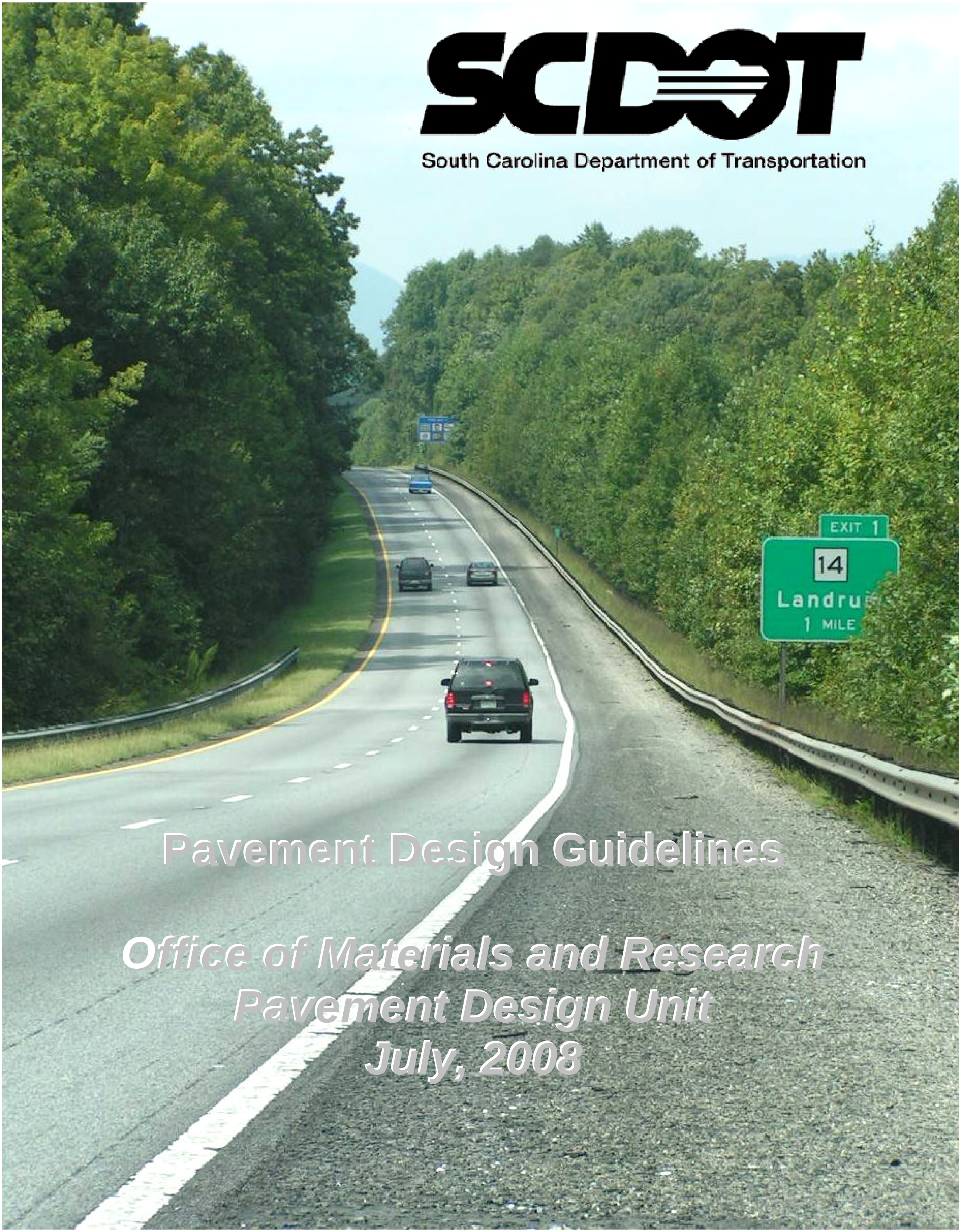
SCDOT

South Carolina Department of Transportation



Pavement Design Guidelines

Office of Materials and Research
Pavement Design Unit
July, 2008

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SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
OFFICE OF MATERIALS AND RESEARCH
PAVEMENT DESIGN GUIDE
June 2008

I. Introduction

When construction, maintenance, and rehabilitation costs are considered, the single most costly element of a highway system is the pavement structure. In an effort to reduce this cost, the Federal Government and all state highway agencies have sponsored a continuous program and research on pavements over the last 80 years. Although much has been learned, the state of knowledge on the behavior of paving materials remains incomplete. Variables associated with traffic loading, location, and environment make absolutely precise pavement performance predictions for new or rehabilitated sections very difficult.

The lack of perfect predictive capability, however, does not mean that pavement design activities are useless. On the contrary, current design procedures almost always provide pavements that perform at least as long as the target design period with a high degree of reliability. These predictions, combined with sound engineering judgment and knowledge of previous pavement performance in an area, is essential for most effectively allocating finite paving resources.

II. Goals of Pavement Design

The purpose of pavement design activity is to provide the most cost-effective pavement structure while optimizing the level of service provided to road users. These goals may frequently conflict. For instance, it may be most cost-effective to annually place a low-cost surfacing on a given segment of pavement. However, the service disruption required for yearly rehabilitation of the pavement would clearly be unacceptable for all but the lowest level traffic conditions. Conversely, a more initially expensive pavement design with a longer life, such as concrete or “perpetual” asphalt pavement, may ultimately be more cost effective, but funding constraints may also make that choice impractical. Cost alone can never wholly define the “best” pavement design.

