of 7.2 Ksi and Poisson ratio of 0.45, consider the conditions that are applicable for the use of Boussinesq's formulae, Determine the vertical and radial stress and vertical deformation at a depth of 6 in. and radial distance of 3 in.

Solution: P = 4500 lb; z = 6 in.; r = 3 in.; E = 7.2 ksi = 7200 psi; μ = 0.45

$$R = \sqrt{(6^2 + 3^2)} = 6.71 \text{ in.}$$

$$\text{Vertical stress} = \sigma_z = \frac{-3Pz^3}{2\pi R^5} = -34.14 \text{ psi (compressive)}$$

$$\text{Radial stress} = \sigma_r = -\frac{P}{2\pi}\left[\frac{3r^2z}{\left(r^2+z^2\right)^{5/2}} - \frac{1-2\mu}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

$$= 7.69 \text{ psi (compressive)}$$

Vertical deformation below the surface:

$$u_{zr} = \frac{P(1+\mu)}{2\pi E}\left[\frac{2(1-\mu)}{R} + \frac{z^2}{R^3}\right] = 0.04 \text{ in.}$$

Tests show that in most cases stresses and deflections predicted by this method are larger than measured values.

BURMISTER'S METHOD FOR TWO-LAYER SYSTEMS

Burmister developed solutions for stresses and displacements in two-layer systems: $\mu = 0.5$ for each layer, in homogeneous, isotropic, and elastic materials, with the surface layer infinite in extent in the lateral direction but of finite depth, the underlying layer infinite in both horizontal and vertical directions, layer in continuous contact. Total surface deflection, Δt, for a two-layer system:

Flexible plate:

$$\Delta_t = 1.5\frac{pa}{E_2}F_2$$

Rigid plate:

$$\Delta_t = 1.18\frac{pa}{E_2}F_2$$

where

p is the unit load on circular plate

a is the radius of plate

E2 is the modulus of elasticity of lower layer

F2 is the dimensionless factor depending on the ratio of moduli of elasticity of the pavement and subgrade (E1/E2) as well as the thickness of upper layer-to-radius of circular loaded ratio (h1/a), as shown in Figure 2.12

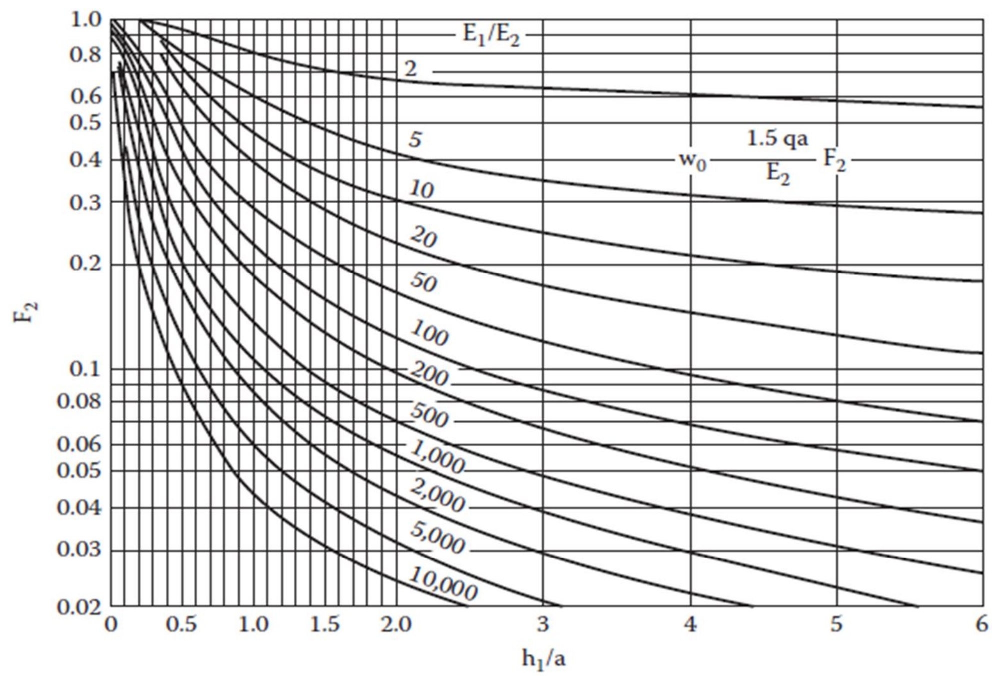


FIGURE 2.12 Values of dimensionless factor (F_2) in Burmister's method for two layer system. (From Huang, Y.H., *Pavement Analysis and Design*, 2nd edn., Upper Saddle River, NJ, p. 60, 2004. Reprinted by permission of Pearson Education, Inc.)

Note that stress and deflections are influenced by the ratio of the modulus of the pavement (everything above subgrade) and subgrade, there is significant effect of the layers above the subgrade, and there is significant difference in stress gradients obtained from Boussinesq's and Burmister's theory.

As an example of the application of Burmister's theory, consider the following problem.

Example:

A 4500 lb load is being applied over a circular area with a stress (unit load) of 100 psi on a pavement with two layers—a hot mix asphalt (HMA) layer with a thickness of 6 in. and a modulus of 360 ksi and a subgrade (considered to be of infinite thickness) of 7.2 ksi. Consider a Poisson's ratio of 0.5 for both layers. What is the surface deflection?

$$\text{Area} = \pi a^2 = \frac{4500}{100} = 45\,\text{in.}^2$$

$$a = 3.8\,\text{in.}$$

$$p = 100\ \text{psi}$$

$$h_1 = 6\ \text{in.}$$

$$h_1/a = 1.6$$

$$E_1 = 360\ \text{ksi}$$

From Figure 2.12, $F_2 = 0.2$

$$E_2 = 7.2\ \text{ksi} = 7200\ \text{psi}$$

$$\Delta_t = 1.5\frac{pa}{E_2}F_2 = 0.016\,\text{in.}$$

$$E_1/E_2 = 50$$

H.W: Calculate the deflection at the surface of a pavement due to a wheel load of 40 KN and a tyre pressure of (0.5 Mn/m^2), the value of E of pavement and subgrade may be assumed uniformly equal to (20 MN/ m^2).

Example : Design a thickness of a flexible pavement by burmister two layer analysis for a wheel load of 40 KN and a tyre pressure of 0.5 Mn/m$^{2.}$ The modulus of elasticity of the pavement material is 150 Mn/m^2 and that of subgrade is 30 Mn/m^2.

Solution:

Tyre pressure = $\frac{Wheel\ load}{\pi * a^2} = \frac{40*1000}{\pi a^2} = 0.5 * 10^6 \rightarrow$ a= 15.95 cm , say a = 16cm

Select thickness of pavement of 2a , i.e. 32cm

$\frac{E1}{E2} = \frac{150}{30} = 5$

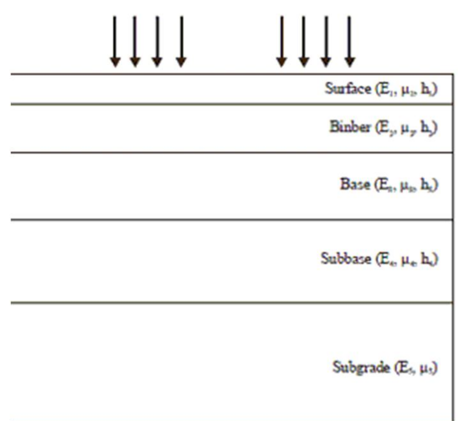
From figure the value of displacement factor = 0.43

$\Delta = \frac{Fw*1.5*P\ a}{E2} = \frac{0.43*1.5*\ 0.5*10^6*0.16}{30*10^6}$ =0.00172 m= 0.172 cm

Allowable settlement is 0.5 cm, hence design is safe.

