regarding the use of the Interim Guides determined that many of the highway departments make some use of the Guides, either in their entirety or with modifications, in the design of pavement structures. Also, of the remaining states reporting no direct use of the Guides, some reported plans to attempt to adapt them to their use, while others report some indirect use, such as in modifying their own procedure for design, for evaluation of traffic, or for incorporating soil support into design. From these results it was concluded that improvement and refinement of the Guides would be a worthwhile effort that should prove to be of assistance to many states. It is emphasized that this revised Guide should be considered as remaining interim in nature, and subject to further adjustment based on experience and additional research. It is not intended to supersede design procedures that are being used satisfactorily by state highway agencies. (For example, other design procedures in use are: *Thickness Design Asphalt Pavement* Structures for Highways and Streets, Manual Series No. 1 (MS-1), The Asphalt Institute, College Park, Md. Revised Eighth Edition, Aug. 1970; *NCSA Flexible Pavement Design Guide for Highways* National Crushed Stone Association, Wash., D.C., May 1972; and *Thickness Design for Concrete Pavements*, Portland Cement Association, 1966.)

The basic design methods and the procedures for their use remain unchanged from those originally prepared in 1962. However, some material has been rearranged and simplified, and considerable additional explanatory material has been added, particularly the following:

1. Procedures and examples for determining traffic load-ometer data.
2. New design examples to cover a wider range of possible designs.
3. The presentation of overlay design procedures in most common use.

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Chapter I — GENERAL FACTORS RELATING TO THE DESIGN OF FLEXIBLE AND RIGID PAVEMENT

1.1 — REFERENCE SPECIFICATIONS

The Specifications for materials and test methods adopted by the American Association of State Highway and Transportation Officials have been accepted and used in this guide wherever applicable. Reference will be made to the appropriate designation when used. Some States are using methods that differ somewhat from the AASHTO procedures. Specification and test methods that are being used satisfactorily by state highway agencies are also acceptable.

1.2 — SCOPE

This interim guide is presented in the form of procedures applicable to the design of pavement structures for any system of highways. The design procedures include the determination of total thickness of the pavement structure, as well as the thickness of the individual structural components. Provision is made for the design for equivalent alternate sections, with the selection of the alternate being primarily a function of availability of materials and comparative costs. The discussion and explanatory material presented here are intended to assist in making the best use of this interim guide.

1.3 — LIMITATIONS

This guide is based on empirical relationships derived from the AASHO Road Test, supplemented by theory and by data developed from current practices of highway construction agencies. Although a revision of the Guides distributed in 1962, it is still considered to be interim in nature, and it is expected that it will be subject to periodic review and revision as may be found necessary through practice and further research. It is essential that the users of this Guide understand its basic limitations.

1.4 — ROADBED SOILS

A pavement structure is a layered system designed to distribute concentrated traffic loads to the subgrade. Preparation of the subgrade usually includes at least grading and compaction of the roadbed soils, and may include other means of providing for optimum support of the pavement structure.

The performance of a pavement structure is directly related to the physical properties and condition of the roadbed soils. The design procedures in this guide are based on the assumption that most soils can be adequately represented for pavement design purposes by means of the soils support value (S) for flexible pavements or the modulus of subgrade reaction (k) for rigid pavements, which compensate for poorer soils by increasing the thickness of the pavement structure. However, certain soils such as those that are excessively expansive, resilient, frost susceptible, or highly organic, require that additional steps be taken to provide for adequate pavement performance.

Other problems related to roadbed soils are the nonuniform support that results from wide variations in soil type or condition; the additional densification under traffic of soils that are not adequately compacted during construction; and construction difficulties, particularly those associated with compaction of cohesionless sands and wet, highly plastic clays.

With these problems in mind, it is recommended that the following be considered during design, and appropriate modifications be made in the specifications, when applicable:

1. The basic criteria for compaction of roadbed soils should include an appropriate density requirement. Inspection procedures should be adequate to assure that the specified density is attained during construction.

2. Soils that are excessively expansive or resilient should receive special consideration. One solution is to cover these soils with a sufficient depth of selected material to overcome the detrimental effects of expansion or resilience. Expansive soils may often be improved by compaction at water contents somewhat over the optimum. In some cases it may be more economical to treat expansive or resilient soils by stabilizing with a suitable admixture, such as lime or cement, or to encase a substantial thickness in a waterproof membrane to stabilize the water content.

3. In areas subject to frost, pockets of frost-susceptible soils may be removed and replaced with selected, nonsusceptible material. Where such soils are too extensive for economical removal, they may be covered with a sufficient depth of suitable material to overcome the detrimental effects of freezing and thawing. The need for such measures and the type and thickness of material required must be determined on the basis of local experience and types of materials economically available.

4. Problems with highly organic soils are related to their extremely compressible nature, and are accentuated when deposits are extremely non-uniform in properties or depth. Local deposits, or those of relatively shallow depth, are often most economically excavated and replaced with suitable selected material. Problems associated with deeper and more extensive deposits have been alleviated by placing surcharge embankments for preconsolidation, sometimes with special provisions for rapid removal of water to hasten consolidation.

5. Special provisions for unusually variable soil types and conditions may include: scarifying and recompacting; treatment of an upper layer of roadbed soils with a suitable admixture; using appreciable depths of more suitable roadbed soils; overexcavation of cut sections, and placing a uniform layer of selected material in both cut and fill areas; or adjustment in the thickness of subbase at transitions from one soil type to another, particularly when the transition is from cut to fill section.

6. Although the design procedure is based on the assumption that provisions will be made for surface and subsurface drainage, unusual situations may require that special attention be given to design and construction of drainage systems. Drainage is particularly important where heavy flows of water are encountered (i.e., springs or seeps); where detrimental frost conditions are present; or where soils are particularly susceptible to expansion or loss of strength with increase in water content. Special subsurface drainage may include provision of additional layers of permeable material beneath the pavement for interception and collection of water, and pipe drains for collection and transmission of water. Special surface drainage may require such facilities as dikes, paved ditches, and catch-basins.

7. Certain roadbed soils pose difficult problems in construction. These are primarily the cohesionless soils, which are readily displaced under equipment used to construct the pavement; and wet clay soils, which cannot be compacted at high water contents because of displacement under rolling equipment and require long periods of time to dry to a suitable water content. Measures that have been applied to alleviate such construction problems include: blending with other soils or adding suitable admixtures to sands to provide cohesion, or to clays to hasten drying or increase shear strength; and covering with a layer of more suitable selected material to act as a working platform for construction of the pavement.

1.5 – GENERAL REQUIREMENTS, BASE AND SUBBASE COURSES

When coarse, open-graded subbase or base courses are used, it may be necessary to provide a means for preventing the intrusion of the underlying fine-grained roadbed soils. Preventive measures usually consist of providing a layer of suitable material to act as a barrier between the roadbed soils and the susceptible subbase or base course. A minimum thickness of 4 inches (100mm) is usually considered as adequate for this purpose. The need for preventive measures, as well as the suitability of materials to act as a barrier, may be evaluated by criteria established by the U. S. Corps of Engineers.(4) These criteria suggest that detrimental intrusion may occur when the ratio (D_{15}/D_{85}) is greater than about 5, where:

D_{15} = particle size wherein 15 percent of the *base or subbase course* particles are smaller than this size.

D_{85} = particle size wherein 85 percent of the *roadbed soil* particles are smaller than this size.

In areas subject to frost action, special consideration should be given to the requirements for subbase and base materials to reduce their susceptibility to detrimental frost action. Local experience is usually the best means for establishing suitable special criteria for subbase and base materials in such areas. One of the most common special criteria consists of modification of the grading requirements to reduce the percentage of fines, or treatment with a suitable admixture.

The following is provided as a guide to specification requirements for compaction of subbase and base courses:

1. Untreated aggregate base and subbase courses should be compacted to a satisfactory density determined by standard methods of test, such as AASHTO Designations T-99 or T-180.

2. Cement-treated and lime-treated subbase and base courses should be compacted to a satisfactory density determined by the standard method of test, AASHTO Designation T-134.

3. Asphalt-treated subbase and base courses should be compacted to a satisfactory density based on the test method used to determine the stability of the mixture; i.e., the Hveem Stabilometer, Hubbard-Field, or Marshall.

1.6 – SERVICEABILITY INDEX

The serviceability of a pavement is defined as the ability to serve high-speed, high-volume automobile and truck traffic. For the AASHO Road Test, a procedure was developed for periodic rating of the serviceability of pavements. (1) This procedure, referred to as the Present Serviceability Rating

(PSR), consisted of the mean of individual ratings by a selected panel of men with long experience in all aspects of highway engineering, and as highway users. A scale with a range of 0 through 5 was established for present serviceability ratings, with a value of 5 as the highest index of serviceability and 0 as the lowest. A procedure was also developed for predicting the present serviceability rating from the combination of a series of physical measurements of the pavement. This combination of values was referred to as the Present Serviceability Index (p).(5)

In order to develop the basic design equations from AASHO Road Test data, it was necessary to establish the relationship between performance and pavement structural design, with performance being related to the ability to satisfactorily serve traffic over a period of time. Performance of the AASHO Road Test pavements was described in terms of the serviceability index at the time of completion of construction and at some later time subsequent to construction. This serviceability-performance concept is the basic philosophy of this design guide and pavements may be designed for the level of serviceability desired at the end of the selected traffic analysis period or after exposure to a specific total traffic volume. Selection of the terminal serviceability index (p_t) is based on the lowest index that will be tolerated before resurfacing or reconstruction becomes necessary. An index of 2.5 is suggested as a guide for design of major highways, and 2.0 for highways with lesser traffic volumes. For relatively minor highways, where economic considerations dictate that initial capital outlay be kept at a minimum, it is suggested that this may be accomplished by reducing the traffic analysis period or total traffic volume rather than by designing for p_t of less than 2.0.

1.7 – TRAFFIC

The basic equations developed from the results of the AASHO Road Test were based on traffic that consisted of multiple applications of identical vehicle loads on each of the test loops. In order to be applicable to the design of pavements, these equations must be extended to use with mixed traffic; i.e., the random mixture of vehicles with different axle loads and number of axles that constitutes normal highway traffic. The procedure used in this design Guide is to convert the varying axle loads to a common denominator, and to express traffic as the sum of the converted axle loads. The common denominator used is an 18-kip (80kN) single-axle load. Thus, traffic is expressed as equivalent 18-kip
8.17 Ton (80kN) single-axle loads. The development of the procedure for converting mixed traffic to equivalent 18-kip (80kN) single-axle loads is discussed in appendix Sections C.2 and D.2.

The prediction of traffic for design purposes must rely on information from past traffic, modified by factors for growth or other expected changes. Most states accumulate past traffic information in the form of loadometer data in the format of the Federal Highway Administration W4 loadometer tables, which are tabulations of number of axles observed within a series of load groups, with each load group usually a 2,000-lb (8.9kN) interval. These tabulations are in a convenient form for conversion, since the number of axles in each load group may be multiplied by an appropriate factor for conversion to equivalent 18-kip (80kN) single axle load applications for the load group, and a summation of these for all load groups is the equivalent 18-kip (80kN) single axle load applications that represents the total traffic for the survey period. It should be noted that the equations used in this Guide are based on the application of a maximum number of loads during a two-year period at the AASHO Road Test. Extrapolation beyond these total load applications should be used with caution, since they cannot be substantiated by Road Test experience.

Individual traffic agencies have developed rather sophisticated procedures for prediction of future traffic. Although traffic prediction techniques are outside the scope of this guide, it is suggested that predictions may also be expressed in terms of predictions for individual load groups for convenient conversion to equivalent 18-kip (80kN) single-axle loads.

Predictions of traffic are made for some convenient period of time, referred to in this Guide as the traffic analysis period. The traffic analysis period often used is 20 years, which is also a common period used in traffic predictions for geometric design. However, any period may be used with this design guide because traffic is expressed as daily or total equivalent 18 kip (80kN) single axle load applications. Regardless of the traffic analysis period used, the total equivalent 18 kip (80kN) single axle load applications is the total traffic that the pavement can be expected to carry from time of construction to the time when the serviceability is reduced to the selected value; i.e., p_t = 2.5 or 2.0. Thus, if traffic is underestimated, this time may be less than the traffic analysis period, and, conversely, if traffic is over-estimated this time can be expected to be longer. Neither the traffic analysis period nor the time a pavement reaches its 'terminal' serviceability index (p_t) should be confused with pavement life. Pavement life may be extended by periodic renewal of the surface. Also surface renewal may be necessary for reasons other than restoration of serviceability (such as renewal of antiskid properties or rejuvenation of weathered surfaces).

The equivalent axle loads derived from many prediction procedures represent the totals for all lanes for both directions of travel. This traffic must be distributed by direction and by lanes for design purposes. Directional distribution is usually made by assigning 50 percent of the traffic to each direction, unless special conditions warrant some other distribution. In regard to lane distribution, 100 percent of the traffic in each direction is usually assigned to all lanes in that direction for purposes of structural design. Some states have developed lane-distribution factors for facilities with more than one lane in a given direction. These factors vary from 80 to 100 percent of the one-direction traffic for design of all lanes when there is a total of four lanes in both directions, and from 60 to 80 percent of the one-direction traffic to one or more of the outer lanes and lesser values to inner lanes when there are six or more lanes in both directions. If there is doubt as to which factor to apply, it is suggested that the highest (most conservative) range be used.

Chapter II — INTERIM GUIDE FOR THE DESIGN OF FLEXIBLE PAVEMENT STRUCTURES

2.1 — INTRODUCTION

A flexible pavement structure may consist of three layers, designated as subbase course, base course, and surface course. The subbase often consists of materials of lower quality than the base course. Where the required pavement structure is relatively thin, the subbase is often omitted. The design procedure includes the determination of total thickness of the pavement structure, as well as of the thickness of the individual components of surface, base and subbase courses. Provision is made for the design of equivalent alternate sections, with the selection of the alternate being primarily a function of availability of materials and comparative costs.

2.2 — LIMITATIONS

It is essential that the users of the design procedure in the guide understand its basic limitations. Briefly stated, these limitations are as follows:

1. It has been necessary to develop an empirical soil support scale with values from 1 to 10. Point 3.0 on the soil support scale represents the silty clay roadbed soils on the AASHO Road Test, and is a firm and valid point. Point 10.0, representing crushed rock base material such as used on the Road Test, is a reasonably valid point. All other points on the scale were assumed from experience, and checked through theoretical computations.

2. The soil support value must be correlated with the soil test or identification method selected for use by the user agency. Suggested procedures for developing such correlations, and an example, are presented in Section C.3 of Appendix C.

3. The structural number (SN) determined by this design procedure must be converted to actual thickness of surfacing, base, and subbase layer by assigning a layer coefficient (a_1, a_2, a_3) to represent the relative strength of the material actually used for each layer. Analysis of the AASHO Road Test data established such layer coefficients for the particular materials (asphaltic concrete surfacing, crushed stone base, and sandy gravel subbase), and for the specific designs, construction standards, environmental conditions, and traffic exposure represented by the Test. Careful consideration must be given by user agencies in selecting applicable coefficients. Suggested procedures for selection of coefficients are presented in Section C.4 of Appendix C.

4. Included in the design procedure is a regional factor, (R) which provides an adjustment in the structural number for local environmental and other considerations. Suggested procedures by which the user agency may select appropriate regional factors are presented in Section 2.4.4 below. (It should be noted that the regional factor and the design procedure outlined in this guide may not adjust for special conditions, such as serious frost conditions or other local problems.)

5. A basic traffic analysis period must be selected. The traffic analysis period should not be confused with pavement life, which is affected by other factors in addition to traffic. A detailed discussion of traffic analysis was presented in Section 1.8.

2.3 — MATERIALS PROPERTIES AND SPECIFICATIONS

2.3.1 — Subbase Course

The subbase course is the portion of the flexible pavement structure between the subgrade and the base course. It usually consists of a compacted layer of granular material, either treated or untreated, or of a layer of soil treated with a suitable admixture. In addition to its position in the pavement, it is usually distinguished from the base course material by less stringent specification requirements for strength, plasticity, and gradation. Because it is obvious that the subbase course must be of significantly better quality than the roadbed soil, the subbase is often omitted if roadbed soils are of high quality.

When roadbed soils are of relatively poor quality and the design procedure indicates the requirement for substantial thickness of pavement, alternate designs should be prepared for structural sections with and without subbase. The selection of an alternate may then be made on the basis of availability and relative costs of materials suitable for base and subbase. Because lower quality materials may be used in the lower layers of a flexible pavement structure, the use of a subbase course is often the most economic solution to construction of pavements over poor roadbed soils.

No specific quality requirements for subbase material are presented in this guide, because many different types of material have been used successfully. However, for use in this design procedure, subbase material, if present, requires the use of a layer coefficient (a_3), in order to convert its actual thickness to structural number (SN). Special consideration must be given to determine the minimum thickness of base and surfacing required over a given subbase material. Procedures that may be used for this purpose are given in Section C.4 of Appendix C. Procedures for assigning appropriate layer coefficients are given in Section 2.4.4 below.

In addition to the major function as a structural portion of the pavement, subbase courses may have additional secondary functions, such as:

1. To prevent the intrusion of fine-grained roadbed soils into base courses; relatively dense-graded materials must be specified if the subbase is intended to serve this purpose.

2. To minimize the damaging effects of frost action. Materials not susceptible to detrimental frost action must be specified if the subbase is intended to serve this purpose.

3. To help in preventing the accumulation of free water within or below the pavement structure. Relatively free-draining material must be specified if the subbase is intended to serve this purpose, and provisions must be made for collecting and removing the accumulated water from the subbase.

4. To provide a working platform for construction equipment.

2.3.2 — Base Course

The base course is the portion of the flexible pavement structure immediately beneath the surface course. It is constructed on the subbase course, or, if no subbase is used, directly on the roadbed soil. It performs its major function as a structural portion of the pavement. It usually consists of aggregates such as crushed stone, crushed slag, crushed or uncrushed gravel and sand, or of

Table II-1

Some Typical Specification Requirements for Stabilized Base Courses

Specification	Cement Treated			Bituminous Treated		Lime Treated
	Class A	Class B	Class C	Class 1	Class 2	
Sieve Analysis, % Passing mm						
63 (2¼")	100	100	100			
19.0 (¾")	–	–	75-95			
4.75 (No. 4)	65-100	55-100	25-60			
2.00 (No. 10)	20-45	–	15-45			
0.425 (No. 40)	15-30	25-50	8-30			
0.075 (No. 200)	5-12	5-20	2-15			
Compressive Strength,* psi at 7 days	650-1000 4.5-6.9MPa	300-650 2.1-4.5MPa				
Soil Support Value (S)	–	–	8.0 min.			10.0 min.
Stability						
Hveem Stabilometer				35 min.	25 min.	
Hubbard-Field				1200 min.	1000 min.	
Marshall-stability				750 min.	500 min.	
Marshall-flow				16 max.	20 max.	
Plasticity Index**	12 max.			6 max.	6 max.	6 max.