ODEMARK'S METHOD OF EQUIVALENT LAYERS

Odemark developed a method whose principle is to transform a system consisting of layers with different moduli into an equivalent system where all the layers have the same modulus, and on which Boussinesq's equation may be used. This method is known as the method of equivalent thickness (MET).



$$\text{Stiffness} = \frac{IE}{(1-\mu^2)}$$

where I is the moment of inertia. For stiffness to remain constant, this expression must remain constant, from which we can say the following:

$$\frac{h_e^3 E_2}{1-\mu_2^2} = \frac{h_1^3 E_1}{1-\mu_1^2}$$

$$h_e = h_1 \left[\frac{E_1}{E_2 \left(\frac{1-\mu_2^2}{1-\mu_1^2} \right)} \right]^{1/3}$$

where he is the equivalent thickness. Note that this is an approximate method, and a correction factor is used to obtain a better agreement with elastic theory. In many cases, Poisson's ratio may as well be assumed to be the same for all materials. Then, the aforementioned equation becomes

$$h_e = f h_1 \left[\frac{E_1}{E_2} \right]^{1/3}$$

$$[\mu_1 = \mu_2]$$

Example:

there are two layers—HMA of thickness 6 in. and modulus 360 ksi and a subgrade of 7.2 ksi modulus. If it is necessary to determine the stress at the bottom of the HMA layer (which is a critical factor for evaluating the fatigue cracking potential), and the only available equation is the one that is available for a one-layer system, that is,

$$\sigma_z = \sigma_o \left[1 - \frac{1}{\left\{ 1 + (a/z)^2 \right\}^{3/2}} \right]$$

First, convert the HMA layer thickness to an equivalent thickness of the subgrade layer, and then utilize the aforementioned equation (for finding stress) by considering z as that equivalent thickness:

$$h_e = fh_1\left[\frac{E_1}{E_2}\right]^{1/3}$$

$[\mu_1 = \mu_2]$

$f = 0.9$

$h_1 = 6$ in.

$E_1/E_2 = 50$

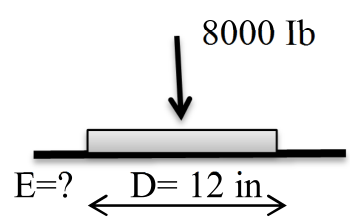
$h_e = 19.6$ in. using z = 19.6 in., a = 3.8 in., and σ0 = 100 psi.

Vertical stress at the bottom of the HMA layer

$$\sigma_z = \sigma_o\left[1 - \frac{1}{\left\{1+(a/z)^2\right\}^{3/2}}\right] = 5.4\,\text{psi}$$

Example: a plate loading test using rigid plate of 12 in diameter was performed on the surface of the subgrade as shown in the Figure below, a total load of 8000 Ib was applied to the plate and a deflection of o.1 in was measured. Assuming that the subgrade has poison ratio = 0.4 determine the elastic modulus of the subgrade.

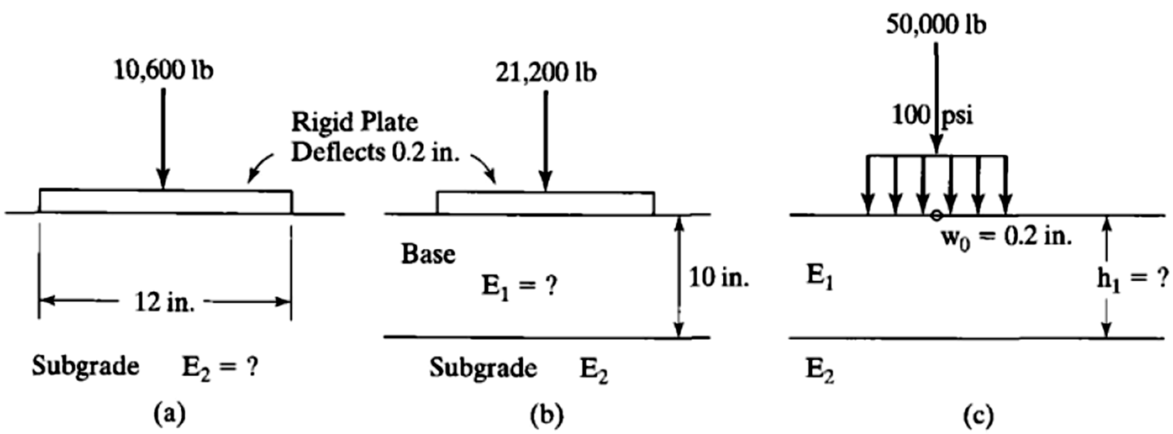
Solution



$q = \frac{8000}{\pi * 36} = 70.74$ psi

$\Delta = \frac{\pi\,(1-V^2) q\, a}{2\,E} \longrightarrow$ $E = \frac{\pi\,(1-0.4^2 * 70.74 * 6}{2 * 0.1} = 5600$ psi

Example: a plate bearing test using a 12 in diameter rigid plate is made on a subgrade as shown in Figure below, the total load required to cause settling by 0.2 in is 10600 Ib. After 10 in gravel base course is placed on the subgrade a plate bearing test is made on the top of the base course, the total load required to cause settling by 0.2 in is 21200 Ib. Assuming poison ratio = 0.5 , determine the thickness of base course required to sustain a 50000 Ib tire exerting a contact pressure of 100 psi over a circular area, maintain deflection of no more than 0.2 in.



Solution:

a) $q = \frac{P}{A} = \frac{10600}{\pi * \frac{1}{4} * 12^2} = 93.7$ psi

Deflection of plate can be calculated by:

$$\frac{\pi\,(1-v^2) q\, a}{2\,E} = \frac{\pi\,(1-0.5^2) * 93.7 * 6}{2 * 0.2} = 3310 \text{ psi}$$

b) $q = \frac{21200}{\pi * \frac{1}{4} * 12^2} = 187.5\ psi$

two layer system (burmister) :

$$\frac{h1}{a} = \frac{10}{6} = 1.67$$

For rigid plate: $\frac{1.18\, q\, a\, f2}{E2} =$ $\qquad f2 = \frac{0.2 * 3310}{1.18 * 187.5 * 6} = 0.5$

For h1/a = 1.67 and f2 = 0.5 from figure E1/E2 can be determined = 4

E1= 13200 psi

c) Ac = $\frac{load}{pressure} = \frac{50000}{100} = 500\ so\ a = 12.6\ in$

Surface deflection

$$\frac{f2\ 1.5\ p\ a}{E2} = \qquad F2 = \frac{0.2 * 3310}{1.5 * 100 * 12.6} = 0.35$$

From figure h1/a = 5.5

h 1= 5.5 a = 5.5 *12.6 =70 in

STRUCTURAL DESIGN

Structural design is based on the concept of limiting stresses and deformations to prevent excessive damage and deterioration of pavements. Overstressed pavements due to traffic loads and environmental effects will result in pavement distress such as fatigue cracking, rutting, etc. as shown in Figure 2.6.

A flexible pavement "flexes" under a load; repeated loadings cause repeated compressive strains as well as tensile strains

FIGURE 2.6 Major distress-causing responses in a "flexible" or asphalt mix pavement.

The application of a wheel load causes a stress distribution which can be represented as shown in Figure 19.2. The maximum vertical stresses are compressive and occur directly under the wheel load. These decrease with an increase in depth from the surface. The maximum horizontal stresses also occur directly under the wheel load but can be either tensile or compressive as shown in Figure 19.2(c). When the load and pavement thickness are within certain ranges, horizontal compressive stresses will occur above the neutral axis whereas horizontal tensile stresses will occur below the neutral axis. The temperature distribution within the pavement structure, as shown in Figure 19.2(d), will also have an effect on the magnitude of the stresses.