* As determined in unconfined compression tests on cylinders 4 inches (102mm) in diameter and 4 inches (102mm) high. Test specimens should contain the same percentage of portland cement, and be compacted to the same density as achieved in construction.

** Performed on samples prepared in accordance with AASHTO Designation T-87, and apply to aggregate prior to mixing with the stabilizing admixture, except that in the case of lime treated base the value applies after mixing.

combinations of these materials. It may be used untreated or treated with suitable stabilizing admixtures such as portland cement, asphalt, or lime. Specifications for base course materials are generally considerably more stringent than for subbase materials in requirements for strength, plasticity, and gradation.

Although no specific quality requirements for base courses are presented in this guide, the specifications presented in AASHTO Designations M-147 and M-75 are typical of the gradation and quality of untreated base aggregates often used. Materials varying in grading and quality from these specifications have been used in certain areas, and have provided satisfactory performance. Additional requirements for quality of base materials, based on test procedures used by the constructing agency, may also be included in materials or construction specifications.

A wide variety of materials unsuitable for use as untreated base course have given satisfactory performance when improved by addition of a stabilizing admixture such as portland cement, asphalt, or lime. Consideration should be given to the use of such treated materials for base courses whenever they are economically feasible, particularly when suitable untreated materials are in short supply. Economic advantages may result not only from the use of low-cost aggregates, but also from possible reduction in the total thickness of the pavement structure that may result from the use of treated materials. Table II-1 is a guide to establishment of specification requirements for stabilized base courses. Careful study is required in the selection of the type and amount of admixture to be used for optimum performance and economy.

For use in this design procedure, base material must be represented by a layer coefficient, in order that its actual thickness may be converted to a structural number, and, if the method in Section C.5 of Appendix C is used, by a soil support value, in order to determine the minimum structural number that must be provided by the surface course.

2.3.3 – Surface Course

The surface course of a flexible pavement structure consists of a mixture of mineral aggregates and bituminous materials, placed as the upper course and usually constructed on a base course. In addition to its major function as a structural portion of the pavement, it must also be designed to resist the abrasive forces of traffic, to reduce the amount of surface water penetrating the pavement, to provide a skid-resistant surface, and to provide a smooth and uniform riding surface.

The success of a surface course depends to a considerable degree on obtaining a mixture with the optimum gradation of aggregate and percent of bituminous binder to be durable and to resist fracture and raveling, without becoming unstable under expected traffic and climatic conditions. The use of a laboratory design procedure is essential to insure that a mixture will be satisfactory.

Although dense-graded aggregates with a maximum size of about 1 *inch* (25mm) are most commonly specified for surface courses for highways, a wide variety of other gradations, from sands to coarse, open-graded mixtures, have been used and have provided satisfactory performance for specific conditions. Surface courses are usually prepared by hot plant mixing with an asphalt cement, but satisfactory performance has also been obtained by cold plant mixing, or even mixing in place, with liquid asphalts or asphalt emulsions.

Construction specifications usually require that a liquid bituminous material be applied on untreated aggregate base courses as a prime coat, and on treated base courses and between layers of the surface course to serve as a tack coat.

No specific quality requirements for surface courses are presented in this guide. Table II-2 is a guide to establishment of specification requirements for surface courses, and is based on typical requirements of many highway agencies in the United States. It is recognized that each agency will prepare specifications that are based on performance, local construction practices, and the most economical use of local materials.

Table II-2

Typical Values of Criteria for Design of Bituminous Mixtures

	Level of Traffic Equivalent Daily 18-kip (80kN) Axle Loads		
	500 to 3000*	50 to 500*	1 to 50*
Hveem Stability[1]			
Stabilometer value	35 min.	35 min.	30 min.
Cohesiometer value at 60°C	50 min.	50 min.	50 min.
Swell, inches (mm)	.03 max (0.76)	.03 max. (0.76)	.03 max. (0.76)
Voids in Total Mix, percent	4 min.	4 min.	4 min.
Hubbard Field[2]			
Stability percent	2000 min.	1200 to 2000	1200 to 2000
Voids in Total Mix, percent	2 to 5	2 to 5	2 to 5
Marshall[3]			
Stability, lbs.	750 min.	500 min.	500 min.
Flow, 0.01 inch (0.25mm)	8 to 16	8 to 18	8 to 20
Voids in Total Mix, percent			
Surfacing & Leveling	3 to 5	3 to 5	3 to 5
Sand or Stone Sheet	3 to 8	3 to 8	3 to 8
Binder or Base	3 to 11	3 to 11	3 to 11
Aggregate Voids Filled, percent			
Surfacing & Leveling	75 to 82	75 to 85	65 to 85
Sand or Stone Sheet	65 to 72	65 to 75	65 to 75
Binder or Base	65 to 72	65 to 75	65 to 75
Compaction, number of blows at each end of test specimen	75	50	35
Immersion – Compression[4]			
Voids in total mix, percent	7.0	6.0	6.0
Compressive strength, psi (MPa)	300 min. (2.1)	250 min. (1.7)	150 min. (1.0)
Retained strength, percent	70 min.	70 min.	70 min.
Dust–asphalt ratio	1.2 max.	1.2 max.	1.2 max.

* Daily load applications are based on a 20 year traffic analysis period.
[1] AASHTO Designation T190; ASTM Designation D1560.
[2] AASHTO Designation T169; ASTM Designation D1138.
[3] ASTM Designation D1559.
[4] AASHTO Designation T165; ASTM Designation D1075. (Also see ASTM STP No. 252, pp 113-129).

It is particularly important that surface courses be properly compacted during construction. Improperly compacted surface courses are more likely to exhibit a variety of types of distress that tend to reduce the life and over-all level of performance of the pavement. Types of distress that are often related to insufficient compaction during construction include: rutting resulting from further densification under traffic, structural failure resulting from excess infiltration of surface water through the surface course, and cracking or raveling of the surface course resulting from embrittlement of the bituminous binder by exposure to air and water in the mixture. Specific criteria for compaction are expected to be established by each highway agency, based on local experience and requirements. It is suggested that a density of 95 to 100 percent of that developed in the laboratory test method used to design the surface course mixtures be used as a guide to establishing density criteria.

2.4 – DEVELOPMENT OF THE FLEXIBLE PAVEMENT DESIGN PROCEDURE

The procedure presented herein as a guide for the design of flexible pavement structures is based on data developed by the AASHO Road Test, supplemented and modified by data from other road tests, from other design procedures, and from theoretical relationships developed in recent research. The design procedure is presented in this guide in the form of nomographs (Figures II-1 and II-2) for ease in solution of the design equation. The equations represented by these nomographs were developed on the basis of the following assumptions:

1. That the basic equations developed from the AASHO Road Test are a valid representation of the relationship between loss in serviceability, traffic, and pavement thickness. In these equations, loss in serviceability is expressed in terms of reduction of serviceability index: traffic is converted to equivalent 18-kip (80kN) single-axle load applications; and pavement thickness is represented by a structural number.

2. That the basic equations developed from the AASHO Road Test for a single type of roadbed soil may be extended to apply to any roadbed soil by means of an abstract soil support scale developed for this purpose.

3. That the basic equations developed from the AASHO Road Test for repeated applications of uniform traffic loads may be extended to apply to mixed traffic by conversion to equivalent 18-kip (80kN) single-axle loads.

4. That the basic equations developed from the AASHO Road Test for a single environmental condition may be extended to apply to other environmental conditions by means of an appropriate Regional Factor.

5. That the basic equations developed from the AASHO Road Test for the subbase, base, and surfacing materials used in constructing the Test Road may be extended to apply to other materials by assignment of appropriate layer coefficients (a_1, a_2, a_3).

6. That the basic equations developed from the AASHO Road Test for accelerated applications of traffic during the two-year test period may be extended to apply to applications of traffic during an extended period of time (up to 20 years).

7. That uniform and high-quality construction will be obtained, particularly with respect to density, gradation, and quality of materials, and smoothness of the pavement surface, both transversely and longitudinally.

The development of the design equations from the basic AASHO Road Test equations is presented in detail in Section C.1 of Appendix C. The use of this equation in the design of flexible pavement structures requires an evaluation of the following for the expected conditions: Terminal Serviceability Index (p_t), Equivalent 18-Kip (80kN) Single-Axle Loads, Soil Support Value (S), Regional Factor (R), Structural Number (SN), and Layer Coefficient (a_1, a_2, a_3). A guide to the evaluation of each is presented in the following sections.

2.4.1 – Soil Support

The basic equation developed from the results of the AASHO Road Test is valid for only one value of soil support, that representing the roadbed soils at the road test site. The embankment on which the AASHO test sections were constructed consisted of 3 feet (0.9m) of A-6 Soil, acquired from adjacent borrow pits. This soil was placed in 4-inch (100mm) lifts, and compacted to an average AASHTO T-99 desnity of 97.7 percent. Approximately 80 percent of all tests were within the specified desnity range of 95 percent to 100 percent. Approximately 11 percent exceeded the maximum specified density and 9 percent were below the specified desnity. The average moisture at the time of compaction was slightly above the 15 percent optimum. Approximately 83 percent of all moisture tests were within the specified ± 2 percent of optimum moisture. More than 16 percent were above the specified maximum and slightly less than 1 percent were below the minimum range. For design purposes, it is necessary to assume a soil support value which takes into account the variation in density and moisture which can be expected in normal construction practices. This variation will be influenced by the quality control exercised. In order to make the design procedure applicable to other roadbed soils, it was necessary to assume a soil support scale to represent the variety of soils that would be encountered at other sites.

To obtain a first point on this scale, the AASHO Road Test roadbed soils were assigned a value of 3.0. To obtain a second point, the performance of AASHO Road Test pavements with a base of sufficient thickness to minimize the effect of the roadbed soils was studied. Selected for study were several sections of the loop carrying 18-kip (80kN) single-axle loads with the greatest thickness of crushed-rock base course. These studies indicated that about 4½ inches (114mm) of surfacing on a substantial thickness of crushed-rock base would carry approximately 1,000 18-kip (80kN) single-axle load applications per day for a 20-year traffic analysis period, with the serviceability index maintained at or above 2.0 for the entire period. This information was used to plot a second point on the soil support scale. This point was labeled 10.0, and assumed to be representative of the highest soil support values that might be encountered, and, thus, the upper limit of the scale. A linear scale between points 3.0 and 10.0 was then assumed, and the scale was also extended downward linearly.

A check on the assumption of linearity of the soil support scale has been made by application of layered elastic theory (see Section C.1, Appendix C). The results of the calculations described in Section C.1 were used to develop a theoretical soil support scale, and this theoretical scale was compared to the scale assumed from the AASHO Road Test. It was concluded that the assumption of the linear soil support scale is reasonable, and that the range of values in the scale also checks reasonably well with the calculated values.

The units of soil support, represented by the assumed soil support scale,

have no direct relationship to any procedure for testing soils. Therefore, it is necessary that each design agency establish a correlation between soil support and some testing procedure before this guide is used for design of pavements. Section C.3 of Appendix C discusses the manner by which one state has established such correlations for its own use. It is suggested that each design agency establish correlations applicable to its own practices and based on its own experience. Although maximum use should be made of developmental work by other agencies, careful consideration should be given before adopting correlations developed by others.

2.4.2 – Regional Factor

The regional factor was included in this design guide equation to make it applicable for design of pavements in areas with climatic and environmental conditions different from those at the AASHO Road Test site. The adjustment is made by means of a separate scale for modifying the structural number.

It is generally recognized that when conditions are adverse, such as during a period of strength loss of the roadbed materials which may occur during spring thaw, there will be greater damage inflicted to the pavement by traffic than during more favorable conditions. This variation in rate of reduction of serviceability with season has been averaged for the AASHO Road Test period to arrive at an approximate regional factor for the AASHO Road Test.(6) The seasonal values varied between 0.1 and 4.8, and with an annual value of regional factor of about 1.0. The lower values apply to both the solidly frozen and the relatively dry conditions of roadbed soils when the rate of loss of serviceability was very low, and the higher values apply to spring conditions at the AASHO Road Test site when roadbed soils were weakened and rate of loss of serviceability was highest.

At present, there is no way to determine directly the regional factor for other locations and conditions. It may be estimated, as it was for AASHO Road Test conditions, by analyzing the duration of certain conditions during a typical year. Based on AASHO Road Test information, values that may be used as a guide for such an analysis are:

Roadbed materials frozen to depth of 5" (130mm) or more	0.2 to 1.0
Roadbed materials dry, summer and fall	0.3 to 1.5
Roadbed materials wet, spring thaw	4.0 to 5.0

Many other procedures have been used to estimate regional factors. A survey of all 50 states(7) indicated that one or more of the following are used by states in assigning a regional factor:

1. Topography
2. Similarity to Road Test location
3. Rainfall
4. Frost penetration
5. Temperature
6. Groundwater table
7. Subgrade type
8. Engineering judgment
9. Type of highway facility
10. Subsurface drainage

There are other conditions, somewhat related to the above, that may require consideration in establishing a Regional Factor, such as:

1. Number of annual freeze-thaw cycles
2. Steep grades with large volume of heavy truck traffic
3. Areas of concentrated turning and stopping movements

In general, the regional factor should not exceed about 4.0, or be less than about 0.5 for conditions in the United States. The regional factor may not adjust for special conditions, such as serious frost conditions, or other local problems.

Even with the various guidelines presented above, considerable judgment must still be exercised in evaluating their effects and in selecting an appropriate regional factor for design. The regular use of a pavement rating system would provide valuable background data for determining a regional factor.

2.4.3 – Structural Number

The solution of the design equation presented in this guide is in terms of a structural number (SN). The structural number is an abstract number expressing the structural strength of pavement required for a given combination of soil support value, total equivalent 18-kip (80kN) single-axle loads, terminal serviceability index, and regional factor. The required SN must be converted to actual thickness of surfacing, base, and subbase by means of appropriate layer coefficients representing the relative strength of the material to be used for each layer. (See Section 2.4.4 for a discussion of layer coefficients.)

The design equation is used to solve for total required SN for the entire pavement, and also, if the procedure outlined in Section C.5 of Appendix C is followed, separately, to determine the minimum SN required for surfacing and base. By solving the equation with soil support value representative of the roadbed soil, an SN for the entire pavement is obtained and is represented by the general equation:

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

where

a_1, a_2, a_3 = layer coefficients representative of surface, base and subbase course, respectively.

D_1, D_2, D_3 = actual thickness, in inches, of surface, base, and subbase courses, respectively.

2.4.4 – Layer Coefficients

As mentioned in the previous section, a coefficient must be assigned to each material used in the pavement structure in order to convert structural number to actual thickness. This layer coefficient expresses the empirical relationship between SN and thickness, and is a measure of the relative ability of the material to function as a structural component of the pavement.

Average values of layer coefficient for the materials used in the AASHO Road Test pavements were determined from the results of the test, and were as follows:

Asphaltic concrete surface course	0.44
Crushed stone base course	0.14
Sandy gravel subbase course	0.11

These values are a valid representation of materials similar to those of the AASHO Road Test, when used in a similar position in the pavement and in similar relative thicknesses. Values of coefficients for other materials have been developed on the basis of field experience, other road tests, and by application of layered elastic theory.

Table II-3 summarizes the results of a survey(7) of the development and extent of use of structural layer coefficients for the various materials used by the states in pavement construction. In most cases, a layer coefficient value, or a range of values, is assigned on the basis of a description of a material type. A few states evaluate or measure the coefficient by means of a laboratory test on the material. Several states also vary the coefficient with the position of the material in the pavement structure.

Because of widely varying environments, traffic, and construction practices, it is suggested that each design agency establish layer coefficients applicable to its own practices and based on its own experience. Careful consideration should be given before adoption of values developed by others. However, Table II-3 and the results of theoretical analyses such as included in Section C.4 of Appendix C, may be used as a guide to ranges of coefficients and to procedures for their determination.

2.5 – USE OF THE DESIGN CHARTS

The flexible pavement design equation presented in this guide was developed from the basic AASHO Road Test equation, extended as discussed in the previous sections. The design equation is presented in the form of two nomographs for simplicity of application. Separate nomographs are presented for a terminal serviceability index (p_t) of 2.5 (Figure II-1) and of 2.0 (Figure II-2). Figure II-1 is intended for use in designing major highways, and assumes that resurfacing or reconstruction will be performed when the level of serviceability reaches 2.5. Figure II-2 may be used for other highways where a somewhat lesser level of serviceability (2.0) may be tolerated. For design of temporary highways or for stage construction it is suggested that an appropriate traffic analysis period be used.

Once the decision has been made relative to the terminal serviceability index (p_t), and the appropriate design chart has been selected, determinations should be made of the following:

1. Representative values of Soil Support for the roadbed soil.
2. The total or daily equivalent 18-kip (80kN) single-axle loads estimated for the design lane for the traffic analysis period. Because selection of the traffic equivalence factors to be used to convert mixed traffic to total equivalent 18-kip (80kN) single-axle loads depends on the structural number, a structural number must be assumed for the initial conversion. The use of an SN of 3 for the determination of 18-kip (80kN) single-axle traffic equivalence factors will normally give results that are sufficiently accurate for design purposes even though the final SN determined is substantially different. This assumption will

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Table II-3 SUMMARY OF STRUCTURAL COEFFICIENTS USED FOR DIFFERENT PAVEMENT COMPONENTS

COMPONENT	ALABAMA	ARIZONA	DELAWARE	MASSA-CHUSETTS	MINNESOTA	MONTANA	NEVADA	NEW HAMPSHIRE
SURFACE COURSES								
Plant Mix (high stab.)	0.44	.35-.44	.35-.40	0.44	0.315	.30-.40	.30-.35	0.38
Road Mix (low stab.)	0.20	.25-.38				0.20	.17-.25	0.20
Sand Asphalt	0.40	.25			Plant mix (low stab.) 0.28			0.20
Base Courses								
Untreated	limestone 0.14 slag 0.14 sand-stone 0.13 granite 0.12	sand & gravel, well graded 0.14 cinders .12-.14 sandy gravel, mostly sand .11-.13	Waterbound Macadam 0.20 crusher run .14 quarry waste 0.11 select borrow 0.08	crushed stone .14	crushed rock (Cl. 5 & 6 gravel) 0.14 sandy gravel 0.07	select surf 0.10 crushed gravel 0.12-0.14	crushed gravel .10-.12 crushed rock .13-.16	crushed gravel 0.10 bank run gravel .07 crushed stone 0.14
Cement Treated 650 psi or more	0.23	600 psi (3.5MPa) .25-.30	soil-cement			400 psi (2.8MPa) or more 0.20		gravel
400 to 650 psi	0.20	300-500 psi (2.1-3.5MPa) .18-.25	0.20					0.17
400 psi or less	0.15	less than 300 psi (2.1MPa) 0.15				.15		
Lime Treated						.15-.20		
Bituminous Treated	Coarse graded 0.30 sand 0.25	sand-gravel .25-.34 sand .20	asph. stab. .10	black base 0.34 penetrated crushed stone .29	0.175-0.21	plant mix 0.30 bit. stab. 0.20	plant mix .25-.34	bit. conc. 0.34 gravel 0.24
SUBBASE	sand & sandy clay 0.11 chert low P.I. 0.10 top soil 0.09 float gravel 0.09 sand & silty clay 0.05	sand-gravel, well graded 0.14 cr. stone or cinders 0.12 sand & silty clay .05-.10	select borrow 0.08	gravel 0.11 select material 0.08	sandy gravel (Cl. 3 & 4 gravel) 0.105 selected granular (12% minus 0.075mm (#200)) 0.07	sand 0.05 sp. borrow 0.07	gravel type 1 .09-.11 select material .05-.09	sand-gravel 0.05

Notes:

1. Indiana, Iowa, Montana, New Jersey, Tennessee, and Puerto Rico – conform to AASHTO Guides
2. North Carolina – conforms to AASHTO Guides, except 0.30 for Bituminous Treated Base
3. North Dakota – conforms to AASHTO Guides, except 0.30 for Bituminous Aggregate Base
4. Maine – conforms to AASHTO Guides with some modification. No further information
5. Maryland – substitution values for materials to replace design thickness of asphalt hot-mix are the AASHTO structural coefficients expressed in equivalent values, in inches

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Table II-3 SUMMARY OF STRUCTURAL COEFFICIENTS USED FOR DIFFERENT PAVEMENT COMPONENTS (Continued)

COMPONENT	NEW MEXICO	OHIO	PENNSYL-VANIA	SOUTH CAROLINA	SOUTH DAKOTA	UTAH	WISCONSIN	WYOMING
SURFACE COURSES								
Plant Mix (high stab.)	.30-.45	0.40	0.44	0.40	.36-.42	0.40	0.44	.30-.40
Road Mix (low stab.)	0.20	A.C. Interim	0.20			0.20	0.20	
Sand Asphalt			0.35	AC binder 0.35			0.40	
Plant Mix Seal	.25					0.40		
BASE COURSES								
Untreated	.10-.15 quarry rock crushed rock .06-.12	aggregate 0.14 w.b. macadam 0.14	crushed stone .14 dense grade 0.18	crushed rock 0.14		0.12	crushed gravel 0.10 w.b. macadam .15-.20 sand-gravel 0.07	all untreated 0.05-0.12
Cement Treated 650 psi (4.5MPa) or more	0.23		soil cement 0.20				0.23	cement treated .15-.25
400 to 650 psi (2.8 to 4.5MPa)	0.17		cement aggr. plant mix 0.30			0.20	0.20	
400 psi (2.8MPa) or less	0.12						0.15	
Lime Treated	.05-.10		Soil-lime .20		0.15		.15-.30	.07-.12
Bituminous Treated	plant mix 0.30 road mix 0.15		soil bit. 0.20 plant mix 0.30	black base 0.30 sand 0.25	hot mix .30 coarse sand 0.24 fine sand 0.18 cold mix coarse .14 fine 0.10	coarse graded 0.30	coarse graded .34 sand 0.30 coarse H.M. 0.23	plant mix 0.20-0.30 emulsion 0.12-0.20
SUBBASE	aggregate .06-.12 borrow .05-.10	0.11	sand-gravel 0.11		sand-silty clay 0.07 3% lime .06 over 3% lime .05	sand & gravel 0.10 sand or sandy clay .06-.10	sand gravel. .11	special borrow & subbase 0.05-0.12

usually result in an overestimation of 18-kip (80kN) equivalent single-axle load applications but the resulting error in SN is not significant.

3. The regional factor applicable to the site.

The chart requires two applications of a straightedge for each solution. First, the soil support value of the roadbed soil (on the left scale) and the total or daily equivalent 18-kip (80kN) single-axle loads for the traffic analysis period (left side of second scale) are used to solve for the unweighted structural number (center scale). This unweighted structural number is used with the selected regional factor (4th scale) to solve for the design SN (right scale) applicable to the total pavement structure. Suitable designs are those whose combinations of materials types and thicknesses satisfy the general equation:

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

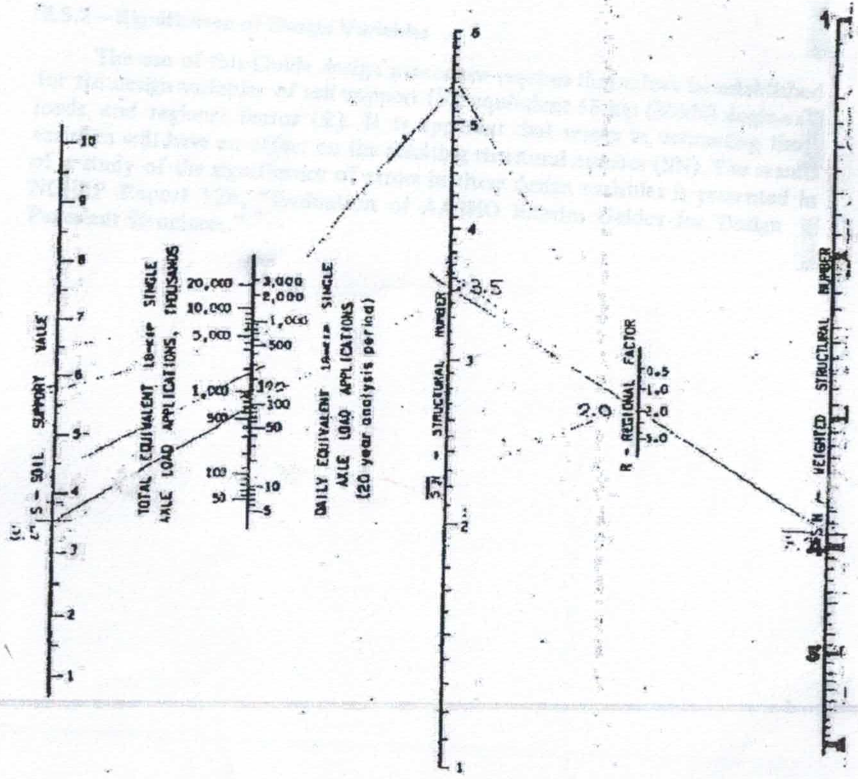


Figure II-1 Design Chart for Flexible Pavements, p_t = 2.5

0.5 < 500
1.0 500 – 1000
1.5 1000 – 2000
2. > 2000

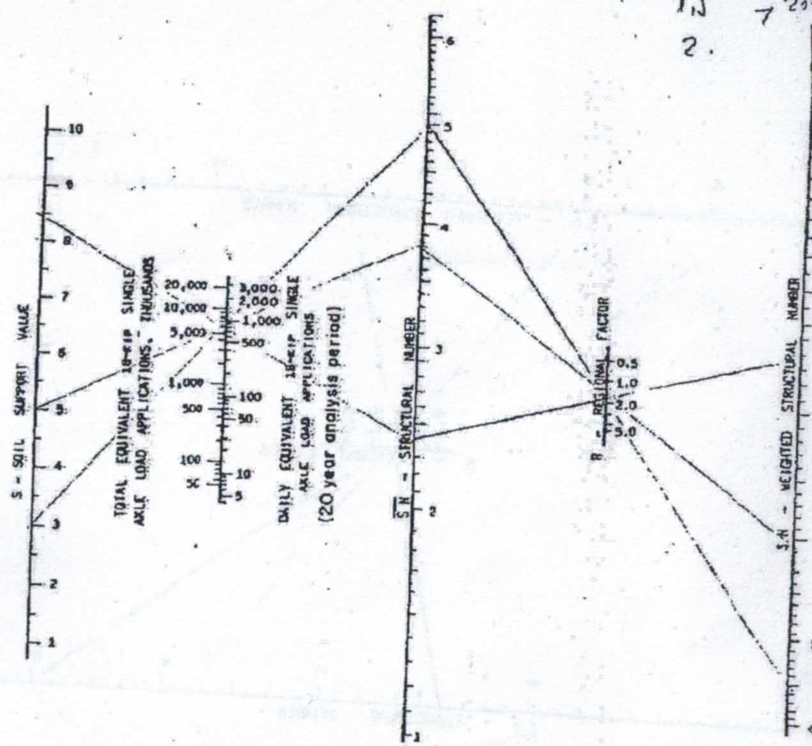


Figure II-2 Design Chart for Flexible Pavements, p_t = 2.0

where:

a_1, a_2, a_3 = layer coefficient for surface, base and subbase course materials, respectively

D_1, D_2, D_3 = thickness of surface, base and subbase courses, respectively, in inches

SN = structural number for the total pavement structure

If it is desired to consider the use of available alternate types of material for one or more of the pavement courses, the procedure outlined above may be used to prepare alternate designs of equal total weighted structural number. The resulting alternate designs may then be compared, and the optimum design may be selected on the basis of economics and other applicable consideration of construction, maintenance, or use.

2.5.1 – Minimum Layer Thickness

In any design procedure it is necessary also to consider construction and maintenance operations in order to avoid the possibility of producing impractical design. Based on these considerations, it is generally impractical a uneconomical to place surface, base or subbase courses of less than some minimum thickness. For purposes of this Guide, the following are considered to be the minimum practical thicknesses that are to be applied to each paveme course:

Surface course	2 inches (50mm)
Base course	4 inches (100mm)
Subbase course	4 inches (if subbase is used) (100m

Because such minimums depend somewhat on local practices and conditions, individual design agencies may find it desirable to modify the above minimum thicknesses for their own use.

2.5.2 – Significance of Design Variables

The use of this Guide design procedure requires that values be established for the design variables of soil support (S), equivalent 18-kip (80kN) single-ax loads, and regional factor (R). It is apparent that errors in estimating the variables will have an effect on the resulting structural number (SN). The results of a study of the significance of errors in these design variables is presented in NCHRP Report 128, "Evaluation of AASHO Interim Guides for Design Pavement Structures."

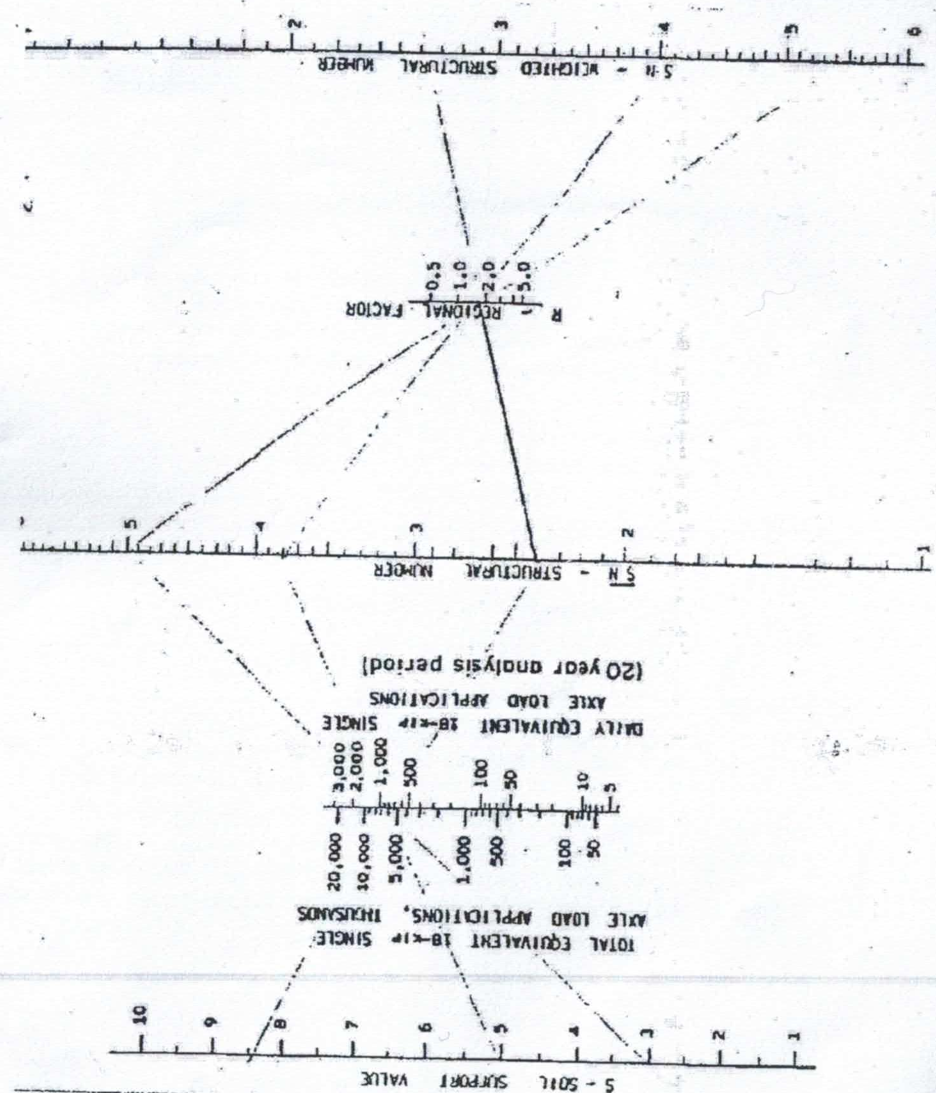


Figure II-2 Design Chart for Flexible Pavements

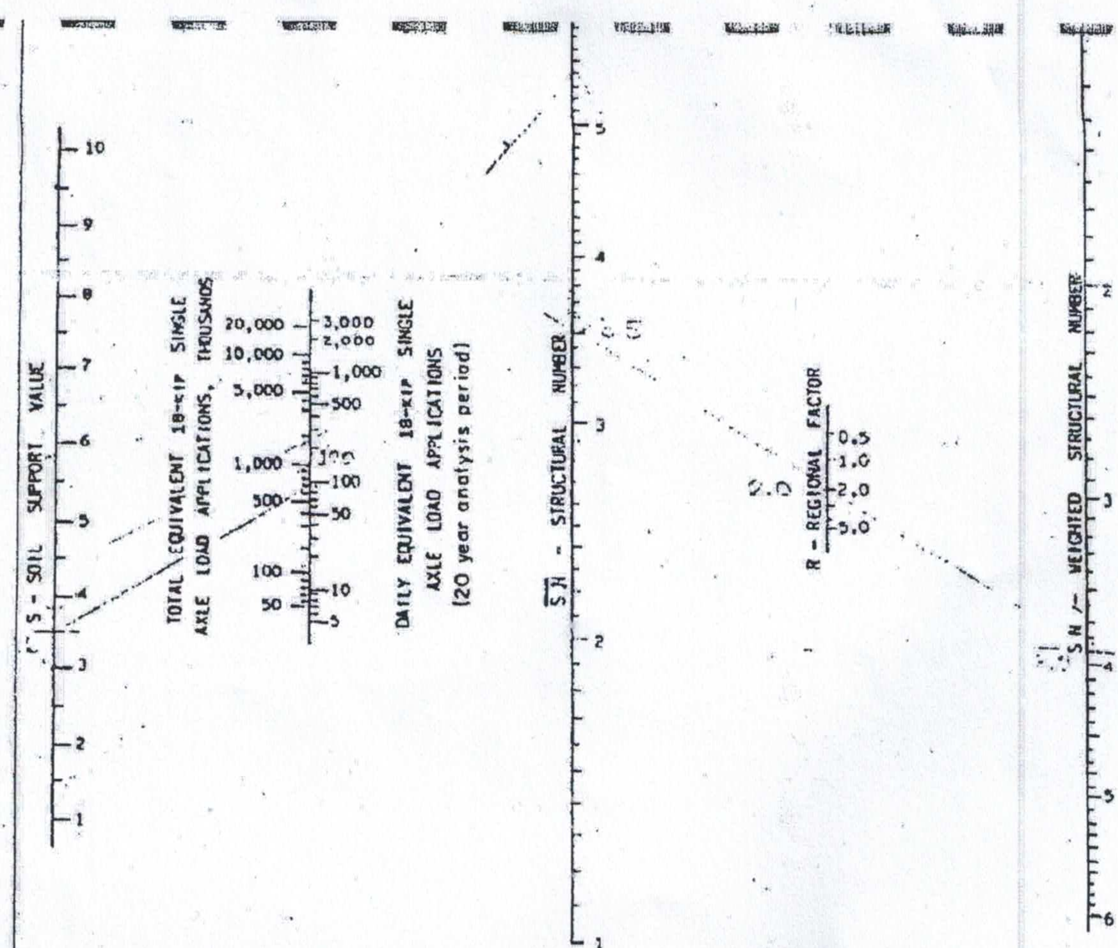


Figure II-1 Design Chart for Flexible Pavements, $p_t = 2.5$

Appendix C — DEVELOPMENT OF FACTORS PERTAINING TO DESIGN OF FLEXIBLE PAVEMENT STRUCTURES

C.1 — DEVELOPMENT OF FLEXIBLE PAVEMENT DESIGN CHARTS

The general AASHO Road Test equation is:(5)

$$G_t = \beta(\log W_t - \log \rho) \qquad \frac{G_t}{\beta} = \log \frac{W_t}{\rho} \tag{C-1}$$

where:

G_t = a function (the logarithm) of the ratio of loss in serviceability at time t to the potential loss taken to a point where $p_t = 1.5$.

β = a function of design and load variables that influence the shape of the p-versus-W serviceability curve.

W_t = axle load applications at end of the time t.

ρ = a function of design and load variables that denotes the expected number of axle load applications to a serviceability index of 1.5.

p_t = serviceability at end of time t.

At the AASHO Road Test, the terms β and ρ in equation (C-1) were related to the load and pavement variables for flexible pavements as follows:

$$\beta = 0.40 + \frac{0.081(L_1 + L_2)^{3.23}}{(\overline{SN} + 1)^{5.19} L_2^{3.23}} \tag{C-2}$$

and:

$$\log \rho = 5.93 + 9.36 \log (\overline{SN} + 1) - 4.79 \log (L_1 + L_2) + 4.33 \log L_2 \tag{C-3}$$

where:

L_1 = load on one single axle or on one tandem-axle set, kips.

L_2 = axle code (1 for single axle and 2 for tandem axle).

$\overline{SN}$ = structural number.

Since the equations for β and ρ both contain the terms L_1, L_2, and $\overline{SN}$, the solution of equation (C-1) for $\overline{SN}$ is an involved iterative process. The solution is simplified if all load factors are expressed in terms of a common denominator. The common denominator used in this guide is an 18,000-lb single-axle load and for these conditions ($L_1 = 18$ kips, $L_2 = 1$), equation (C-2) becomes

$$\beta = 0.40 + \frac{0.081(18 + 1)^{3.23}}{(\overline{SN} + 1)^{5.19}}$$

or

$$\boxed{\beta = 0.40 + \frac{1094}{(\overline{SN} + 1)^{5.19}}} \tag{C-4}$$

and equation (C-3) becomes:

$$\log \rho = 5.93 + 9.36 \log (\overline{SN} + 1) - 4.79 \log (18 + 1) + 4.33 \log 1$$

$$\text{or: } \boxed{\log \rho = 9.36 \log (\overline{SN} + 1) - 0.20} \tag{C-5}$$

Rewriting equation (C-1) as:

$$\log W_t = \log \rho + G_t/\beta \qquad \text{(C-6)}$$

and inserting equations (C-4) and (C-5) into equation (C-6):

Ecuación idem (E.7 Huang)

$$\boxed{\log W_{t_{18}} = 9.36 \log (\overline{SN} + 1) - 0.20 + \frac{G_t}{0.40 + \dfrac{1094}{(\overline{SN} + 1)^{5.19}}}} \qquad \text{(C-7)}$$

where:

$$G_t = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right)$$

$W_{t_{18}}$ = number of 18-kip (80kN) single-axle load applications to time t.