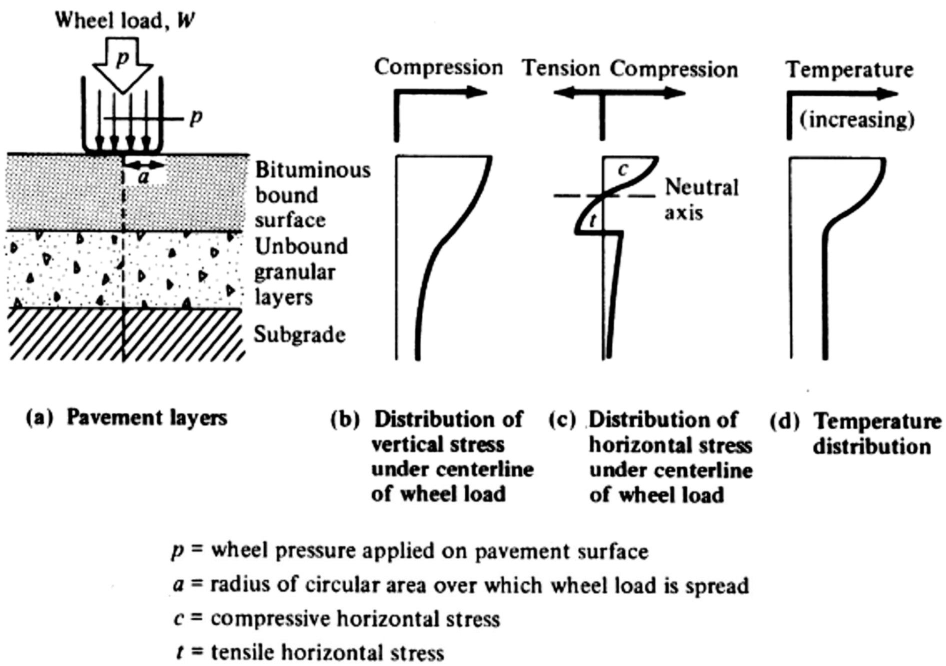


Figure 19.2 Stress Distribution within a Flexible Pavement

Structural design process is achieved primarily by two different techniques:

(1) by using ***empirical methods***—that is, charts and equations developed from experimental studies carried out with a set of traffic, environment, and pavements

or

(2) by using a ***mechanistic method***, in which concepts of mechanics are used to predict responses and performance of the pavement. Note that a purely mechanistic approach is not possible at this time—the responses can be predicted by employing concepts of mechanics, but the performance has to be predicted by empirical models.

Pavements can be designed either by using the *empirical approach* or by using the *mechanistic-empirical approach (ME).*

Pavement Performance

The primary factors considered under pavement performance are the structural and functional performance of the pavement.

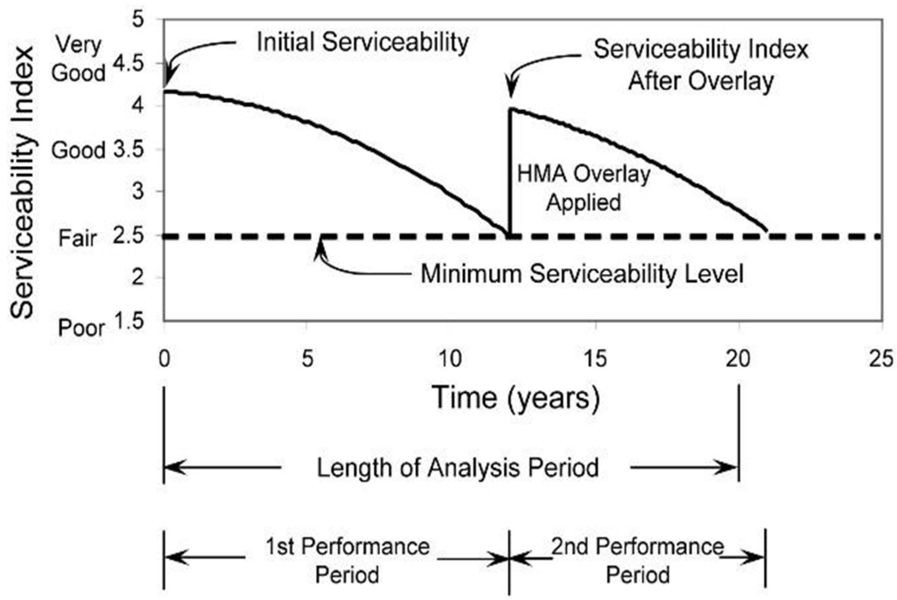
Structural performance is related to the physical condition of the pavement with respect to factors that have a negative impact on the capability of the pavement to carry the traffic load. These factors include cracking, faulting, raveling, and so forth.

Functional performance is an indication of how effectively the pavement serves the user. The main factor considered under functional performance is riding comfort.

To quantify pavement performance, a concept known as the **serviceability** performance was developed. Under this concept, a procedure was developed to determine the present serviceability index (PSI) of the pavement, based on its roughness and distress, which were measured in terms of extent of cracking, patching, and rut depth for flexible pavements. The original expression developed gave the PSI as a function of the extent and type of cracking and patching and the slope variance in the two wheel paths which is a measure of the variations in the longitudinal profile. The scale PSI ranges from 0 to 5, where 0 is the lowest PSI and 5 is the highest.

Two serviceability indices are used in the design procedure:

The **initial serviceability index (pi),** which is the serviceability index immediately after the construction of the pavement; and the **terminal serviceability index (pt),** which is the minimum acceptable value before resurfacing or reconstruction is necessary. Recommended values for the terminal serviceability index are 2.5 or 3.0 for major highways and 2.0 for highways with a lower classification.



ΔPSI = ΔPSI traffic + ΔPSI environmental

ΔPSI environmental = ΔPSI swelling + ΔPSI frost

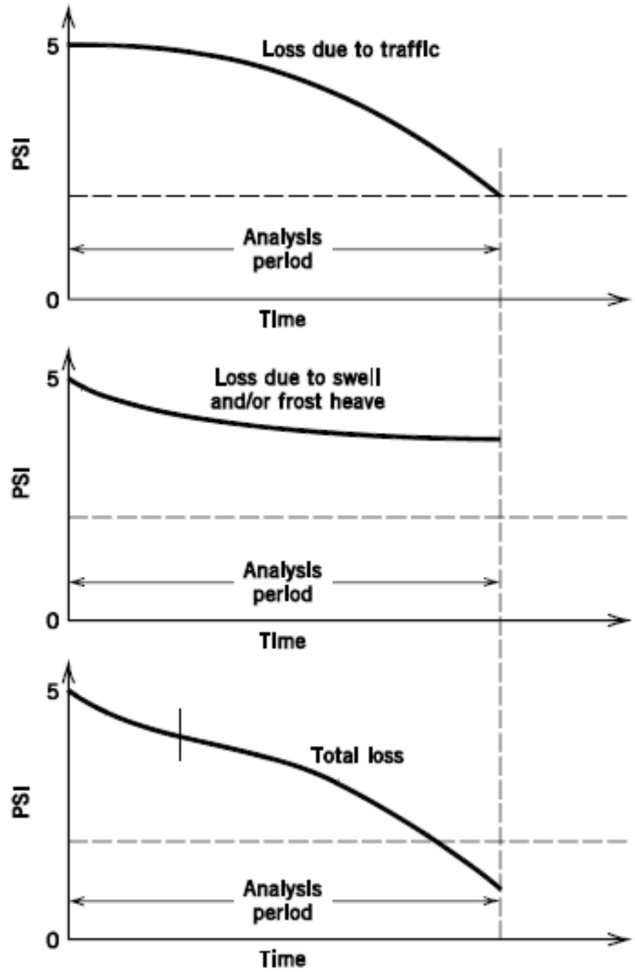


Figure 4.4 Pavement performance trends.
Redrawn from *AASHTO Guide for Design of Pavement Structures*, Washington, DC, The American Association of State Highway and Transportation Officials, 1993. Used by permission.

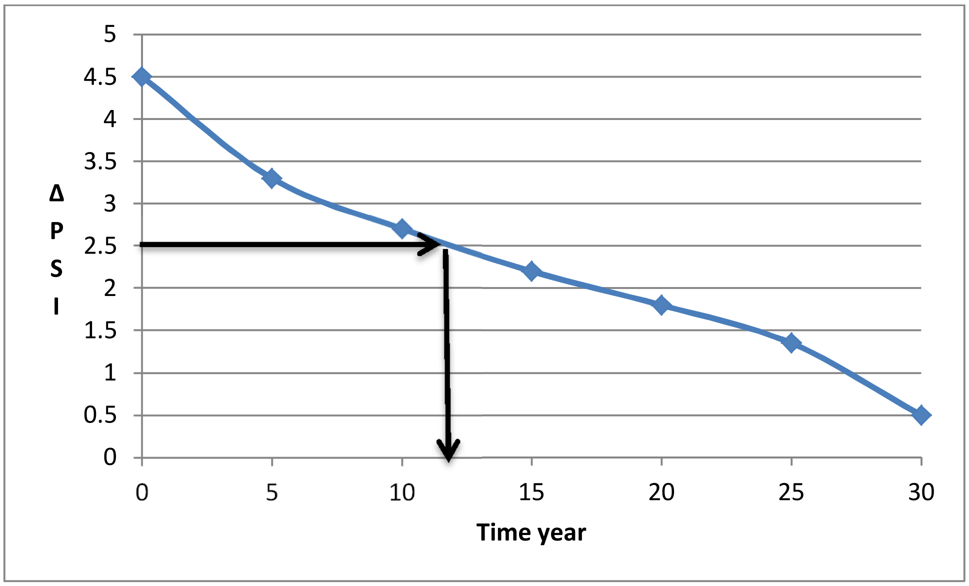
Example : For the following highway pavement information find performance period if Po = 4.5, Pt =2.5, analysis period = 30 yr

Time (year)	0	5	10	15	20	25	30
Loss in PSI due to Environmental	0	0.3	0.5	0.7	0.9	1.25	2
Loss in PSI due to traffic	0	0.9	1.3	1.6	1.8	1.9	2

Solution :

Total loss ΔPSI =loss due to envir. + loss in PSI due to traffic

Time (year)	0	5	10	15	20	25	30
Loss in PSI due to Environmental	0	0.3	0.5	0.7	0.9	1.25	2
Loss in PSI due to traffic	0	0.9	1.3	1.6	1.8	1.9	2
Total Loss	0	1.2	1.8	2.3	2.7	3.15	4
ΔPSI =(Pi –Total loss)	4.5	3.3	2.7	2.2	1.8	1.35	0.5



Performance year = 11.25 year

Traffic Loads

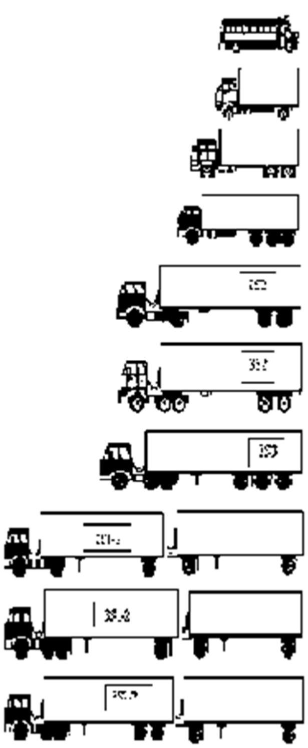
Pavement structural design requires a quantification of all expected loads a pavement will encounter over its design life. This quantification is usually done in one of two ways:

1. Equivalent single axle loads (ESALs). This approach converts wheel loads of various magnitudes and repetitions ("mixed traffic") to an equivalent number of "standard" or "equivalent" loads.
2. Load spectra. This approach characterizes loads directly by number of axles, configuration and weight. It classify traffic loading in terms of the number of

load applications of various axle configurations (single, dual, tridem, and quad) within a given weight classification range. It does not involve conversion to equivalent values. Structural design calculations using load spectra are generally more complex than those using ESALs.

Load spectra are the frequency distributions of axle load magnitudes by axle configuration (single, tandem, tridem, quad) and season of year (monthly, typically).

Roadway Functional Classification: Urban Interstate



Vehicle Classification	% AADTT	Average Number of Axles Per Vehicle: Single	Tandem	Tridem
4	2.4	1.62	0.39	0
5	18.4	2	0	0
6	9.2	1.02	0.99	0
7	0	1	0.26	0.83
8	12.6	2.38	0.67	0
9	44.8	1.13	1.93	0
10	2.3	1.19	1.09	0.89
11	2.3	4.29	0.26	0.06
12	1.1	3.52	1.14	0.06
13	6.9	2.15	2.13	0.35
Total	100			

ESAL

The traffic load is determined in terms of the number of repetitions of an 18,000-lb (80 kilo newtons (kN)) single-axle load applied to the pavement on two sets of dual tires. This is usually referred to as the equivalent single-axle load (ESAL). The dual tires are represented as two circular plates, each 4.51 in. radius, spaced 13.57 in. apart. This representation corresponds to a contact pressure of 70 lb/in^2.

To determine the ESAL, the number of different types of vehicles such as cars, buses, single-unit trucks, and multiple-unit trucks expected to use the facility during its lifetime must be known. The distribution of the different types of vehicles expected to use the proposed highway can be obtained from results of classification counts that are taken by state highway agencies at regular intervals. These can then be converted to equivalent 18,000-lb loads using the equivalency factors given in Table 19.3.

Flexible highway pavements are usually designed for a 20-year period. Since traffic volume does not remain constant over the design period of the pavement, it is essential that the rate of growth be determined and applied when calculating the total ESAL. Annual growth rates can be obtained based on traffic volume counts over several years. The overall growth rate in the United States is between 3 and 5 percent per year, although growth rates of up to 10 percent per year have been suggested for some interstate highways. The growth factors (Grn) for different growth rates and design periods can be obtained from Equation 19.1.

$G_{rn} = [(1+r)^n - 1]/r$ ………..(19.1)

Where : $r = \frac{i}{100}$ and is not zero. If annual growth is zero, growth factor = design period

i= growth rate

n= design life, yrs

A general equation for the accumulated ESAL for each category of axle load is obtained as

$ESAL_i = f_d * G_{rn} * AADT_i * 365 * N_i * F_{Ei}$ …….(19.2)

Where

$ESAL_i$ = equivalent accumluted 18000 Ib (80 KN) single axle load for the axle category i

f_d =design lane factor (Table 8-6)

G_{rn} = growth factor for a given growth rate 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

(fd) is used in the determination of ESAL. Either lane of a two-lane highway can be considered as the design lane whereas for multilane highways, the outside lane is considered. The identification of the design lane is important because in some cases more trucks will travel in one direction than in the other or trucks may travel heavily loaded in one direction and empty in the other direction.

Table 19.3a Axle Load Equivalency Factors for Flexible Pavements, Single Axles, and p_t of 2.5

	Pavement Structural Number (SN)					
Axle Load (kips)	*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	.606
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.57	1.49	1.47	1.51	1.55
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	22.5	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	112.0	102.0	79.0	60.0	53.0	55.0

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.

Table 19.3b Axle Load Equivalency Factors for Flexible Pavements, Tandem Axles, and p_t of 2.5

	Pavement Structural Number (SN)					
Axle Load (kips)	*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	.451	.493	.548	.534	.495	.470
30	.611	.648	.703	.695	.658	.633
32	.813	.843	.889	.887	.857	.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	1.75	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

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.