Equation (C-7) provides a basis for developing design charts involving the factors $W_{t_{18}}$, $\overline{SN}$, and p_t (values of 2.0 and 2.5 are used in this guide). However, these charts would only be applicable to soil and climatic conditions similar to those found at the AASHO Road Test. Each chart would have scales for total weighted load applications ($W_{t_{18}}$), for structural number ($\overline{SN}$), and for a single point on a scale for soil support representative of Road Test conditions.

To account for variations in climatic conditions from those found at the AASHO Road Test, it is assumed that the total load applications is an inverse function of the Regional Factor such that:

$$W_{t_{18}} = N_{t_{18}}\left(\frac{1}{R}\right)$$

or:

$$\log W_{t_{18}} = \log N_{t_{18}} + \log\left(\frac{1}{R}\right) \qquad \text{(C-8)}$$

where:

$N_{t_{18}}$ = total unweighted load applications.
R = Regional Factor.

To develop the design chart for soil support conditions other than those at the AASHO Road Test, it was necessary to establish a soil support scale. The starting point was a soil support of 3.0, which represented the supporting value of the roadbed soils at the Road Test. A second point was obtained by studying the performance of pavement structures on a known aggregate base which was sufficiently thick to minimize the effect of the roadbed soil. Examination of the performance of several sections on the loop carrying 18,000-lb (80kN) single-axle loads indicated that 4½ inches (114mm) of asphalt concrete surfacing on a substantial thickness of crushed-rock base would carry 1,000 18-kip (80kN) single-axle load applications per day (or 7,300,000 for a 20-year design period): By projecting a line from 1.98 on the SN scale (4.5 X .44) through 7,300,000 on the design traffic scale, a second point on the soil support scale was established. This point was given a value of 10.0, and it represented the soil support value of the crushed-rock base material used at the Road Test.

To establish intermediate points, a linear scale between points 3.0 and 10.0 is assumed such that

$$\log W'_{t_{18}} = \log N'_{t_{18}} + [f(S)] \qquad \text{(C-9)}$$

where:

$f(S) = K(S_i - S_o)$.
S_i = soil support value for any condition "i".
S_o = soil support value for Road Test conditions.
$N'_{t_{18}}$ = total load applications for Road Test condition.
$W_{t_{18}}$ = total load applications for condition "i".
K = a constant.

or:

$$10^{K(S_i - S_o)} = \frac{W_{t_{18}}}{N'_{t_{18}}} \qquad \text{(C-10)}$$

Therefore, for the following AASHO Road Test conditions:

SN	S	$W_{t_{18}}$ (daily)
1.98	10	1000
1.98	3	2.5

a solution of equation (C-10) yields:

$$10^{K(10-3)} = \frac{1000}{2.5}$$

$$K = 0.372$$

The influence of soil support is then defined by:

$$\underbrace{\log W_{t_{18}} = 0.372(S_i - 3.0) + \log N'_{t_{18}}} \qquad \text{(C-11)}$$

The reasonableness of this scale has been checked against a number of widely used design procedures and through theoretical computations.

By combining equations C-8 and C-11 with equations C-7, the equation whose solutions are presented in nomograph form in the design charts is developed as given below:

$$\boxed{\log W_{t_{18}} = 9.36 \log (SN + 1) - 0.20 + \frac{G_t}{0.40 + \dfrac{1094}{(SN+1)^{5.19}}} + \log\frac{1}{R} + 0.372(S_i - 3.0)} \qquad \text{(C-12)}$$

idem (E.4 Huang)

→ ábac. fig 2 pt=… pt=2

This equation represents the relationship between a weighted structural number (adjusted for varying soil support and regional factor) and total equivalent 18-kip (80kN) single-axle load applications for any selected terminal serviceability index.

A check on the assumption of linearity of the soil support scale has been made by application of layered plastic theory.(7) The response of a pavement to one dual wheel of an 18-kip (80kN) single-axle load, with assumed circular contact areas and tire spacing equal to one load radius, was used for this analysis. The variables considered in this analysis were:

1. Modulus of the surface layer, E_1 = 150,000 and 600,000 psi. (1 and 4.1GPa)
2. Modulus of the base layer, E_2 = 15,000 psi. (103MPa)
3. Modulus of the subgrade layer, E_3 = 3,000, 7,500, and 15,000 psi. (20.7, 51.7 and 103MPa)
4. Thickness of the surface layer, D_1 = 3, 4, 5, 6, 8, and 10 inches. (76, 102, 128, 153, 204 and 255mm)
5. Thickness of the base layer, D_2.

The surface and base modulus, and one level of subgrade modulus (E_3 = 3,000 psi) (20.7MPa), are similar to that established at the AASHO Road Test. The other values of subgrade modulus were selected primarily to represent a wide range of subgrades from poor to good, based on an assumed correlation with CBR values. The modular values of surfacing, base, or subgrade materials may also be determined by means of the laboratory test method for Modulus of Resilience (M_r) described in Report 128.(7) The values of soil support which correspond approximately to the values of subgrade modulus used are:

Modulus	Soil Support
3,000 (20.7MPa)	3.0
7,500 (51.7MPa)	5.1
15,000 (103MPa)	7.3

Base thicknesses used for each level of surface thickness were determinec from the relationship: $SN = a_1D_1 + a_2D_2$, where SN is the structural number and a_1 and a_2 are structural layer coefficients for the surface and base, respectively. For these conditions, the tensile strain in the bottom of the asphalt concrete and the vertical compressive strain on the subgrade were calculated using a computer solution of the layered elastic equation. The values for weighted load applications to a given level of serviceability were calculated using the following equation developed from AASHO Road Test results:

$$\log W_{18} = 9.36 \log (\overline{SN} + 1) - 0.20 + \frac{G_t}{0.40 + \dfrac{1094}{(\overline{SN} + 1)^{5.19}}}$$

For each structural number, the number of weighted 18-kip load applications (W_{18}) was calculated for terminal serviceability indices (p_t) of 2.5 and 2.0. Finally, on the basis of the above, relationships were developed between both the tensile strain in the bottom of the asphalt concrete and vertical compressive strain on the subgrade, and values of W_{18} for terminal serviceability indices of 2.5 and 2.0 for AASHO Road Test conditions.

C.2 — DETERMINATION OF TRAFFIC EQUIVALENCE FACTORS FOR FLEXIBLE PAVEMENTS

To use the flexible pavement design procedure presented in this guide, mixed traffic must be converted to an equivalent number of 18-kip single-axle loads. The procedure for accomplishing this conversion includes:

1. Derivation of Traffic Equivalence Load Factors.
2. Conversion of mixed traffic to equivalent 18-kip single axle load applications.
3. Lane distribution considerations.

To express varying axle loads in terms of a common denominator, it is necessary to develop traffic equivalence factors. These factors, when multiplied by the number of axle loads within a given weight category, give the number of 18-kip single-axle load applications which has an equivalent effect on the performance of the pavement structures. Analysis of the AASHO Road Test design equations permits the determination of such factors.

The design equation for flexible pavements developed in Section C.1 may also be written as:

$$\log W_t = 5.93 + 9.36 \log (\overline{SN} + 1) - 4.79 (L_1 + L_2) + 4.331 \log L_2 + G_t/\beta \quad \text{(C-13)}$$

All terms in this equation are as defined in Section A.1

If L_1 equals 18 kips, and L_2 equals 1 for single axles, equation (C-13) becomes:

$$\log W_{t_{18}} = 5.93 + 9.36 \log (\overline{SN} + 1) - 4.79 \log (18 + 1) + G_t/\beta_{18} \quad \text{(C-14)}$$

For any other axle load L_1, equal to X, equation (C-13) becomes:

$$\log W_{t_x} = 5.93 + 9.36 \log(\overline{SN} + 1) - 4.79 \log(L_x + L_2) + 4.33 \log L_2 + G_t/\beta_x \quad \text{(C-15)}$$

Subtracting equation (C-14) from equation (C-15) gives:

$$\log W_{t_x}/W_{t_{18}} = 4.79 \log(18 + 1) - 4.79 \log(L_x + L_2) + 4.33 \log L_2 + G_t/\beta_x - G_t/\beta_{18} \quad \text{(C-16)}$$

For single axles (L_2 = 1), equation (C-16) reduces to:

$$\log W_{t_x}/W_{t_{18}} = 4.79 \log(18 + 1) - 4.79 \log(L_x + 1) + G_t/\beta_x - G_t/\beta_{18} \quad \text{(C-17)}$$

or, for tandem axles, (L_2 = 2), to:

$$\log W_{t_x}/W_{t_{18}} = 4.79 \log(18 + 1) - 4.79 \log(L_x + 2) + 4.33 \log 2 + G_t/\beta_x - G_t/\beta_{18} \quad \text{(C-18)}$$

The ratio of W_{t_x} to $W_{t_{18}}$ gives the relationship between any axle load, X, single or tandem, and an 18-kip single-axle load. The ratio is defined as an equivalence factor, and is evaluated by solving equations (C-17) and (C-18) for any value X. Because the term β is a function of $\overline{SN}$ as well as L_x (Section C.1), the equivalence factor also varies with $\overline{SN}$. The computed equivalence factors for a wide range of axle loads (single and tandem) are summarized in Tables C.2-1 through C.2-4 for structural numbers from 1 to 6 and p_t values of 2.0 and 2.5.

The prediction of traffic for design purposes must rely on information from past traffic, modified by factors for growth or other expected changes. Most states accumulate past traffic information in the form of loadometer data of the Federal Highway Administration W4 loadometer tables, which are tabulations of number of axles observed within a series of axle load groups, with each group usually a 2,000-lb (8.9kN) interval. These tabulations are in a convenient

$lb = \frac{kg}{2,2045} = 0.4536\ kg$

1 Ton. = 9.8 kN

Table C.2-1

Traffic Equivalence Factors, Flexible Pavement

Single Axles, p_t = 2.0

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.002	0.003	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.03	0.04	0.04	0.03	0.03	0.03
10	44.5	0.08	0.08	0.09	0.08	0.08	0.08
12	53.4	0.16	0.18	0.19	0.18	0.17	0.17
14	62.3	0.32	0.34	0.35	0.35	0.34	0.33
16	71.2	0.59	0.60	0.61	0.61	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.59	1.56	1.55	1.57	1.60
22	97.9	2.49	2.44	2.35	2.31	2.35	2.41
24	106.8	3.71	3.62	3.43	3.33	3.40	3.51
26	115.7	5.36	5.21	4.88	4.68	4.77	4.96
28	124.6	7.54	7.31	6.78	6.42	6.52	6.83
30	133.4	10.38	10.03	9.24	8.65	8.73	9.17
32	142.3	14.00	13.51	12.37	11.46	11.48	12.07
34	151.2	18.55	17.87	16.30	14.97	14.87	15.63
36	160.1	24.20	23.30	21.16	19.28	19.02	19.93
38	169.0	31.14	29.95	27.12	24.55	24.03	25.10
40	177.9	39.57	38.02	34.34	30.92	30.04	31.25

Table C.2-2

Traffic Equivalence Factors, Flexible Pavement

Tandem Axles, p_t = 2.0

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.01	0.02	0.02	0.01	0.01	0.01
14	62.3	0.02	0.03	0.03	0.03	0.02	0.02
16	71.2	0.04	0.05	0.05	0.05	0.04	0.04
18	80.1	0.07	0.08	0.08	0.08	0.07	0.07
20	89.0	0.10	0.12	0.12	0.12	0.11	0.10
22	97.9	0.16	0.17	0.18	0.17	0.16	0.16
24	106.8	0.23	0.24	0.26	0.25	0.24	0.23
26	115.7	0.32	0.34	0.36	0.35	0.34	0.33
28	124.6	0.45	0.46	0.49	0.48	0.47	0.46
30	133.4	0.61	0.62	0.65	0.64	0.63	0.62
32	142.3	0.81	0.82	0.84	0.84	0.83	0.82
34	151.2	1.06	1.07	1.08	1.08	1.08	1.07
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.76	1.75	1.73	1.72	1.73	1.74
40	177.9	2.22	2.19	2.15	2.13	2.16	2.18
42	186.8	2.77	2.73	2.64	2.62	2.66	2.70
44	195.7	3.42	3.36	3.23	3.18	3.24	3.31
46	204.6	4.20	4.11	3.92	3.83	3.91	4.02
48	213.5	5.10	4.98	4.72	4.58	4.68	4.83

Table C.2-3

Traffic Equivalence Factors, Flexible Pavement

Single Axles, p_t = 2.5

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	17.8	0.003	0.004	0.004	0.003	0.003	0.002
6	26.7	0.01	0.02	0.02	0.01	0.01	0.01
8	35.6	0.03	0.05	0.05	0.04	0.03	0.03
10	44.5	0.08	0.10	0.12	0.10	0.09	0.08
12	53.4	0.17	0.20	0.23	0.21	0.19	0.18
14	62.3	0.33	0.36	0.40	0.39	0.36	0.34
16	71.2	0.59	0.61	0.65	0.65	0.62	0.61
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.61	1.57	1.49	1.47	1.51	1.55
22	97.9	2.48	2.38	2.17	2.09	2.18	2.30
24	106.8	3.69	3.49	3.09	2.89	3.03	3.27
26	115.7	5.33	4.99	4.31	3.91	4.09	4.48
28	124.6	7.49	6.98	5.90	5.21	5.39	5.98
30	133.4	10.31	9.55	7.94	6.83	6.97	7.79
32	142.3	13.90	12.82	10.52	8.85	8.88	9.95
34	151.2	18.41	16.94	13.74	11.34	11.18	12.51
36	160.1	24.02	22.04	17.73	14.38	13.93	15.50
38	169.0	30.90	28.30	22.61	18.06	17.20	18.98
40	177.9	39.26	35.89	28.51	22.50	21.08	23.04

Table C.2-4

Traffic Equivalence Factors, Flexible Pavement

Tandem Axles, p_t = 2.5

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.02	0.02	0.02	0.02	0.01	0.01
14	62.3	0.03	0.04	0.04	0.03	0.03	0.02
16	71.2	0.04	0.07	0.07	0.06	0.05	0.04
18	80.1	0.07	0.10	0.11	0.09	0.08	0.07
20	89.0	0.11	0.14	0.16	0.14	0.12	0.11
22	97.9	0.16	0.20	0.23	0.21	0.18	0.17
24	106.8	0.23	0.27	0.31	0.29	0.26	0.24
26	115.7	0.33	0.37	0.42	0.40	0.36	0.34
28	124.5	0.45	0.49	0.55	0.53	0.50	0.47
30	133.4	0.61	0.65	0.70	0.70	0.66	0.63
32	142.3	0.81	0.84	0.89	0.89	0.86	0.83
34	151.2	1.06	1.08	1.11	1.11	1.09	1.08
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.75	1.73	1.69	1.68	1.70	1.73
40	177.9	2.21	2.16	2.06	2.03	2.08	2.14
42	186.8	2.76	2.67	2.49	2.43	2.51	2.61
44	195.7	3.41	3.27	2.99	2.88	3.00	3.16
46	204.6	4.18	3.98	3.58	3.40	3.55	3.79
48	213.5	5.08	4.80	4.25	3.98	4.17	4.49

form for conversion, as the number of axles in each load group may be multiplied by the appropriate traffic equivalence factor to give $W_{t_{18}}$ for each load group. This can be accomplished as follows:

$$\begin{aligned} W_1 &= N_1 \cdot e_1 = N_t \cdot P_1 \cdot e_1 \\ W_2 &= N_2 \cdot e_2 = N_t \cdot P_2 \cdot e_2 \\ W_i &= N_i \cdot e_i = N_t \cdot P_i \cdot e_i \\ W_n &= N_n \cdot e_n = N_n \cdot P_n \cdot e_n \end{aligned} \qquad \text{(C-19)}$$

where:

W_i = equivalent 18-kip single-axle loads for load group i.
N_i = number of axles expected for load group i.
N_t = total number of axles.
P_i = percent of axles in load group i.
e_i = traffic equivalence factor for load group i.

The number of equivalent axle loads for all axle groups is then summed to give one number representative of mixed traffic:

$$W_{t_{18}} = W_1 + W_2 \ldots + W_i \ldots + W_n$$

$$\text{or } W_{t_{18}} = \sum_{i=1}^{n} W_i \qquad \text{(C-20)}$$

$$\text{or } W_{t_{18}} = N_t \sum_{i=1}^{n} P_i e_i$$

Equations (C-17) and (C-18) are used to compute the traffic equivalence factors, (e_i). These factors are, however, a function of SN. Therefore, to arrive at the design $W_{t_{18}}$ it is necessary to assume a value for SN, use the traffic equivalence factors listed for the assumed value of SN, and then solve equation (C-20). The use of an SN of 3.0 for the determination of 18-kip (80kN) single-axle equivalence factors will normally give results that are sufficiently accurate for design purposes even though the final SN determined is substantially different. This assumption will usually result in an overestimation of 18-kip (80kN) equivalent single axles, but the resulting error in SN is not significant. When most accurate results are desired and the computed SN is appreciably different from the assumed value, a new value should be assumed, the design traffic number ($W_{t_{18}}$) recomputed, and SN determined for the new $W_{t_{18}}$. This procedure should be continued until the assumed and computed values of SN are as close as desired.

The number of equivalent axle loads derived using equation C-20 represents the total for all lanes and both directions of travel. This number must be distributed by direction and by lanes for design purposes. Directional distribution is usually made by assigning 50 percent of the traffic to each direction, unless special considerations warrant some other distribution. In regard to lane distribution, most States assign 100 percent of the traffic in each direction to the design lane. Some States have developed lane distribution factors for multilane facilities. The range of factors used is given in Table C.2-5.

Table C.2-5

Lane Distribution Factors on Multilane Roads

Number of Lanes In Both Directions	Percent of $W_{t_{18}}$ In Design Lane
2	100
4	80-100
6*	60-80

If lane or directional distribution factors are utilized and pavements are designed on the basis of distributed traffic, consideration should be given to the use of variable cross sections. Heavier structural sections should be provided in the outside lanes when warranted on the basis of the lane distribution analysis.

Example of Determination of Equivalent 18-Kip Single Axle Loads from Traffic Volume and Loadometer Station Data.

Assume the following traffic volume data is representative for the design section. Calculation is for a 20-year design period.

Initial ADT = 4,000 vehicles per day
Projected 20-year ADT = 6,000 vehicles per day
Percent of trucks = 20 %
(based on 24-hour traffic volume)
Average ADT for a 20-year design period in both directions;

$$\frac{4{,}000 + 6{,}000}{2} = 5{,}000 \text{ vehicles per average day}$$

$$\text{Average 20-year ADT in one direction} = \frac{5{,}000}{2} = 2{,}500 \text{ vehicles}$$

The average number of trucks per day in one direction for the 20-year design period =

20% trucks X 2,500 vehicles = 500 trucks

The total number of trucks in one direction for the 20-year design period would be 500 trucks X 20 years X 365 days/year = 3,650,000 trucks.

For this traffic volume, all trucks in one direction would be assumed to travel in the design lane.

An 18-kip (80kN) axle equivalent rate per 1,000 trucks is shown in most State loadometer tables, by highway classification. An average rate is given for all trucks and a rate is given for each individual truck type. The figure for all trucks appears at the bottom of the table under the column titled "Total All Trucks and Combinations, Probable No.," and across from the row titled "18 Kip Axle Equivalents, Rate Per 1,000." The figure for each truck type appears in the same row under the column titled by each truck type.

For this example it is assumed that an 18-kip rate per 1,000 for all trucks is obtained from the W-4 loadometer table for other Main Rural (Non-Interstate) Highways.

18-kip (80kN) rate per 1,000 trucks = 605.23

The average number of trucks per day in one direction from the foregoing calculations = 500 trucks.

The average number of equivalent 18 kip (80kN) single axle load applications per day in the design lane = $\frac{500 \times 605.23}{1{,}000} = 302.615$

For design pruposes a figure of 300 can be used with the equivalent daily 18-kip (80kN) single axle load applications scale, Fig. II-1 or Fig. II-2.

The total number of trucks in the design lane for the 20-year period from the foregoing calculations = 3,650,000 trucks.

The total number of equivalent 18 kip single axle loads in the design lane for the 20-year period =

$$3{,}650{,}000 \times \frac{605.23}{1{,}000} = 2{,}209{,}000$$

For design purposes 2,200,000 can be used with the total equivalent 18-Kip (80kN) single axle load applications scale Fig. II-1.

The 18-kip (80kN) rate for each truck type can be used with the percent of each truck type determined from on-site traffic counts to develop a more accurate estimate of loading. If the distribution of trucks is much different from the statewide average this procedure might be appropriate.

Where loadometer information is available from a station that can be assumed to represent the traffic distribution for the design section, the axle weight distribution can be used directly. An example of the development of this data in the form used in the W-4 loadometer tables is given in Table C.2-6.

For example, assume the axle distribution shown in Table C.2-6 to be based on the axle weights obtained from 3,146 trucks. The 18-kip (80kN) rate per truck would be

$$\frac{1{,}826.8 \text{ 18 kip axle loads}}{3{,}146 \text{ trucks}} = 0.58067 \frac{\text{18 kip axle loads}}{\text{trucks}}$$

Using the same traffic volume that was used in the first example (Initial ADT = 4,000, 20-year ADT = 6,000, Percent Trucks = 20%), the number of trucks would be the same as calculated previously. The number was found to be:

Average number of trucks for 20 years in design lane = 500

Total number of trucks in design lane for 20 years = 3,650,000

The 18-kip equivalent single axle loading would be:

for average day in 20-year design

500 X 0.580 = 290 equivalent 18 Kip single axle loads per day

for total loads during design period

3,650,000 X 0.580 = 2,117,000 equivalent 18 Kip single axle loads

The foregoing example illustrates a simplified procedure for the calculation of 18-kip (80kN) equivalent single axle loads for design. This procedure assumes:

- The traffic volume increases uniformly from the beginning to the end of the design period
- The percent of trucks of the traffic volume is constant over the design period
- The axle weight distribution from which the 18-kip (80kN) rates were developed remains constant over the design period

If the facility justifies more exact calculations, projected changes in traffic volume and truck weights by vehicle type may be taken into account by calculating 18-kip (80kN) rates per truck type in the same manner as illustrated here for total trucks.

Table C.2-6

Example of Determination of Equivalent 18-kip (80kN) Single Axle Loads from Loadometer Station Data

Axle Load Groups, lbs	Representative Axle Load, lbs	Equiv. Factor[1]	No. of Axles[2]	Equiv. 18-kip Single Axles
Single Axles:				
Under 3,000	2,000	0.0003	512	0.2
3,000-6,999	5,000	0.012	536	6.4
7,000-7,999	7,500	0.0425	239	10.2
8,000-11,999	10,000	0.12	1,453	174.4
12,000-15,999	14,000	0.40	279	111.6
16,000-18,000	17,000	0.825	106	87.5
18,001-20,000	19,000	1.245	43	53.5
20,001-21,999	21,000	1.83	4	7.3
22,000-23,999	23,000	2.63	3	7.9
24,000 and over	–	–	0	–
			Subtotal	459.0
Tandem Axles:				
Under 6,000	4,000	0.01	9	–
6,000-11,999	9,000	0.008	337	2.7
12,000-17,999	15,000	0.055	396	21.8
18,000-23,999	21,000	0.195	457	89.1
24,000-29,999	27,000	0.485	815	395.3
30,000-32,000	31,000	0.795	342	271.9
32,001-33,999	33,000	1.00	243	243.0
34,000-35,999	35,000	1.245	173	215.4
36,000-37,999	37,000	1.535	71	109.0
38,000-39,999	39,000	1.875	9	16.9
40,000-41,999	41,000	2.275	0	–
42,000-43,999	43,000	2.74	1	2.7
44,000 and over	–	–	0	–
			Subtotal	1,367.8
			Total	1,826.8

Total, all trucks = 3,146

[1] For $p_t = 2.5$, and SN = 3.0

[2] Loadometer station data for 3,146 trucks

C.3 SOIL SUPPORT CORRELATION

The basic design equation developed from the results of the AASHO Road Test is valid for only one value of soil support, that representing the roadbed soils at the test site. In order to make the design procedure applicable to other roadbed soils, it was necessary to assume a soil support scale to represent the variety of soils which would be encountered at other sites. The development of the scale and its incorporation into the design charts is described in Section C.1.

The soil support value, however, has no defined relationship to existing test procedures; hence, before the design charts can be used, it is necessary for each user agency to establish the relationship between the soil support value and the method used to determine the support strength of the soil; e.g., CBR, R-Value, Group Index, Resilient Modulus (M_r). Several States have already developed such relationships in accord with the recommendations of the AASHO Committee on Design. An example of the method used by one State is described in the following.

Relationship Between CBR and Soil Support Developed by Utah State Department of Highways

Samples of the AASHO Road Test roadbed and crushed-stone base course materials were obtained for laboratory testing. The California Bearing Ratio (CBR), using dynamically compacted specimens, was determined for each material. The CBR for the roadbed material (soil support value = 3.0) was determined to be about 2.8, while that for the crushed-stone base course material (soil support value = 10.0) was about 200. A logarithmic scale between 2.8 and 200 was assumed(9) to obtain the correlation between CBR and soil support (Figure C.3.1). It should be noted, however, that this correlation is valid only for conditions in this State.

Four soil types were then tested using five separate laboratory test procedures(10) (three CBR dynamic and static compacted specimens and AASHTO T-193, and two R-Value procedures 240 psi (1.66MPa) and 300 psi (2.07MPa) exudation pressures). The results of these tests are summarized in Table C.3-1. Each value shown is the mean of 30 individual tests. The soil support value shown in the last column was determined from the correlation study (Figure C.3-1) described previously. These results were then plotted as shown in Figures C.3-2 through C.3-5. Each relationship assumed that:

1. The soil support scale is linear.
2. The scale for CBR (dynamic compacted specimens) as used by Utah, is logarithmic.
3. The relationship shown in Figure C.3-1 is valid.
4. The curves in Figures C.3-2 through C.3-5 pass through the origin.
5. There is a consistent relationship between test values and soil types for a particular test method. The relationship between all test procedures is given in Figure C.3-6.

Again, it is emphasized that the above is an example of one state's attempt to establish a relationship between the soil support scale for AASHO Road Test conditions and a test method used in the state to determine the supporting value of roadbed soils. The assumption for the relationship between CBR (dynamically

compacted) and soil support value was critical, and is valid only for the state in question. Other users should develop similar relationships, and verify them by observations of field performance.

Table C.3-1

Summary of Laboratory Test Results Repeatability Study, State of Utah

Soil Type	Soil Support	Dynamic CBR	Static CBR	AASHTO 3-Point CBR	R-Value (240psi)*	R-Value (300psi)*
A-7-6	3.9	4.9	7.2	1.9	8.4	12.0
A-4-5	4.9	8.9	8.0	5.2	10.5	14.8
A-2-4	7.2	38.9	42.6	9.9	68.2	72.2
A-1-9	8.4	78.0	116.5	17.2	75.5	77.2

* Exudation pressure

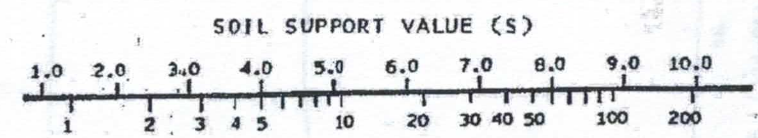


Figure C.3-1 Correlation Between Soil Support Value and CBR, Utah Department of Highways