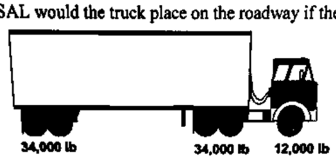
Table 19.4 Growth Factors

Design Period, Years (n)	*Annual Growth Rate, Percent (r)* No Growth	2	4	5	6	7	8	10
1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3.0	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4.0	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5.0	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6.0	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7.0	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8.0	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9.0	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10.0	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11.0	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12.0	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13.0	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14.0	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15.0	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16.0	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17.0	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18.0	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19.0	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20.0	24.30	29.78	33.06	36.79	41.00	45.76	57.28
25	25.0	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30.0	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35.0	49.99	73.65	90.32	111.43	138.24	172.32	271.02

SOURCE: *Thickness Design–Asphalt Pavements for Highways and Streets*, Manual Series No. 1, The Asphalt Institute, Lexington, KY, February 1991. Used with permission.

H.W

Determine the ESAL would the truck place on the roadway if the roadway has:



1) Flexible pavement?

2) Rigid pavement?

TABLE 8.6 Lane Distribution Factor (AASHTO, 1993)

No. of Lanes in Each Direction	% of 18-kip ESAL in the Design Lane
1	100
2	80–100
3	60–80
4	50–70

Example

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 (pt) 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.

ESAL = fd * Gjt *AADT * 365* Ni *FEi…………….. Eq 19.2

2-axle single-unit trucks = 0.45 * 29.78 *12,000 * 0.33 *365 * 2 *0.010 = 0.3874 10^6

3-axle single-unit trucks = 0.45* 29.78 * 12,000 * 0.17 * 365 * 3 *0.0877 = 2.6343 *10^6

Thus, Total ESAL= 3.0217 *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.

Structural Design using Road Note 31

Road Note 31 gives recommendations for the structural design of bituminous surfaced roads in tropical and subtropical climates. It is aimed at highway engineers responsible for the design and construction of new road pavements and is appropriate for roads which are required to carry up to 30 million cumulative equivalent standard axles in one direction.

THE DESIGN PROCESS

There are three main steps to be followed in designing a new road pavement. These are:

(i) Estimating the amount of traffic and the cumulative number of equivalent standard axles that will use the road over the selected design life;

(ii) Assessing the strength of the subgrade soil over which the road is to be built;

(iii) selecting the most economical combination of pavement materials and layer thicknesses that will provide satisfactory service over the design life of the pavement (It is usually necessary to assume that an appropriate level of maintenance is also carried out).

Example:

Design a pavement structure of a residential road in Baghdad city, Road Note 31-TRL design method is proposed to be used for this project. The following information is available: the resilient modulus= 9000 psi and ESAL=11.4*10^6.

Knowing that to minimize the cost, the total thickness of pavement layers needs to be not more than 600mm?

Solution:

MR= CBR *1500 = → CBR= 9000/1500 = 6

(T7,S3)

Ch1 → not found

Ch2→150+125+150+150=575

Ch3→ not found

Ch4 →50+150+125+125+150=600

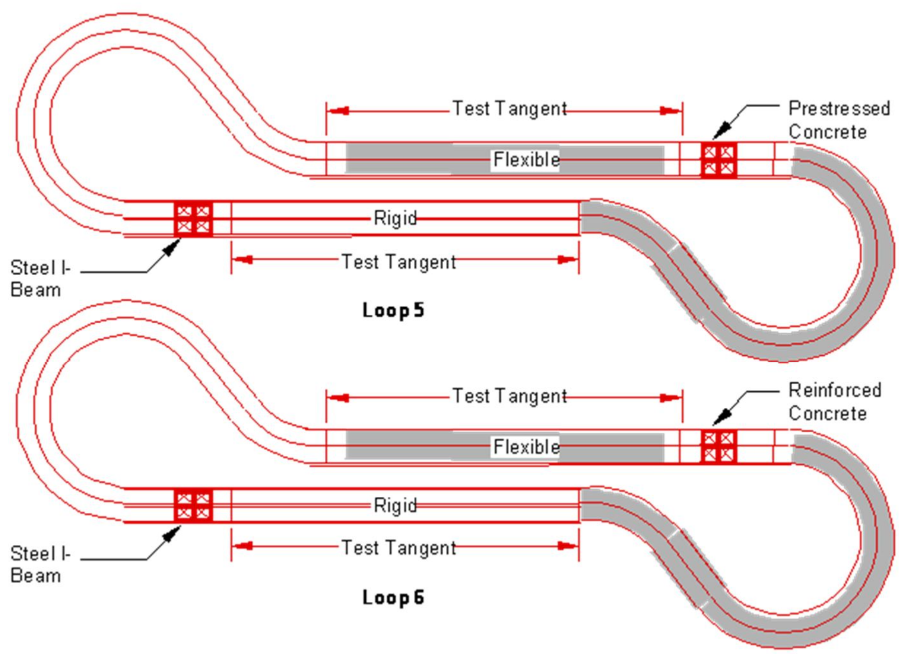
Ch5→ 125+225+250=600

Ch6→125+150+200+125=600

Ch7→50+175+275=500 Ch8→not found

AASHTO Design Method

The AASHTO method for design of highway pavements is based primarily on the results of the AASHTO road test that was conducted in Ottawa, Illinois. It was a cooperative effort carried out under the auspices of 49 states, the District of Columbia, Puerto Rico, the Bureau of Public Roads, and several industry groups. Tests were conducted on short-span bridges and test sections of flexible and rigid pavements constructed on A-6 subgrade material. The pavement test sections consisted of two small loops and four larger ones with each being a four-lane divided highway. The tangent sections consisted of a successive set of pavement lengths of different designs, each length being at least 100 feet. The principal flexible pavement sections were constructed of asphalt mixture surface, a well-graded crushed limestone base, and a uniformly graded sand-gravel subbase. Three levels of surface thicknesses ranging from 1 to 6 inches were used in combination with three levels of base thicknesses ranging from 0 to 9 inches. Test traffic consisting of both single-axle and tandem-axle vehicles were then driven over the test sections until several thousand load repetitions had been made. Data were then collected on the pavement condition with respect to extent of cracking and amount of patching required to maintain the section in service. The longitudinal and transverse profiles also were obtained to determine the extent of rutting, surface deflection caused by loaded vehicles moving at very slow speeds. These data then were analyzed thoroughly, and the results formed the basis for the AASHTO method of pavement design.



Design Considerations

The factors considered in the AASHTO procedure for the design of flexible pavement as presented in the 1993 guide are:

• Pavement performance

• Traffic

• Roadbed soils (subgrade material)