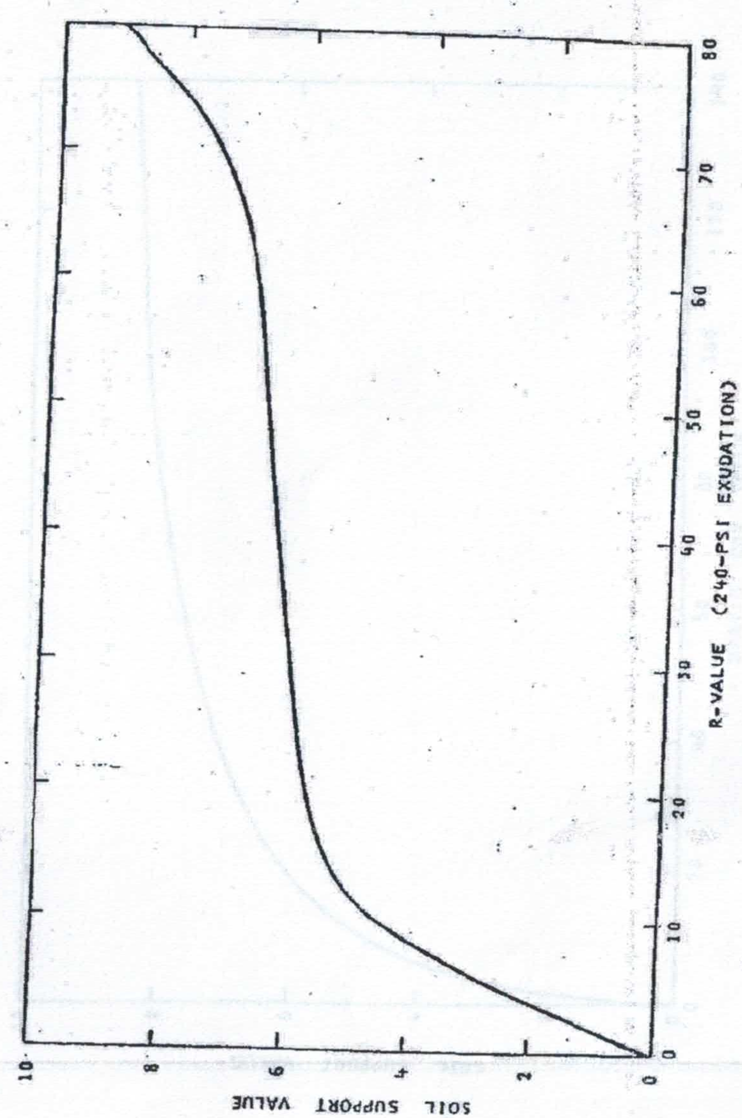


Figure C.3-2

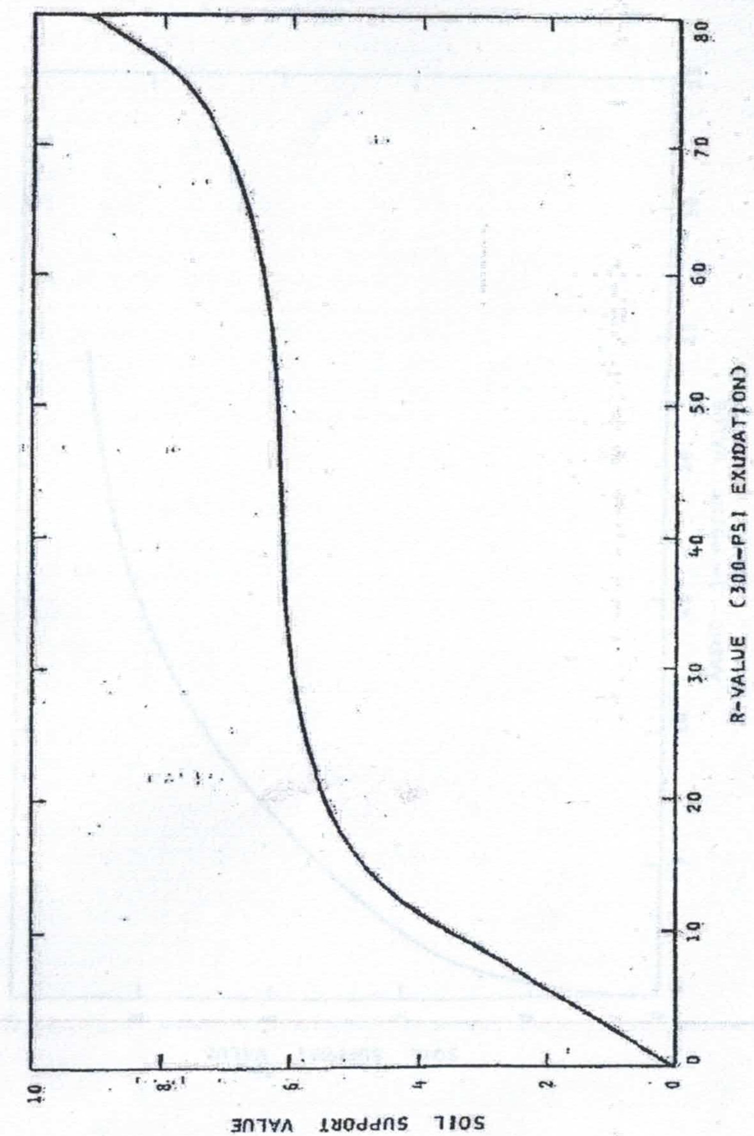


Figure C.3-3

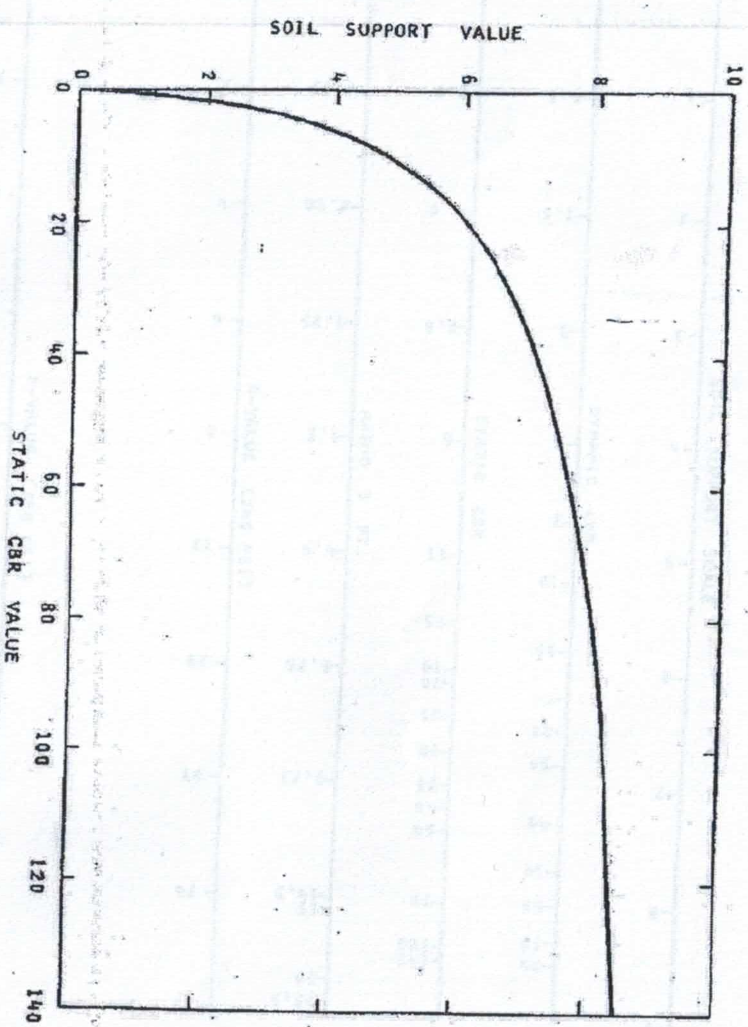


Figure C.3-4

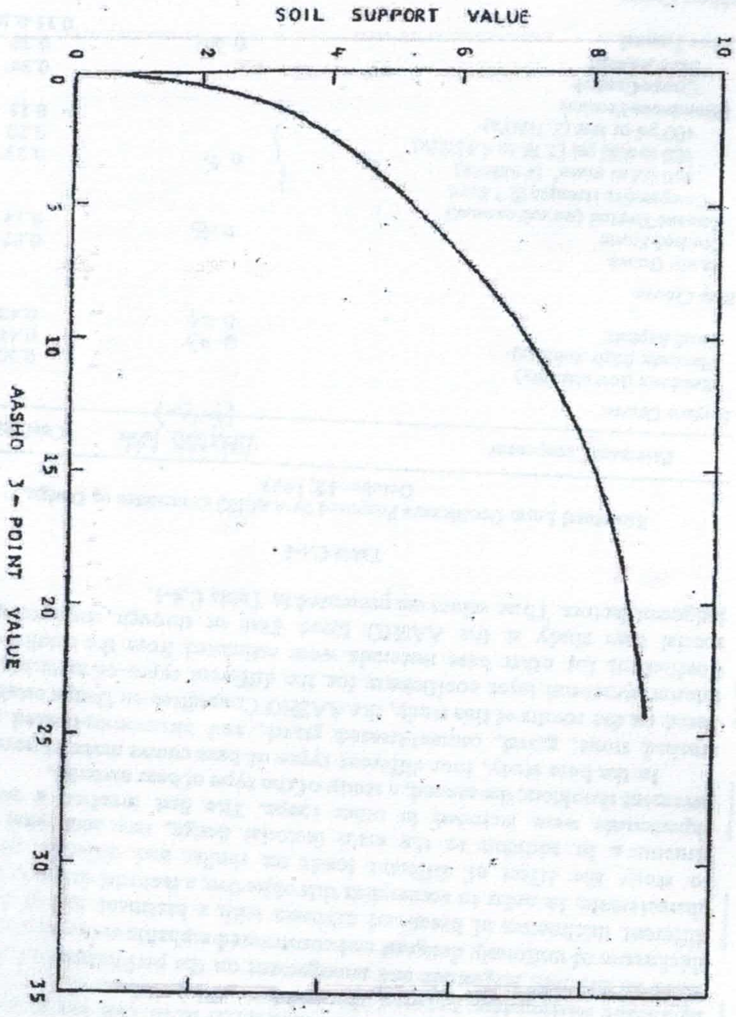


Figure C.3-5

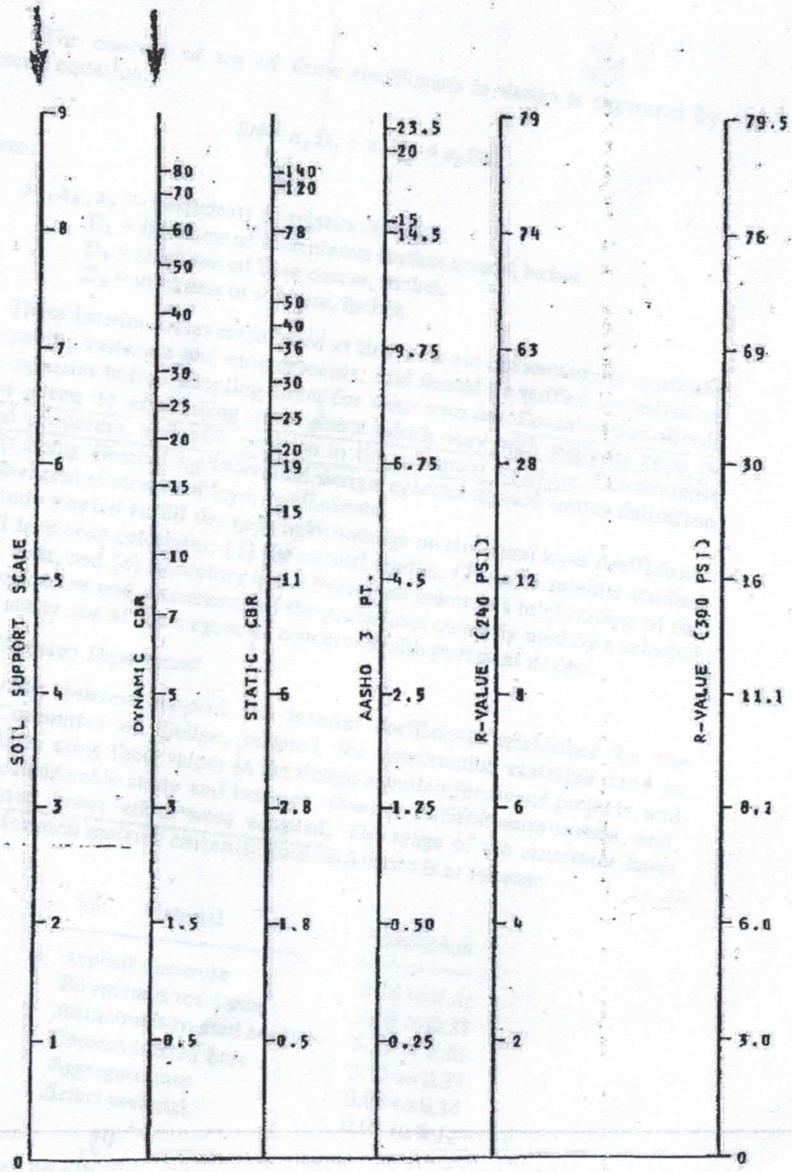


Figure C.3-6 Test Values as Determined from the Graphs in Figures C.3-2 through C.3-5

C.4 — DEVELOPMENT OF STRUCTURAL LAYER COEFFICIENTS

One of the major objectives of the AASHO Road Test was to determine significant relationships between the number of repetitions of specified axle loads of different magnitude and arrangement on the performance of different thicknesses of uniformly designed and constructed asphaltic concrete surfaces on different thicknesses of bases and subbases with a basement soil of a known characteristic. In order to accomplish this objective, a factorial design was set up to study the effect of different loads on similar and different pavement structures. In addition to the main factorial design, two additional special experiments were included in other loops. The first involved a study of pavement shoulders; the second, a study of the type of base material.

In the base study, four different types of base course material were used: crushed stone, gravel, cement-treated gravel, and bituminous-treated gravel. Based on the results of this study, the AASHO Committee on Design established interim structural layer coefficients for the different types of materials used. Coefficients for other base materials were estimated from the results of the special base study at the AASHO Road Test or through engineering and judgment factors. These values are presented in Table C.4-1.

Table C.4-1

Structural Layer Coefficients Proposed by AASHO Committee on Design, October 12, 1961.

Pavement Component	VIALIDAD ARG. (Folg.)	Coefficient[3]
Surface Course		
Roadmix (low stability)		0.20
Plantmix (high stability)	0.43	0.44*
Sand Asphalt	0.41	0.40
Base Course		
Sandy Gravel		0.07[2]
Crushed Stone	0.15	0.14*
Cement-Treated (no soil-cement)		
Compressive strength @ 7 days		
650 psi or more[1] (4.48MPa)	0.15	0.23[2]
400 to 650 psi (2.76 to 4.48MPa)		0.20
400 psi or less (2.76MPa)		0.15
Bituminous-Treated		
Coarse-Graded		0.34[2]
Sand Asphalt	0.20	0.30
Lime-Treated		0.15-0.30
Subbase Course		
Sandy Gravel	0.10	0.11*
Sand or Sandy-Clay	0.05 - 0.10	0.05-0.10

* Established from AASHO Road Test Data.

[1] Compressive strength at 7 days.

[2] This value has been estimated from AASHO Road Test data, but not to the accuracy of those factors marked with an asterisk.

[3] It is expected that each state will study these coefficients and make such changes as experience indicates necessary.

The concept of use of these coefficients in design is expressed by the general equation:

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

where:

a_1, a_2, a_3 = coefficients of relative strength.
D_1 = thickness of bituminous surface course, inches.
D_2 = thickness of base course, inches.
D_3 = thickness of subbase, inches.

These interim values established at that time are not necessarily applicable to all paving materials and environments, and should be verified by individual design agencies before adopting them for their own use. Consideration should also be given to establishing coefficients which vary with material type or material properties, and with position in the pavement structure. Considerable effort is being directed by individual design agencies toward better definition and refinement of structural layer coefficients.

Study needed to fill the gaps in knowledge on structural layer coefficients may fall into four categories: (1) theoretical studies, (2) major satellite studies, (3) field tests, and (4) laboratory tests. Presented below is a brief outline of the study approaches and a summary of the procedures currently used by a selected group of states and of other agencies concerned with pavement design.

Arizona Highway Department

Initially Arizona adopted the interim coefficients established by the AASHO Committee on Design, adapted for construction materials used in Arizona. After using these values in the design equation for several projects, and following considerable study and research, these coefficients were revised, and, in most cases, lower values were adopted. The range of the structural layer coefficient for each material currently used in Arizona is as follows:

Material	Coefficient
Asphalt Concrete	0.34 to 0.42
Bituminous road mix	0.25 to 0.37
Bituminous-treated base	0.30 to 0.35
Cement-treated base	0.15 to 0.29
Aggregate base	0.08 to 0.14
Select material	0.05 to 0.12

80

The method used to select the coefficient for a specific material is summarized in Table C.4-2 (page 317). To illustrate the use of this procedure, assume the case of a select material with the following properties:

1. Non-plastic.
2. 0 to 10 percent passing the 0.075mm (No. 200 sieve).
3. 75 percent passing the 2.36mm (No. 8 sieve).
4. Crushed material passing the 75mm (3 inch slot).

The coefficient for this material would be determined by adding the following to the base value of 0.05: 0.01 for non-plastic; 0.02 for percent passing the 0.075mm (No. 200) sieve; 0.01 for percent passing the 2.36mm (No. 8 sieve); and 0.01 for crushed to pass the 75mm (3-inch slot.) The resulting total of 0.10 is the layer coefficient which would be used for this select material in design. Coefficients for aggregate base, bituminous-treated base, and asphalt concrete are determined in a similar manner.

The Asphalt Institute

Based on special studies, it was concluded that the structural components (surfacing, base, and subbase) could be treated as linear combinations of equivalent thickness of each layer; i.e., $D = a_1 D_1 + a_2 D_2 + a_3 D_3$. A method for determining equivalency factors was based on a survey of prior performance, including the results of the AASHO Road Test and the WASHO Road Test, together with theoretical considerations. AASHO Road Test Special Report 61-E included a development of structural coefficients based on Performance Serviceability Index, Class 2 cracking, and deflection. On the basis of the analysis presented in this report, it was concluded that asphalt concrete can be from 2 to 6 times as effective as good crushed stone. Three separate multiple linear regression analyses were performed on the AASHO Road Test data, using the following three models:

$$SN = 2.0D_1 + D_2 + 0.75D_3$$
$$SN = 2.5D_1 + D_2 + 0.75D_3$$
$$SN = 3.0D_1 + D_2 + 0.75D_3$$

This analysis was made to determine which coefficient gave the least error. It was concluded that any of these transformations for SN would fit the data with approximately the same error, and, consequently, that one inch of high-quality asphalt concrete surfacing would be equivalent to 2 to 3 inches (50 to 75mm) of good dense-graded crushed-stone base, and that 1 inch of asphalt concrete base would be approximately equivalent to 2 inches (50mm) of such crushed stone.

A theoretical investigation using layered elastic thoery to interpret the AASHO Road Test data, suggested that, based on a comparison of vertical pressures on the subgrade, it appears reasonable to use an equivalency factor of at least 2 to 1. On the basis of these studies, The Asphalt Institute decided on an equivalency factor of at least 2 to 1 for asphalt concrete surface or base to aggregate base, and 2.67 to 1 for asphalt concrete surface or base to aggregate subbase.

$$\frac{4}{2.67} = 0.75$$

California Division of Highways

As a result of the performance of the thicker asphalt concrete sections on the AASHO Road Test, the cohesion value scale in the California design procedure was revised. The original California design equation was based on an assumed cohesion of 100 for gravel, and that no materials would be less than 100.

Tests performed on the AASHO Road Test gravel base material indicated a cohesion of only 20, and for the crushed-rock base, about 33. As a consequence,

Table C.4-2 Determination of Structural Layer Coefficients, Arizona Highway Department

SELECT MATERIAL

AGGREGATE BASE

ASPHALTIC PAVEMENT & BASES

CEMENT TREATED BASE

the basic design value for cohesionless materials was changed from 100 to 20, and a value of 30 was assigned to crushed-rock bases.

California's analysis of the special base study on the AASHO Road Test indicated that cement-treated bases had an equivalency of approximately 1.65 to 1. This agrees rather closely with the California factor of 1.75 to 1. From the analyses of bituminous base data, however, it was apparent that the magnitude of the load had a marked effect on the equivalency. To evaluate this, the property of cohesion was used to develop an empirical formula fitting the AASHTO equations:

$$C = \text{cohesion at } 72°F \times \left(\frac{8}{W+2}\right)^{2.5}$$

in which:

C = equivalent cohesion.
W = applied wheel load in kips (≥6).

Also for gravel equivalency (GE):

$$GE = \left(\frac{C}{\text{cohesion of gravel}}\right)^{0.2}$$

Using these equations, Figure C.4-1 was developed empirically to provide a means for adjusting equivalency for asphalt mixes that do not have tensile strength characteristics of the asphalt concrete at the AASHO Road Test, and a series of equivalencies based on predicted traffic was proposed. These proposed equivalencies for California material are summarized in Table C.4-3, and compared with equivalencies for AASHTO materials computed by the California method.

Table C.4-3

Proposed Equivalencies for Bituminous Material, California

(Thickness of Gravel Layer Required to Equal 1 inch of Asphalt Concrete)

Class of Road	TI Range	GRAVEL EQUIVALENCY – INCHES AASHTO Material	California Material
Heavy Industrial	12	2.0	1.6
	11	2.1	1.7
Heavy Truck Traffic	10	2.2	1.8
	9	2.3	1.9
Medium Truck Traffic	8	2.4	2.0
	7	2.6	2.1
Light Truck Traffic	6	2.8	2.3
Residential Streets	5	3.0	2.5
	4	3.0	2.5

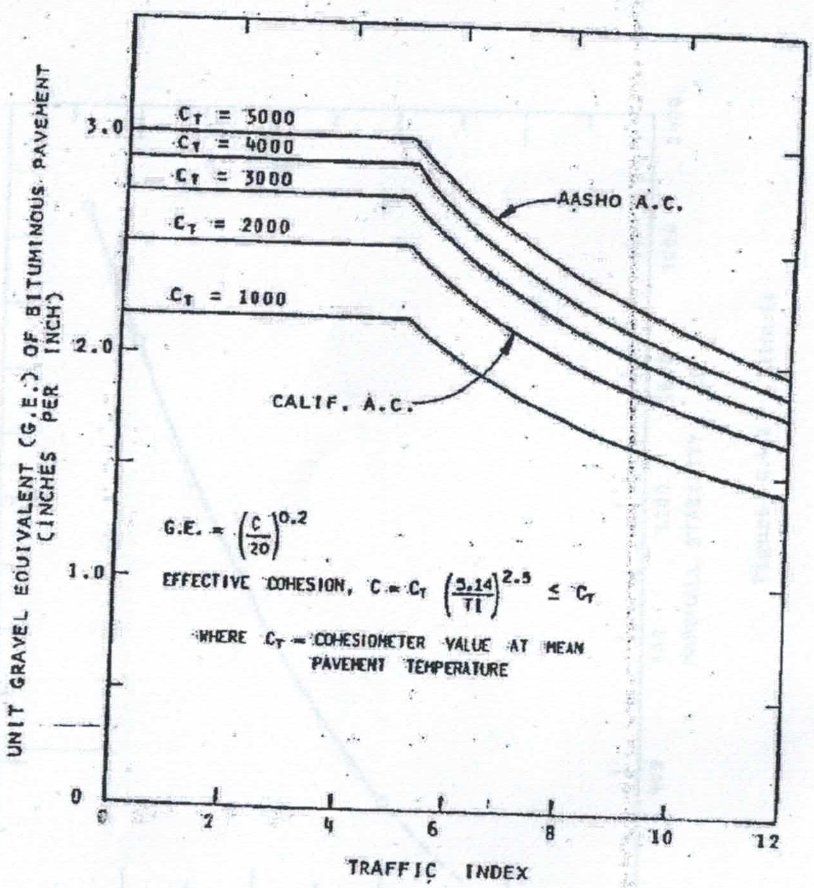


Figure C.4-1 Gravel Equivalent of Bituminous Pavement Based on AASHO Test Road Analysis - California

Illinois Division of Highways

Illinois has modified the interim coefficients established by the AASHO Committee on Design to account for differences in strength of pavement materials. These modifications are based on the premise that the value of a coefficient for a particular layer is not constant, but varies with the strength of the material used in that layer. Relationships between layer coefficient and material strength, as measured by Illinois test procedures, were established for surface, base, and subbase materials. Experience with the interim coefficients, together with results of tests conducted by the state, were used in establishing these relationships.

The coefficient for the surface course (a_1) was correlated with Marshall stability value (Figure C.4-2). The upper value of 0.44 represents the bituminous concrete used on the AASHO Road Test; the lowest point, the lowest stability road mix; the intermediate points, the Illinois Division of Highways bituminous subclass I-11.

A relationship between the coefficient for the base course (a_2) and materials strength was developed for four general categories of base: granular materials, bituminous-stabilized granular materials, portland cement-stabilized granular materials, and lime-stabilized granular materials. Figure C.4-3 shows the relationship developed between coefficients for granular base and CBR. The upper and lower limits represent the Road Test crushed-stone base material and the Road Test sand and gravel subbase material when used as a base course, respectively.

The coefficient for bituminous-stabilized granular base course materials was considered to vary with Marshall stability (Figure C.4-4). The upper point represents the bituminous-treated base on the AASHO Road Test; the intermediate point, grade 11 gravel stabilized with either emulsified or liquid asphalts; and the lower point, the AASHO Road Test sand and gravel material without treatment.

The coefficient for portland cement-stabilized granular base course materials was assumed to vary with seven-day compressive strength (Figure C.4-5). Here the upper point represents the AASHO Road Test cement-treated base; the lower point, the same sand and gravel material without cement stabilization; and the intermediate point, a material with the minimum compressive strength for adequate durability of a soil-cement base.

For lime-flyash-stabilized granular base course materials, it was assumed that the coefficient varies with the 21-day compressive strength (Figure C.4-6), since performance data for lime-treated base course materials were not available.

The coefficient for subbase material was correlated with CBR (Figure C.4-7). The upper and lower points represent the sand and gravel subbase material used on the AASHO Road Test and material representative of the AASHO Road Test sandy clay, respectively.

Louisiana Department of Highways

For the design of flexible pavements, Louisiana uses the coefficients for Louisiana materials summarized in Table C.4-4 (page 87). The coefficient for surface course is a function of Marshall stability, while the coefficients of the untreated and lime-treated base and subbase courses are a function of the Texas Triaxial Value. Cement-stabilized base is a function of compressive strength, while

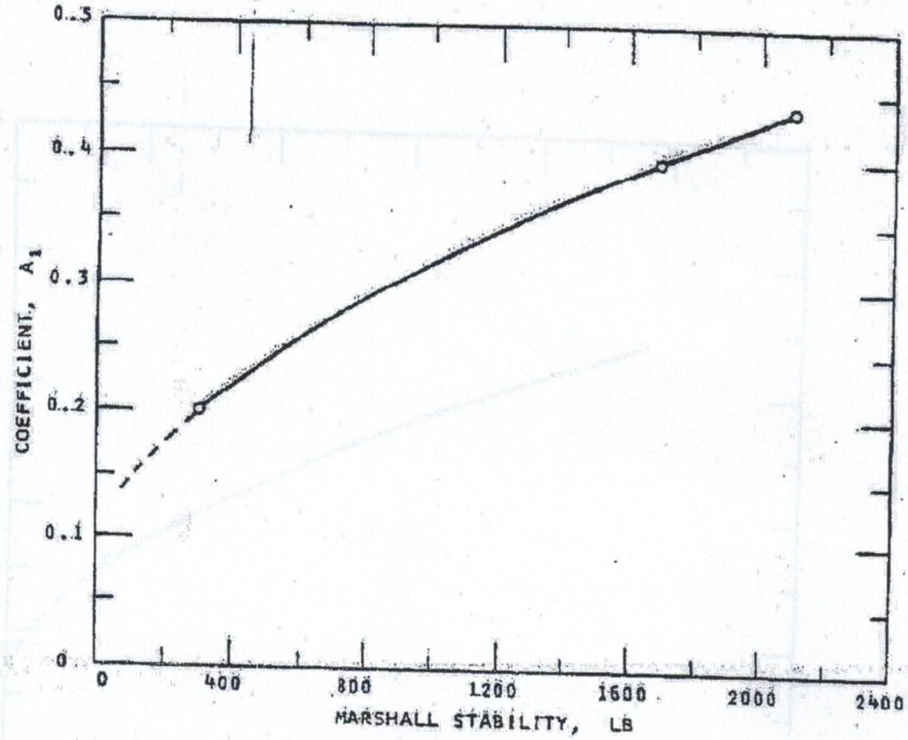


Figure C.4-2 Illinois

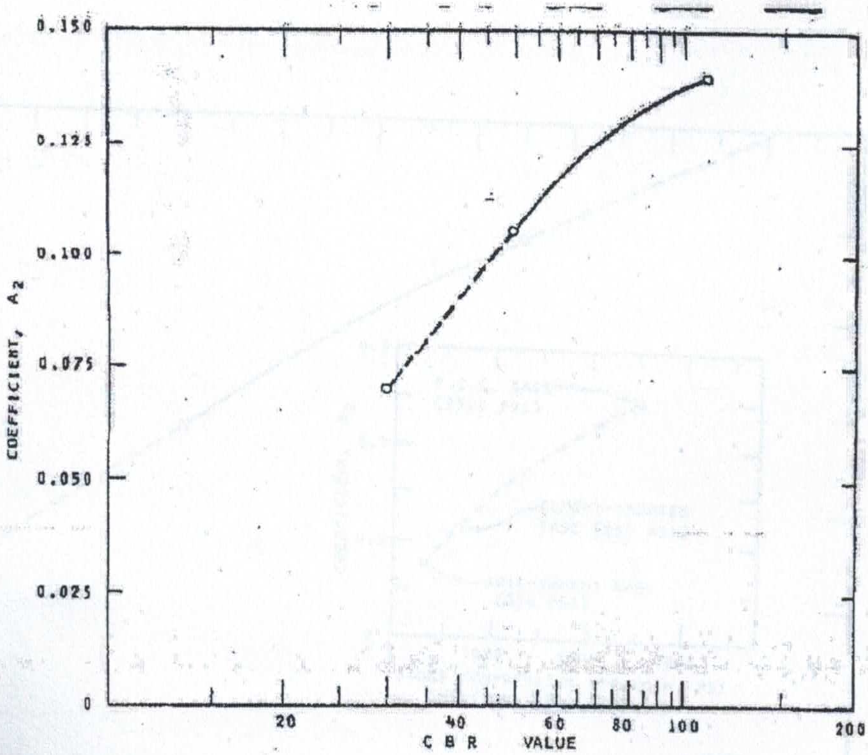


Figure C.4-3 Illinois

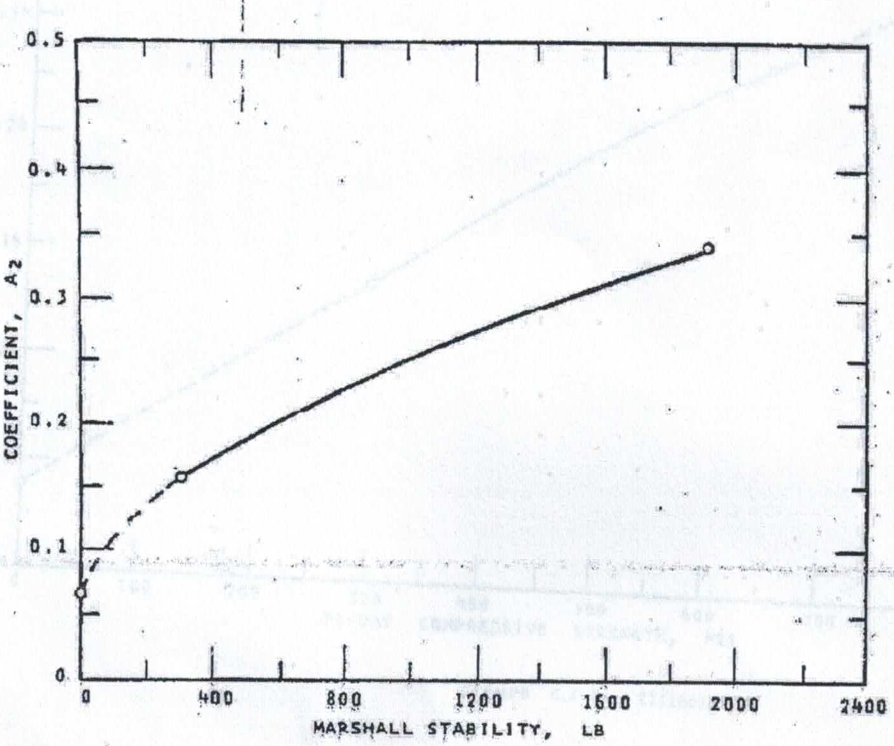


Figure C.4-4 Illinois

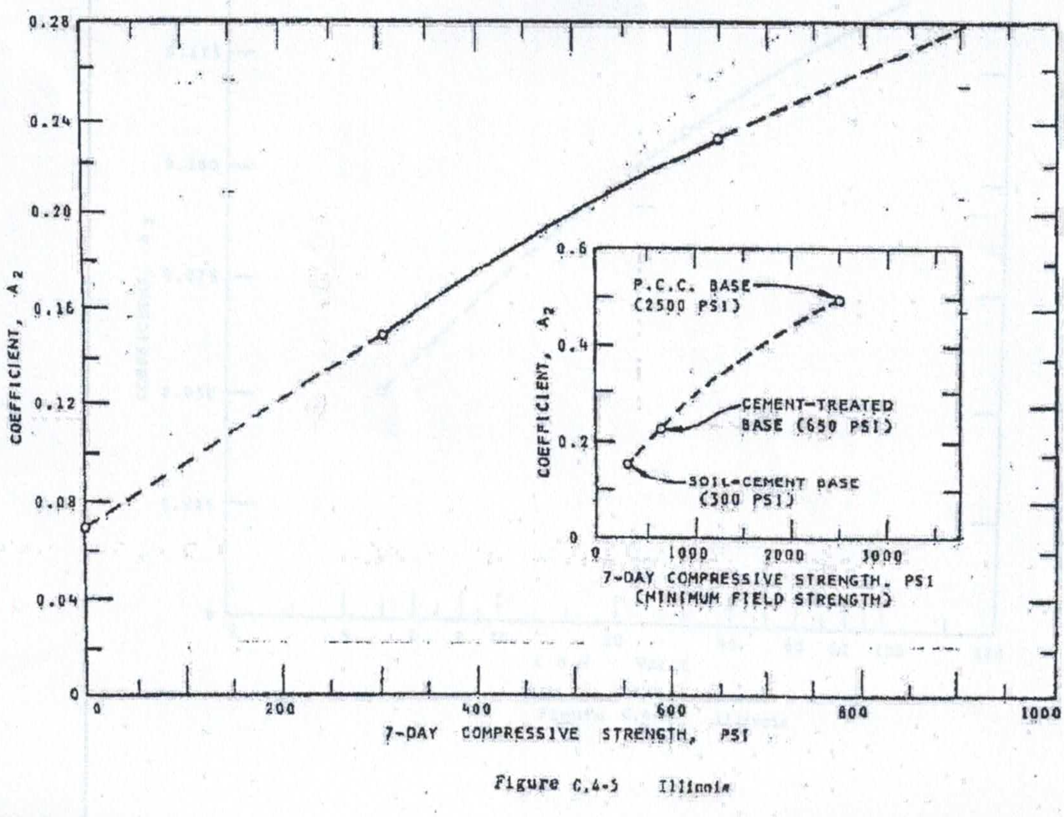


Figure C.4-5 Illinois

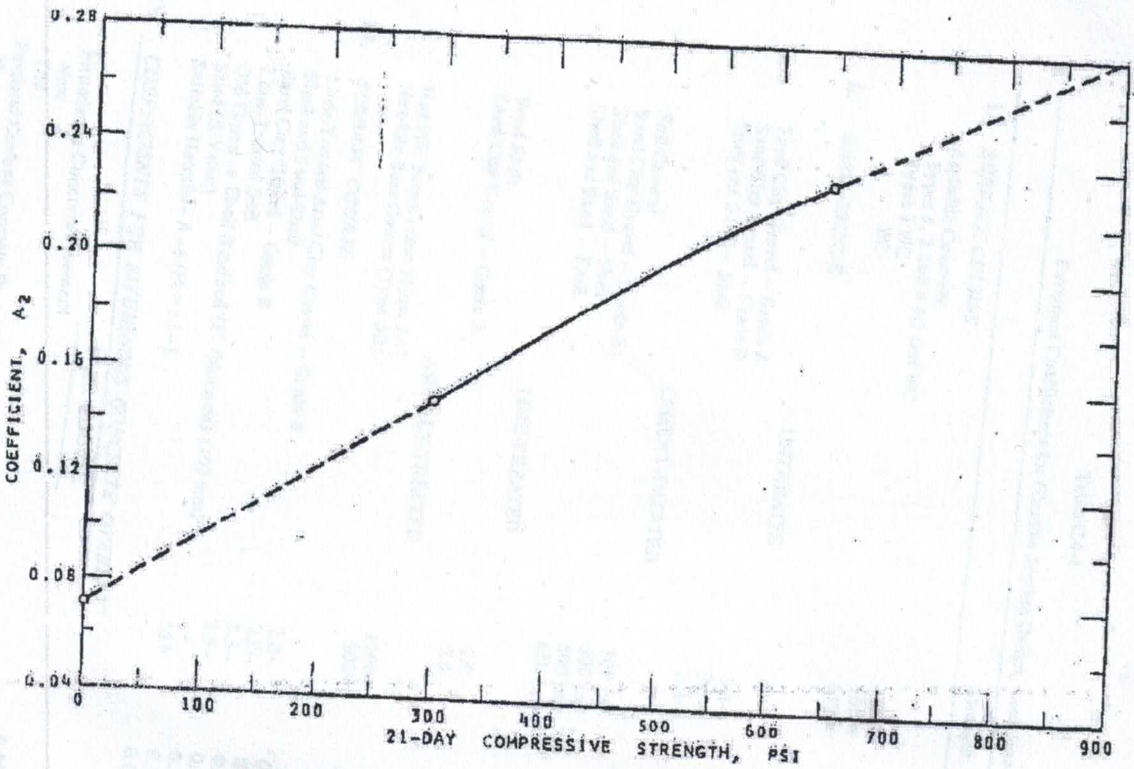


Figure C.4-6 Illinois

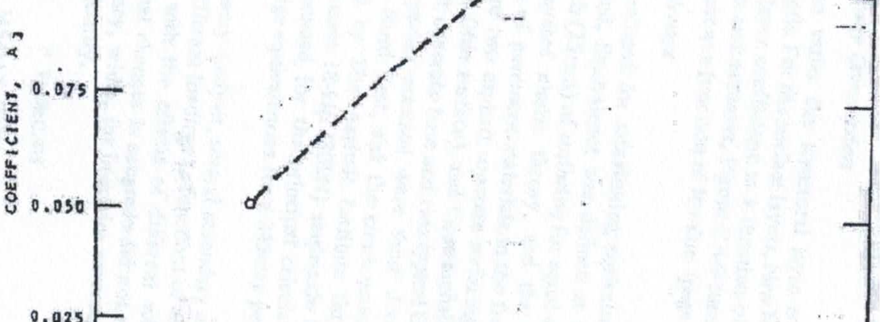


Figure C.4-7 Illinois

Table C.4-4

Pavement Coefficients for Flexible Section Design, Louisiana

	Strength	Coefficient
I. *SURFACE COURSE*		
Asphaltic Concrete		
Types 1, 2 and 4 BC and WC	1000+	0.40
Types 3 WC	1800+	0.44
BC	1500+	0.43
II. *BASE COURSE*		
UNTREATED		
Sand Clay Gravel – Grade A	3.3–	0.08
Sand Clay Gravel – Grade B	3.5–	0.07
Shell and Sand – Shell	2.2–	0.10
CEMENT-TREATED		
Soil-Cement	300 psi+	0.15
Sand Clay Gravel – Grade B	500 psi+	0.18
Shell and Sand – Shell	500 psi+	0.18
Shell and Sand – Shell	650 psi+	0.23
LIME-TREATED		
Sand Shell	2.0–	0.12
Sand Clay Gravel – Grade B	2.0–	0.12
ASPHALT-TREATED		
Hot-Mix Base Course (Type 5A)	1200+	0.34
Hot-Mix Base Course (Type 5B)	800+	0.30
III. *SUBBASE COURSE*		
Lime-Treated Sand Clay Gravel – Grade B	2.0–	0.14
Shell and Sand-Shell	2.0–	0.14
Sand Clay Gravel – Grade B	3.5–	0.11
Lime-Treated Soil	3.5–	0.11
Old Gravel or Shell Roadbed (8" thickness) (200 mm)	–	0.11
Sand (R-Value)	55+	0.11
Suitable Material–A–6 (PI = 15–)	–	0.04
IV. *COEFFICIENTS FOR BITUMINOUS CONCRETE OVERLAY*		
BASE COURSE		
Bituminous Concrete Pavement		
New		0.40
Old		0.24
Portland Cement Concrete Pavement		
New		0.50
Old, fair condition		0.40
Old, failed		0.20
Old, pumping		0.10
Old, pumping (to be underscaled)		0.35

the asphalt-stabilized base materials vary as a function of the Marshall stability. Note that coefficients used in the design of overlays for both bituminous concrete and portland cement concrete pavements are also given in the table.

New Mexico State Highway Commission

New Mexico also varies the structural layer coefficients of pavement components with strength. For the surface layers, New Mexico uses Figure C.4-8 to vary the structural layer coefficient as a function of Marshall stability. For untreated granular bases and subbases, Figure C.4-9 summarizes the variation in structural layer coefficient as a function of R-value (page 90).

Ohio Department of Highways

A method was developed for establishing equivalencies on a continuous basis for a 245-day period. Equivalence was defined as that thickness of base necessary to replace 1 inch (25mm) of surfacing for equal deflection. Deflections were calculated using layered elastic theory, and the results of static and dynamic laboratory tests of pavement materials in the frequency domain. The materials investigated were two asphalt concrete surfacings (the AASHO Road Test surface and a typical Ohio surface), and three asphalt concrete bases (the AASHO Road Test asphalt concrete base and two typical Ohio asphalt concrete bases). The subgrade properties assumed were those determined from 1960 trenchings at the AASHO Road Test, and the continuous hourly temperature data were those reported by The Asphalt Institute for the College Park, Maryland, location. Continuous 18-kip (80kN) single-axle loads, moving at 50 mph (80.45km/h) were assumed for the principal calculations. These calculations resulted in the average equivalencies for a 245-day period summarized in Table C.4-5.

In addition to the primary analysis, several secondary analyses were made to determine the effects of different loadings as functions of weight, speed, time, and contact area, together with the effects of different subgrades and layer thicknesses. It was shown that changes in subgrade did not make a significant change in the layer equivalency, while, for in-service pavements, vehicle speed can affect equivalencies significantly.

Table C.4-5

245-DAY DATA, OHIO

	AASHTO Surf. AASHTO Base	AASHTO Surf. B-21 Base	T-35 Surf. B-35 Base	T-35 Surf. AASHTO Base	T-35 Surf. B-21 Base
	Equivalence (in.)				
Average	1.24	1.28	0.78	0.83	0.90
High (4th hr)	1.53	1.51	1.03	1.05	1.06
Low (4th hr)	1.05	1.12	0.39	0.41	0.53
	Deflection (10^{-3} in.)				
Average	16.5	16.9	18.5	18.9	19.5
High (4th hr)	23.7	23.2	31.7	31.3	30.7
Low (4th hr)	10.9	11.6	12.5	12.7	13.6

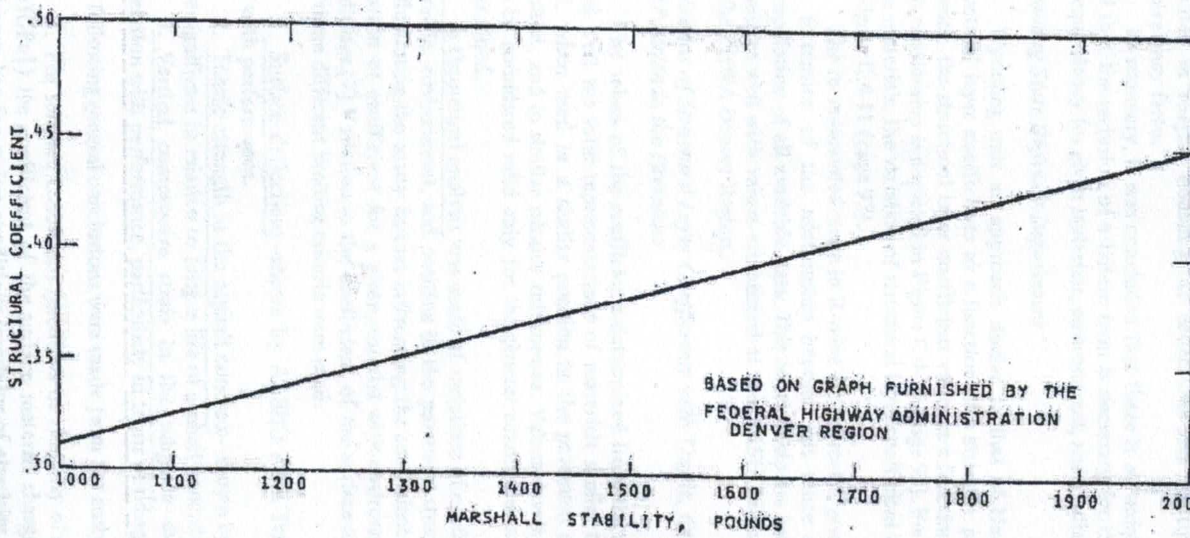


Figure C.4-8 Chart for Estimating Structural Coefficients of Bituminous Pavement Mixtures Based on Marshall Stability, New Mexico

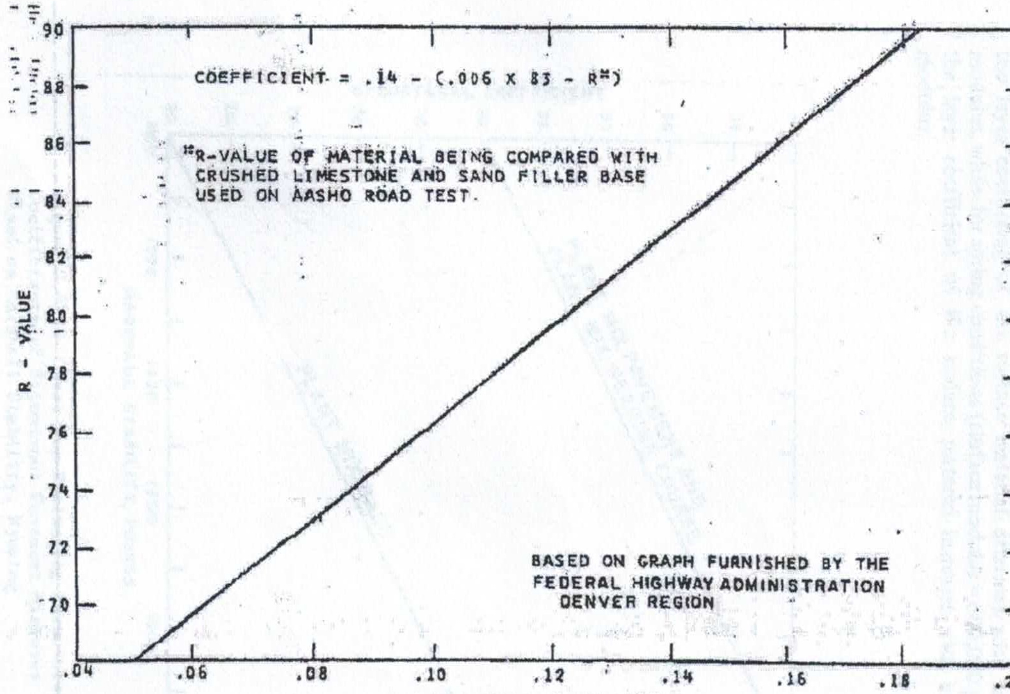


Figure C. 4-9 Chart for Estimating Structural Coefficients of Granular Subbase and Base Material Based on Stabilometer R-Value, New Mexico

It was also shown that the layer equivalency is a function of the thickness of the base layer. For this analysis, decreasing the base thickness by two-thirds resulted in roughly doubling the deflections and slightly decreasing the layer equivalency factor.