• Materials of construction

• Environment

• Drainage

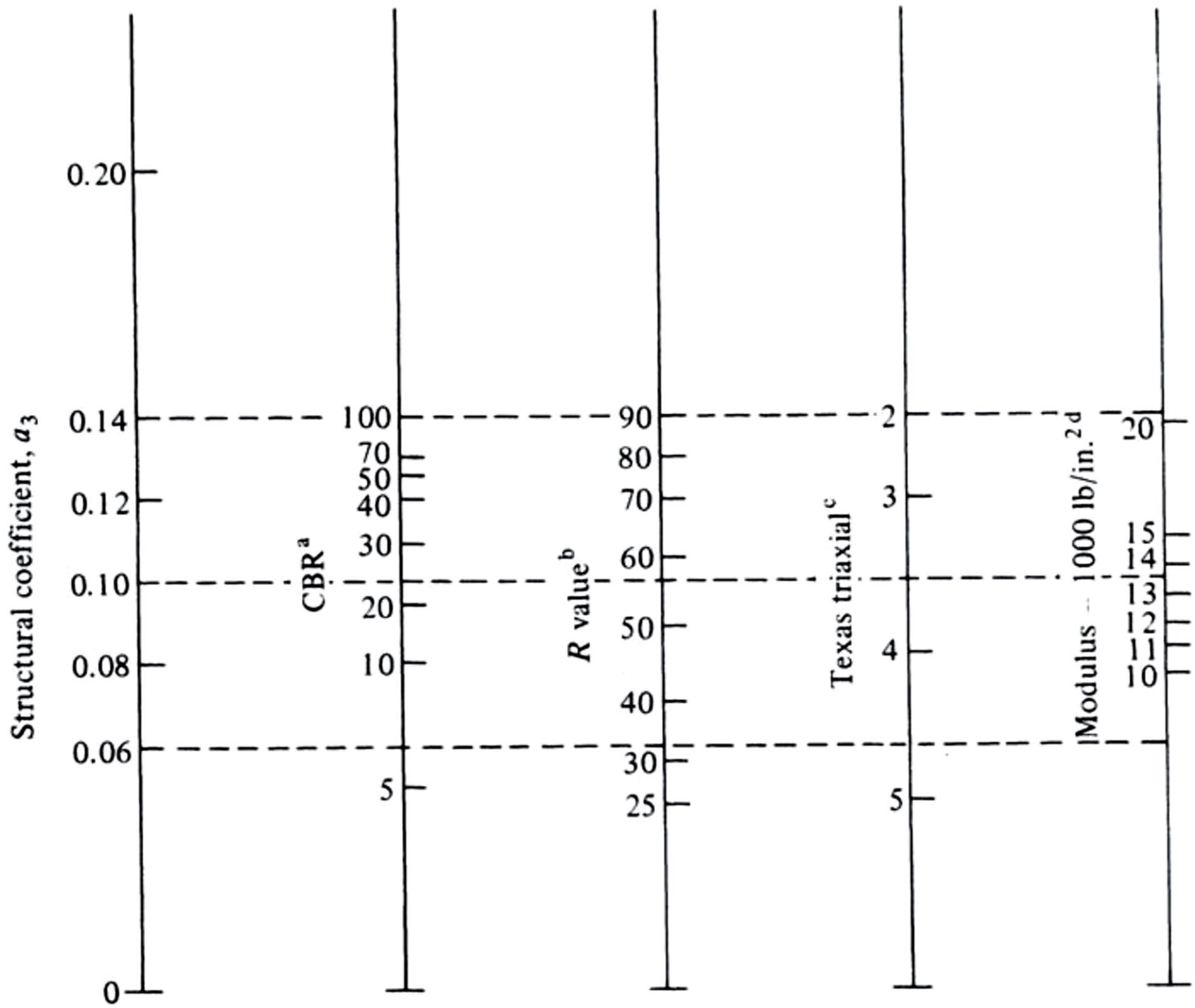
• Reliability

Roadbed Soils (Subgrade Material). The 1993 AASHTO guide also uses the resilient modulus (Mr) of the soil to define its property. However, the method allows for the conversion of the CBR or R value of the soil to an equivalent Mr value using the following conversion factors:

Mr (lb/in^2) =1500 CBR (for fine-grain soils with soaked CBR of 10 or less)...(19.3)

Mr (lb/in^2) = 1000+ 555 R value (for $R \leq 20$)………. (19.4)

Subbase Construction Materials: The quality of the material used is determined in terms of the layer coefficient, a3, which is used to convert the actual thickness of the subbase to an equivalent SN. The sandy gravel subbase course material used in the AASHTO road test was assigned a value of 0.11. Layer coefficients are usually assigned, based on the description of the material used. Charts correlating the layer coefficients with different soil engineering properties have been developed. Figure 19.3 shows one such chart for granular subbase materials.



[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 1 28, 1972.

Figure (19.3) Variation in Granular Subbase Layer Coefficient, a3, with Various Subbase Strength Parameters

Base Course Construction Materials :Materials selected should satisfy the general requirements for base course materials A structural layer coefficient, a2, for the material used also should be determined. This can be done using Figure 19.4

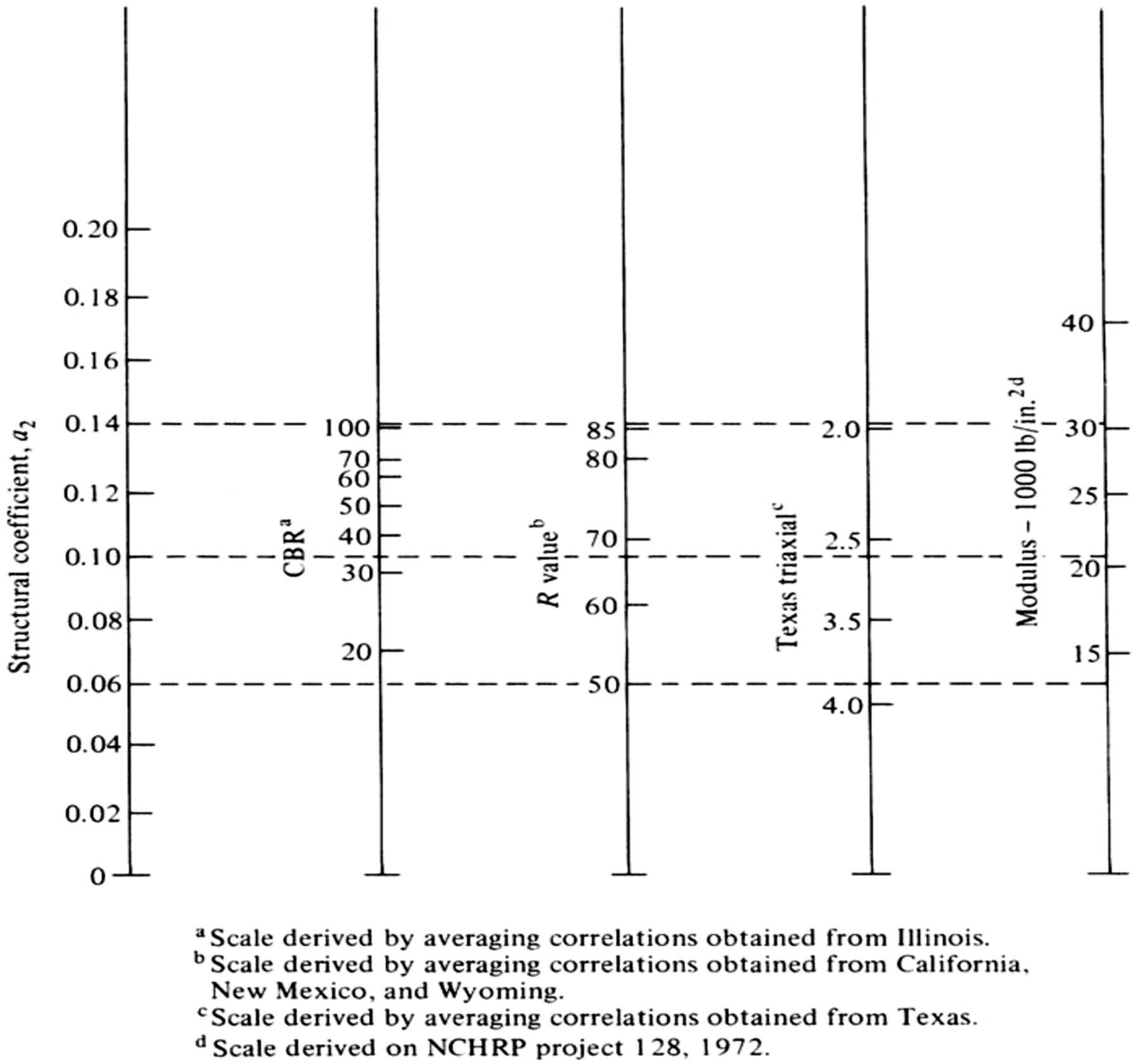


Figure (19.4) Variation in Granular Base Layer Coefficient, a2, with Various Subbase Strength Parameters

Surface Course Construction Materials. The most commonly-used material is a hot plant mix of asphalt cement and dense-graded aggregates with a maximum size of 1 inch. The structural layer coefficient (a1) for the surface course can be extracted from Figure 19.5, which relates the structural layer coefficient of a dense grade asphalt concrete surface course with its resilient modulus at 68°F.

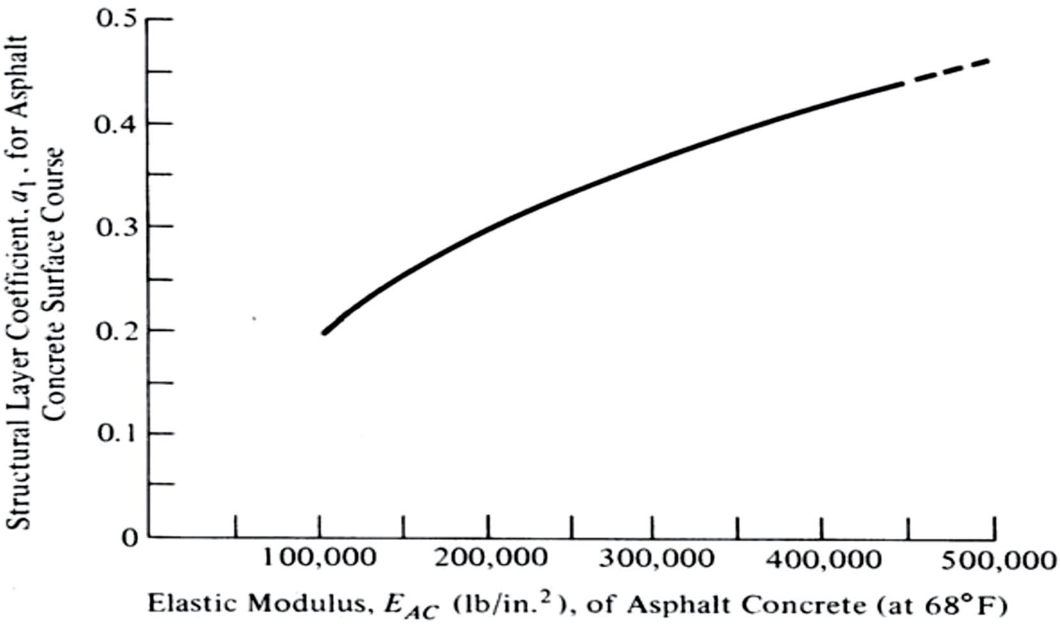


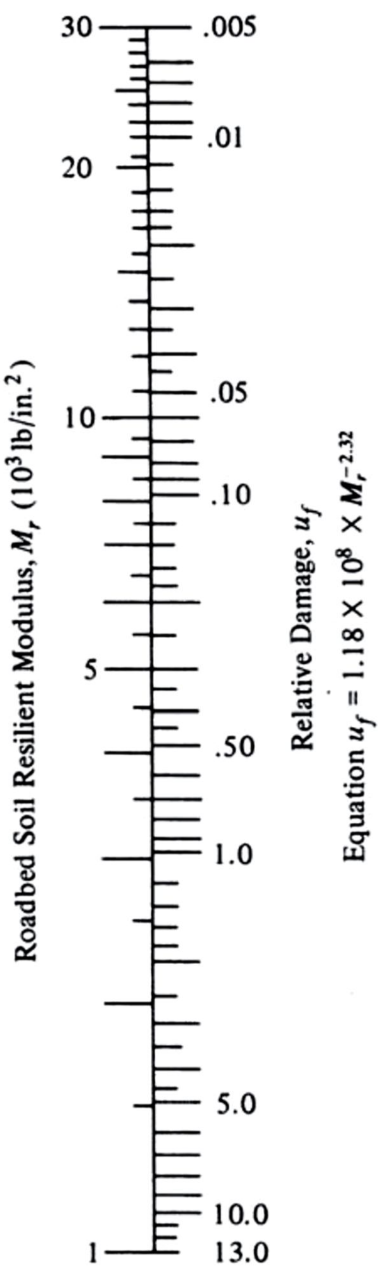
Figure (19.5) Chart for Estimating Structural Layer Coefficient of Dense-Graded/Asphalt Concrete Based on the Elastic (Resilient) Modulus

Environment.

Temperature and rainfall are the two main environmental factors used in evaluating pavement performance in the AASHTO method. The 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. The effect of rainfall is due mainly to the penetration of the surface water into the underlying material. However, this effect is taken into consideration in the design procedure, and the methodology used is presented later under “Drainage.”

Test results have shown that the normal modulus (that is, modulus during summer and fall seasons) of materials susceptible to frost action can reduce by 50 percent to 80 percent during the thaw period. Also, resilient modulus of a subgrade material may vary during the year, even when there is no specific thaw period. This occurs in areas subject to very heavy rains during specific periods of the year. It is likely that the strength of the material will be affected during the periods of heavy rains.

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:	$\Sigma u_f =$	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}) = 7250 (corresponds to $\bar{u}_f$)

Figure (19.6) Chart for Estimating Effective Roadbed Soil Resilient Modulus for Flexible Pavements Designed Using the Serviceability Criteria

Example

Computing Effective Resilient Modulus

Figure 19.6 shows roadbed soil resilient modulus Mr for each month estimated from laboratory results correlating Mr with moisture content. Determine the effective resilient modulus of the subgrade.

Solution: The solution of the problem is given in Figure 19.6. The value of uf for each Mr is obtained directly from the chart. The mean relative damage uf is 0.133, which in turn gives an effective resilient modulus of 7250 lb/in^2.

Drainage.

The effect of drainage on the performance of flexible pavements is considered by modifying the structural layer coefficient. The modification is carried out by incorporating a factor mi for the base and subbase layer coefficients (a2 and a3). The mi factors are based both on the percentage of time during which the pavement structure will be nearly saturated and on the quality of drainage, which is dependent on the time it takes to drain the base layer to 50 percent of saturation.

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)

*Time required to drain the base layer to 50% saturation.
SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

Table 19.6 Recommended m_i Values

	Percent of Time Pavement Structure Is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	*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

SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

Reliability. It has been noted that the cumulative ESAL is an important input to any pavement design method. However, the determination of this input is usually based on assumed growth rates which may not be accurate. 1993 AASHTO guide proposes the use of a reliability factor that considers the possible uncertainties in traffic prediction and performance prediction. Reliability design levels (R%), which determine assurance levels that the pavement section designed using the procedure will survive for its design period, have been developed for different types of highways. For example, a 50 percent reliability design level implies a 50 percent chance for successful pavement performance— that is, the probability of design performance success is 50 percent.

Table 19.7 shows suggested reliability levels based on a survey of the AASHTO pavement design task force. Reliability factors, $F_R \geq 1$, based on the reliability level selected and the overall variation, S_o^2 also have been developed. S_o^2 accounts for the chance variation in the traffic forecast and the chance variation in actual pavement performance for a given design period traffic, W18.

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 FR is given as $\log_{10} FR = -ZR * So$

Where ZR= standard normal variate for a given reliability (R%)

ZR Represents the probability that serviceability will be maintained at adequate levels from a user's point of view throughout the design life of the facility.

So= estimated overall standard deviation

Table 19.8 values of ZR 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

SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

Structural Design

The objective of the design using the AASHTO method is to determine a flexible pavement Structural Number (SN) adequate to carry the projected design ESAL. This design procedure is used for ESALs greater than 50,000 for the performance period. The design for ESALs less than this is usually considered under low-volume roads. The 1993 AASHTO guide gives the expression for SN as