In summary, it was concluded that there is no unique equivalence factor, and that the inclusion of a failure term is necessary for theoretical calculations of equivalency for given materials, environment, and loading.

Wyoming State Highway Department

Wyoming uses an approach similar to that of New Mexico for varying structural layer coefficients as a function of a strength parameter. For surface courses, the structural layer coefficient varies as a function of Marshall stability. This relationship is depicted in Figure C.4-10 (page 92). For granular subbase and base materials, the variation of structural layer coefficient with R-value is shown in Figure C.4-11 (page 93).

The recommended range in R-value for untreated granular material is 65 to 80. Extension of the relationship beyond this range requires considerable extrapolation of all available data. This relationship has been used since 1968. It correlates well with values established at the AASHO Road Test and those used by the FHWA Denver Region.

Variations of Structural Layer Coefficients with Traffic, Environment, and Position in the Pavement

The values of the coefficients determined from the results of the AASHO Road Test are valid representations of materials similar to those of the Road Test, when used in a similar position in the pavement compacted to similar densities, and in similar relative thicknesses. Values developed by others should also be considered valid only for the general conditions under which they were determined.

A theoretical analysis was made of variations of coefficient with variations in traffic, environment, and position in the pavement structure, for the purpose of illustrating the many factors influencing the coefficient and, particularly, the variation of coefficient for a given material with environmental and geometric conditions.(7) Variation in the coefficient of the surface material was analyzed, and three different limiting criteria were used:

1. Surface deflections—shown by AASHO Road Test results to correlate well with performance.

2. Tensile strength in the asphalt concrete—shown by several investigators to be significant in relation to fatigue life of asphalt concrete pavements.

3. Vertical compressive strain in the subgrade—shown to have direct correlation with performance, particularly in terms of riding quality.

The following general conclusions were made from the analysis:

1. For summer conditions (modulus of elasticity of surface = 150,000 psi [1.03GPa]) the coefficient of the surface material changes little with surface thickness, but for spring conditions (modulus of elasticity of surface = 600,000 psi [4.14GPa]) the layer coefficient of the surface material increases appreciably as the surface thickness increases from 3 to 7 (76 to 178mm) inches.

2. The coefficient of the surface material is relatively constant for varying subgrade modulus (3,000 to 15,000 psi) (21 to 103MPa) except for an increase in surface modulus for thicker surfaces (6 inches [150mm]) with the lowest subgrade modulus.

3. For summer conditions (modulus of surface = 150,000 psi [1.03GPa]) the layer coefficient of the surface material decreases with increasing base modulus, while for spring conditions (surface modulus = 600,000 psi [4.3GPa]), the layer coefficient of the surface material increases with increasing base modulus.

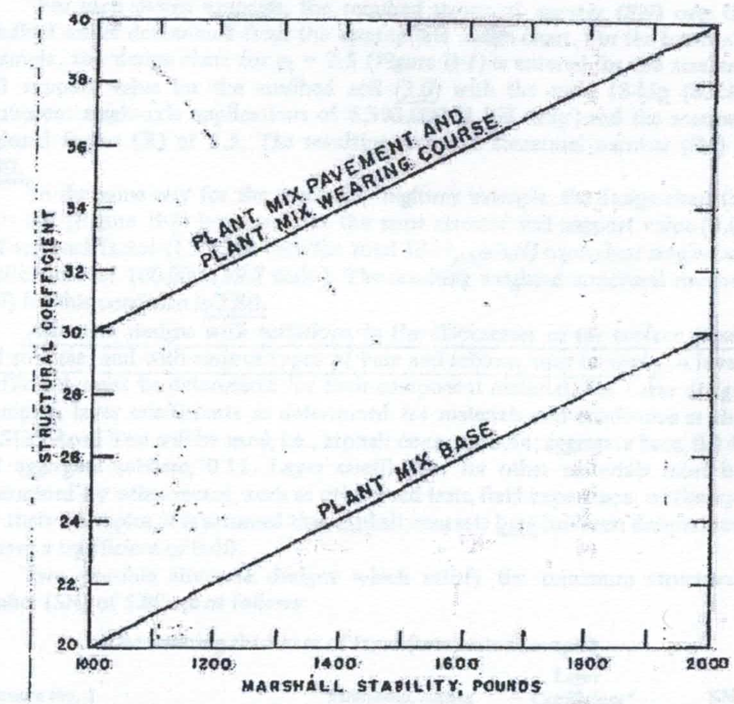


Figure C.4-10 Chart for Estimating Structural Coefficients of Bituminous Pavement Mixtures Based on Marshall Stability, Wyoming

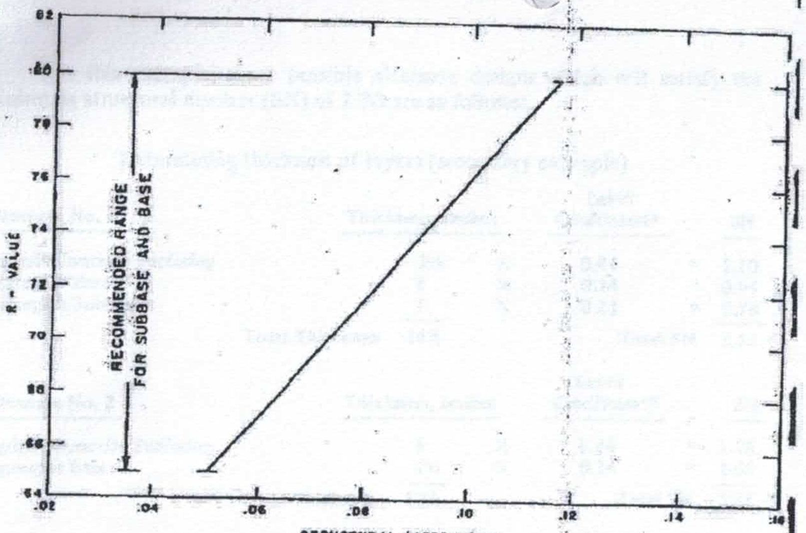


Figure C.4-11 Chart for Estimating Structural Coefficients of Granular Subbase and Base Material Based on Stabilometer R-Value, Wyoming

C.5 – DESIGN EXAMPLES, FLEXIBLE PAVEMENTS

This section presents examples of the use of the Guide for design of flexible pavement structures. Two design problems are included, one corresponding to traffic conditions of a typical interstate highway and the other for traffic conditions similar to that found on secondary highways. The interstate highway problem is also solved using an alternate design procedure. Other problems, such as swelling soils, frost conditions, stage construction, maintainability, economics, must also be considered, in addition to the factors included in the structural design analysis.

Selection of the terminal serviceability index is based on the lowest serviceability which will be tolerated before resurfacing or reconstruction. For the interstate highway an index of 2.5 is assumed, and 2.0 is assumed for the secondary highway example.

Total equivalent 18-kip (80kN) single-axle loads for the design lane is determined from the estimate of the type and amount of traffic expected over the design period and the appropriate traffic equivalence factors. The procedure used to reduce mixed traffic to a design traffic number is given in Section C.2. An example of the determination of total equivalent 18-kip (80kN) single-axle loads from loadometer station data, and of distribution to direction and design lane, is given in Table C.2-6. Since selection of the equivalence factors to be used to convert mixed traffic to total equivalent 18-kip (80kN) single-axle loads depends on the structural number (SN), a SN must be assumed for the initial conversion. The use of an average or assumed SN value for the determination of 18-kip (80kN) single-axle equivalence factors instead of the SN value determined by design normally will not result in significant design differences. If the SN resulting from solution by the design chart differs from the assumed value appreciably, a new value should be assumed, the number of 18-kip (80kN) single-axle loads recalculated, and the design operation repeated.

For these examples, it is assumed that the total 18-kip (80kN) single-axle loads expected in the design lane during the next 20 years will equal 8,500,000 (1,165 daily applications) for the interstate highway, and 100,000 (13.7 daily applications) for the secondary road.

Selection of a representative value of soil support for the roadbed soil is based on the results of a soil survey and laboratory tests. For both example problems it is assumed that the soil support value (S) for the roadbed soil is 3.0.

A regional factor of 1.5 is assumed to be representative of conditions at the sites for both examples.

For each design example, the required structural number (SN) over the roadbed soil is determined from the appropriate design chart. For the interstate example, the design chart for $p_t = 2.5$ (Figure II-1) is entered for the assumed soil support value for the roadbed soil (3.0) with the total 18-kip (80kN) equivalent single-axle applications of 8,500,000 (1,165 daily) and the assumed regional factor (R) of 1.5. The resulting weighted structural number (SN) is 5.80.

In the same way for the secondary highway example, the design chart for $p_t = 2.0$ (Figure II-2) is entered for the same assumed soil support value (3.0) and regional factor (1.5), but with the total 18-kip (80kN) equivalent single-axle applications of 100,000 (13.7 daily). The resulting weighted structural number (SN) for this condition is 2.80.

Alternate designs with variations in the thicknesses of the surface, base, and subbase, and with various types of base and subbase, may be made. A layer coefficient must be determined for each component material. For these design examples, layer coefficients as determined for materials and conditions at the AASHO Road Test will be used; i.e., asphalt concrete, 0.44; aggregate base, 0.14; and aggregate subbase, 0.11. Layer coefficients for other materials must be determined by other means, such as other road tests, field experience, or theory. For these examples, it is assumed that asphalt concrete base has been determined to have a coefficient of 0.40.

Two possible alternate designs which satisfy the minimum structural number (SN) of 5.80 are as follows:

Determining thickness of layer (interstate example)

Alternate No. 1	Thickness, Inches		Layer Coefficient*		SN
Asphalt Concrete Surfacing	6	x	0.44	=	2.64
Aggregate Base	12	x	0.14	=	1.68
Aggregate Subbase	13½	x	0.11	=	1.48
Total Thickness	31½			Total SN	5.80

Alternate No. 2	Thickness, Inches		Layer Coefficient*		SN
Asphalt Concrete Surfacing	4	x	0.44	=	1.76
Asphalt Concrete Base	8	x	0.40	=	3.20
Aggregate Subbase	8	x	0.11	=	0.88
Total Thickness	20			Total SN	5.84

For this example, three possible alternate designs which will satisfy the minimum structural number (SN) of 2.80 are as follows:

Determining thickness of layers (secondary example)

Alternate No. 1	Thickness, Inches		Layer Coefficient*		SN
Asphalt Concrete Surfacing	2½	×	0.44	=	1.10
Aggregate Base	6	×	0.14	=	0.84
Aggregate Subbase	8	×	0.11	=	0.88
Total Thickness	16½			Total SN	2.82 ✓

Alternate No. 2	Thickness, Inches		Layer Coefficient*		SN
Asphalt Concrete Surfacing	4	×	0.44	=	1.76
Aggregate Base	7½	×	0.14	=	1.05
Total Thickness	11½			Total SN	2.81 ✓

Alternate No. 3	Thickness, Inches		Layer Coefficient		SN
Asphalt Concrete Surfacing	2	×	0.44	=	0.88
Asphalt Concrete Base	5	×	0.40	=	2.00
Total Thickness	7			Total SN	2.88 ✓

*These values represent materials similar to these used on the AASHO Road Test when used in a similar position in the pavement and in about the same relative thicknesses.

C.6 — ALTERNATE PROCEDURE FOR DETERMINING THICKNESS OF LAYERS

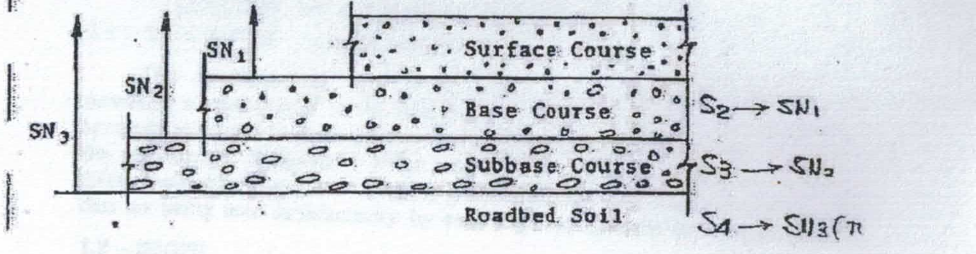
An alternative procedure may be used to determine the thickness for each of the component layers of the pavement. In this procedure representative values of soil support (S) are determined for the subbase and base materials. In the same manner that the soil support value for the loadbed is used to determine a total SN for the pavement, the design chart is used with the subbase S value to solve the minimum SN for the combination of base and surface course. This concept is presented graphically and algebraically in Figure C.5-1, where the following notation is used:

a_1, a_2, a_3 = layer coefficient for surface, base and subbase course materials, respectively.

D_1, D_2, D_3 = thickness of surface, base and subbase courses, respectively, in inches.

SN_1, SN_2, SN_3 = Structural Number for the surface, surface plus base, and total pavement, respectively.

If it is desired to consider the use of more than one type of material for any of the pavement courses, this procedure may also be used to prepare alternate designs.



$$D_1^* \geq \frac{\overline{SN}_1}{a_1}$$

$$\boxed{\overline{SN}_1^* = a_1 D_1^* \geq \overline{SN}_1}$$

$$D_2^* \geq \frac{\overline{SN}_2 - \overline{SN}_1^*}{a_2}$$

$$\boxed{SN_2 = \overline{SN}_1^* + \overline{SN}_2^* \geq \overline{SN}_2}$$

$$D_3^* \geq \frac{\overline{SN}_3 - (\overline{SN}_1^* + \overline{SN}_2^*)}{a_3}$$

$$SN_3 = SN_2 + a_3 D_3 \geq SN_3$$

WHERE:

1) a, D, and $\overline{SN}$ are as defined in the text, and are minimum required values.

2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

Figure C.5-1 Alternate Procedure for Determining Thicknesses of Layers

The information assumed for the previously described interstate highway xample is used to illustrate this alternate procedure. This information, together with assumed values of soil support for granular base nd for granular subbase, is summarized, as follows:

Serviceability Index (p_t)	2.5
Regional Factor (R)	1.5
Soil Support values (S)	
granular base	9.5
granular subbase	6.5
roadbed soil	3.0
Total Equivalent Axle Loads (1,165 daily applications)	8,500,000
Layer Coefficients	
Asphalt Concrete Surfacing (a_1)	0.44
Aggregate Base (a_2)	0.14
Aggregate Subbase (a_3)	0.11

For this design example, the required structural number over each of the component materials is determined from the design chart for $p_t = 2.5$ (Figure II-1). This chart is entered successively for the assumed soil support values for the roadbed material, granular subbase, and granular base (3.0, 6.5, and 9.5) with the total 18-kip (80kN) equivalent single-axle applications of 8,500,000 (1,165 daily) and the assumed regional factor of 1.5. The resulting SN's are 5.80, 3.80, and 2.55, respectively.

The minimum required thickness of each pavement component is determined by dividing the appropriate value of SN by the layer coefficient for that material. The SN representative of the minimum thickness of asphalt concrete surfacing is the SN required over aggregate base, or 2.55. Thus, the minimum thickness of asphalt concrete surfacing is 2.55 divided by 0.44 or 5.8 inches (147mm). The SN representative of the minimum thickness of aggregate base is the SN required over aggregate subbase (3.80), less the 2.55 provided by the asphalt concrete surfacing, or 1.25. The minimum thickness of aggregate base is then 1.25 divided by 0.14, or 8.9 inches (226mm). The SN representative of the aggregate subbase is the SN required over roadbed material (5.80), less the SN required over the aggregate subbase (3.80), or 2.00. The thickness of aggregate subbase is then 2.00 divided by 0.11, or 18.2 inches (462mm). Thus this alternative design consists of 5.8 inches (147mm) of asphalt concrete surfacing, 8.9 inches (226mm) of aggregate base, and 18.2 inches (462mm) of aggregate subbase.

In practice, thicknesses are usually expressed to the nearest ½ inch (13mm). In such "rounding off," the minimum thickness requirements should be maintained. In this case, only minor adjustments would be made to the surfacing and base, resulting in thickness of 6 and 9 inches (152 and 225mm), respectively. This "rounding up" of the surfacing and base thicknesses, allows the reduction of the subbase thickness to 17½ inches (445mm) while still satisfying the total SN of 5.80, as shown by the following check calculations:

	Thickness, Inches		Layer Coefficient		SN
Asphalt Concrete Surfacing	6	x	0.44	=	2.64
Aggregate Base	9	x	0.14	=	1.26
Aggregate Subbase	17½	x	0.11	=	1.92
Total Thickness	32½		Total SN		5.82

If desired, other changes in thickness of components can be readily made within the requirements for minimum thicknesses. For example, if the maximum thickness of aggregate base which may be compacted in one lift is 6 inches (152mm) the 9 inch (229mm) thickness in this alternate is inefficient from a construction standpoint. This base thickness may be increased to 12 inches (305mm) (2-6 inch [152mm] lifts) and the subbase reduced, with the allowable reduction based on layer coefficients and SN, as follows:

	Thickness		Layer Coefficient		SN
Asphalt Concrete Surfacing	6.0	x	0.44	=	2.64
Aggregate Base	12.0	x	0.14	=	1.68
Aggregate Subbase	13.5	x	0.11	=	1.48
Total Thickness	31½		Total SN		5.80

All minimum requirements for SN are also satisfied by this design variation.

SN₁ 2.5
SN₂ 3.80
5.80