$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 M_3$

where

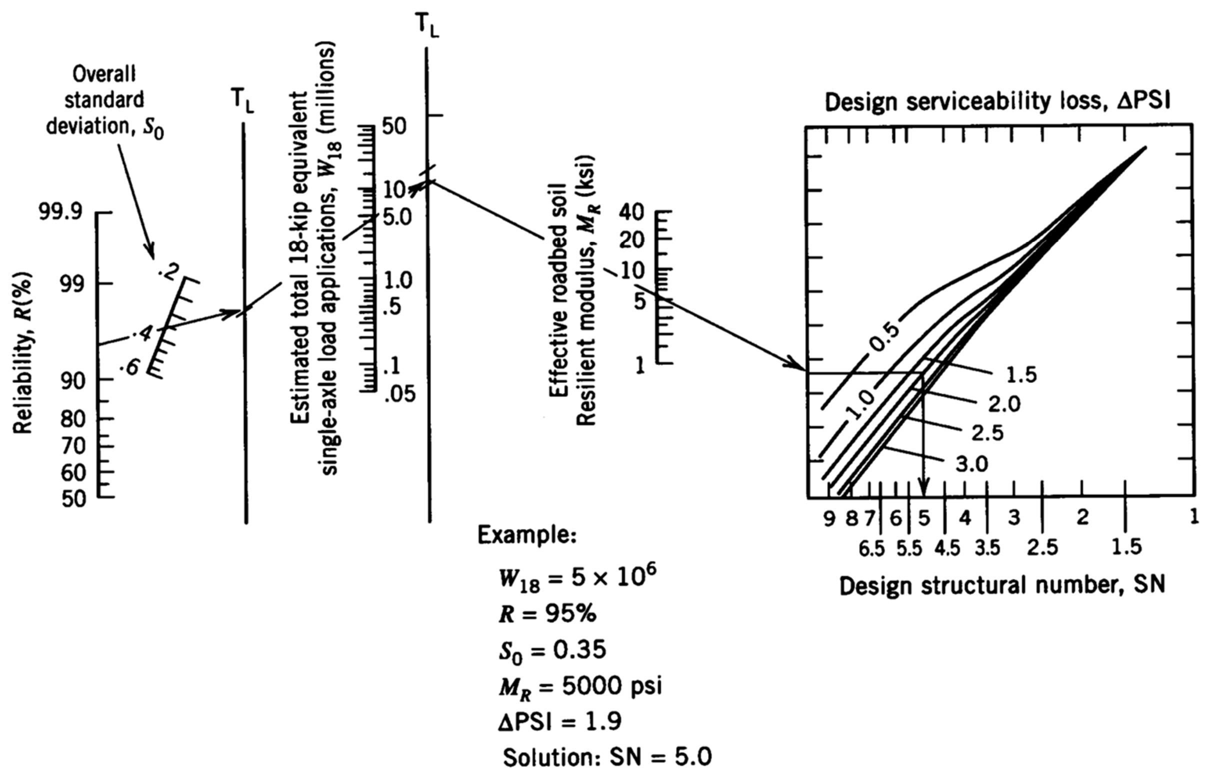
mi = drainage coefficient for layer i

a1, a2, a3 layer coefficients representative of surface, base, and subbase course, respectively

D1, D2, D3 = actual thickness in inches of surface, base, and subbase courses, respectively.

The basic design equation given in the 1993 guide is

$$\log_{10} W_{18} = Z_R S_o + 9.36 \log_{10}(\text{SN} + 1) - 0.20 + \frac{\log_{10}[\Delta\text{PSI}/(4.2 - 1.5)]}{0.40 + [1094/(\text{SN} + 1)^{5.19}]} + 2.32 \log_{10} M_r - 8.07 \quad (19.7)$$



Example

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, Mr = 31,000 lb/in^2

CBR value of subbase course material = 22, Mr = 13,500 lb/in^2

CBR value of subgrade material = 6

Determine a suitable pavement structure, Mr 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 (So) = 0.49 (range is 0.4 to 0.5)

Initial serviceability index pi = 4.5

Terminal serviceability index pt = 2.5

The nomograph in Figure 19.8 is used to determine the design SN

PSI = 4.5 – 2.5 = 2.

Determine the appropriate structure layer coefficient for each construction material.

(a) Resilient value of asphalt = 450,000 lb/in^2. From Figure 19.5, a1 = 0.44.

(b) CBR of base course material = 100. From Figure 19.4, a2 = 0.14.

(c) CBR of subbase course material = 22. From Figure 19.3, a3 =0.10.

Determine appropriate drainage coefficient mi. Since only one set of conditions is given for both the base and subbase layers, the same value will be used for m1 and m2. The time required for water to drain from within pavement=1week, 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, mi=0.80.

Determine appropriate layer thicknesses from Eq.

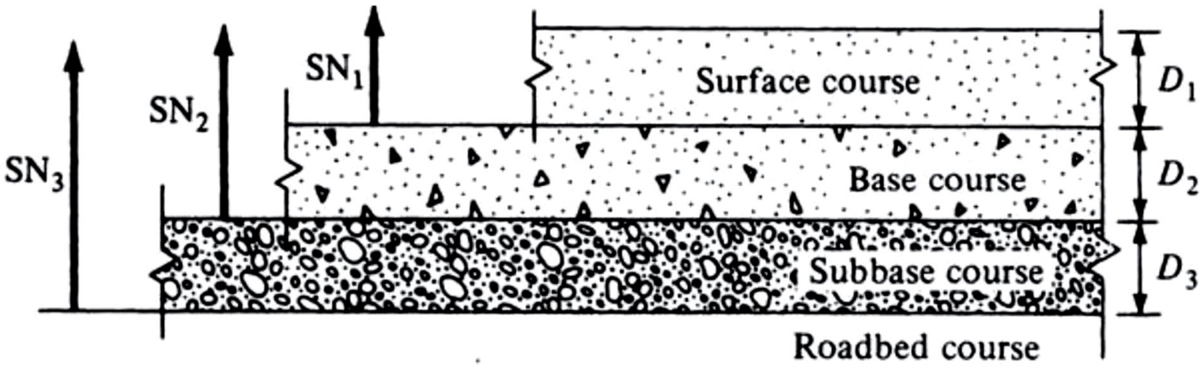


Table 19.9 AASHTO-Recommended Minimum Thicknesses of Highway Layers

	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 Mr in Figure 19.8, we obtain SN3 = 4.4 and

SN2 = 3.8. Note that when SN is assumed to compute ESAL, the assumed and computed SN3 values must be approximately equal. If these are significantly different, the computation must be repeated with a new assumed SN.

Mr for base course = 31,000 lb/in^2 $\longrightarrow$ from figure SN1 = 2.6

Giving D1 $= \frac{2.6}{0.44} = 5.9\ in$ Using 6 in., for the thickness of the surface course

D1* = 6 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.

Example:

A pavement is to be designed to last 10 year. PSI = 4.2, Pt = 2.5, MR subgrade =15000 ib/in^2 , R=95%, So=0.4, for design the average daily car = 30000, truck =1000, tractor semitrailer =350, the axle weight as follows:

Car 2 (2000 ib / axle)

Single unit truck = 8000 ib single

= 22000 ib single

Tractor trailer = 10000 ib single

= 16000 ib tandem

= 44000 ib triple

M2=M3 =1, 4 in thickness for HMA, 10 in for crushed stone as subbase determine thickness for base if a2= 0.2, a1 =0.44, a3= 0.11

Solution:

Find ESAL as follows

Car 2 kip single = 0.0002*2 = 0.0004

Truck 8 kip LEF =0.041

22 LEF =2.09

Tractor 10 kip LEF =0.102

16 Kip LEF =0.057

44 Kip LEF = 0.769

Design daily ESAL = 0.0004 *30000 + 2.131*1000 + 0.928*350 = 2467.8

ESAL for design period = 2467.8*365*10= $9*10^6$

SN= a1D1+ a2D2m2 + a3D3m3

3.94 = 0.44*4 + 0.2 *D2*1 + 0.11*10*1

D2= 5.4 use 6 in

H.W Design the pavement for an expressway consisting of an asphalt concrete surface, a crushed-stone base, and a granular subbase using the 1993 AASHTO design chart. The cumulative ESAL in the design lane for a design period of 15 years is $7* 10^6$. The area has good quality drainage with 10% of the time the moisture level is approaching saturation. The effective roadbed soil resilient modulus is 7 ksi, the subbase has a CBR value of 80, the resilient modulus of the base is 40 Ib, and the resilient modulus of asphalt concrete is $4.5 * 10^5$ psi. Assume a reliability level of 95% and So of 0.45.

Solution : D1 = 6.5 in. D2 = 6 in. D3 = 8 in.