III. New Pavement Design Procedures

The SCDOT uses the 1972 edition of *AASHTO Guidelines for Pavement Design* for new pavement designs, with some exceptions. Although AASHTO pavement design guidelines were extensively revised in 1986 and 1993, significant practical problems in the revisions have led the agency to continue with the earlier procedures. AASHTO has officially adopted a new mechanistic-empirical design guide that is completely different from past procedures. This procedure is based on mechanistic principles and uses advances in computer technology to more accurately simulate pavement behavior. However, this procedure will require extensive local calibration before it will reliably provide superior results when compared to current methods. It is expected that this calibration period will take five to seven years. This is consistent with the calibration of the current procedure, which was conducted from 1964 to 1972.

A. Pavement Design Procedures for New Flexible Pavements

Inputs to the design procedure for new flexible pavements are Soil Support Value (SSV), Equivalent 18,000 Pound Single Axle Loads in the Critical Lane (ESALs), Regional Factor (R), Terminal Serviceability (p_t), and Coefficients of Relative Strength for various paving materials (a_i).

1. Soil Support Value

Soil Support Value (SSV) is a term defining the relative subgrade support quality in relation to the soils at the AASHO Road Test. Values above 3.0 indicate that the soil has better support qualities than the A-6 soils at the Road Test site; less than 3.0 indicates poorer characteristics. The procedures typically used by the SCDOT to determine SSV are outlined in Appendix A.

A common source of confusion regarding SSV is that it is based on a single laboratory test or field observation. In fact, the final SSV assigned to a project is ultimately based on the subjective judgment of the potential variability of the soils encountered on a given project. For instance, in the case of a smaller project such as an intersection improvement constructed at-grade, the SSV assigned may be closely based on the results of laboratory testing since the soils encountered will likely be very similar to those tested. On the other hand, a large project with substantial borrow quantity that extends over many miles may receive a lower SSV than the smaller project even if the laboratory test results are similar. The lower value is given because the potential for variation is much greater for the larger project and must be accounted for in the pavement design.

2. <u>Traffic</u>

To use the AASHTO performance prediction equation, mixed vehicle types in the traffic stream must be converted into an equivalent number of 18-kip single axle loads. For trucks, some typical factors are shown in Table I. However, the actual ESAL value used for each type of truck varies depending on the traffic level. Because the effect of automobiles upon pavement is so slight, they are not considered in traffic analysis.

To simplify the task of dealing with mixed traffic further, Road Groups were developed based on the typical mix of traffic on different types of roads. The Traffic Engineering office within the SCDOT selects the appropriate Road Group for each project. The Road Groups are shown in Table II. For example, assume a four-lane road will average 10,000 vehicles per day in both directions over ten years. Of this traffic, 20% (or 2000) of the vehicles are trucks of various types corresponding to Road Group “O”.

The outside right lane is referred to as the “critical lane” in pavement design because it carries the greatest percentage of the traffic and, as a result, is more severely damaged and reaches failure first. It is the critical lane traffic that is used to design pavement. For roads with two lanes it is assumed that 80% of the traffic is using the critical lane. For roads with three lanes in each direction, it is assumed that 65% of the traffic is in the critical lane. For roads with four lanes in each direction, it is assumed that 60% of the traffic is the critical lane. So, for our example, it is assumed that 80% of the trucks (or 1600) are utilizing the outside lanes. It is further assumed that the traffic is equal in both directions, so the outside lane in each direction carries 800 trucks per day.

Eight hundred trucks per day of Road Group “O” is equivalent to (800 times 0.9027, see Table II) 722.2 ESALs per day, or 2,635,884 ESALs over 10 years. If the AASHTO flexible pavement design nomograph is used, this corresponds to an input value of 361.1 ESALs per day for 20 years.

Table I – Typical Equivalent Single Axle Loads for Various Truck Types (from Vehicle Classification Studies 1966-1970)

Vehicle Type	Equivalent Single Axle Loads on Flexible Pavement	Equivalent Single Axle Loads on Rigid Pavement
2 axles, dual tire axle on rear (Class 5)	0.1783	0.1775
3 axles, 2 dual tire axles on rear (Class 6)	0.6269	1.0152
2 axles on tractor, 1 dual tire axle on trailer (Class 8)	0.7691	0.7684
3 axles on tractor, 2 dual tire axles on trailer (Class 9)	1.0914	1.8532

Table II - Truck Type Distribution for Various Road Groups (Revised March 5, 1999)

ROAD GROUP	DISTRIBUTION OF TRUCKS BY TYPE (%)					ESALs PER TRUCK (FLEXIBLE)	ESALs PER TRUCK (RIGID)
	Class 5	Class 6	Class 8	Class 9	All others		
A	94	-	-	-	6	.1864	.1821
B	90	5	-	4	1	.2419	.2637
C	81	5	5	7	2	.2841	.3189
D	73	6	6	10	5	.3023	.3533
E	68	6	8	12	6	.3443	.4172
F	64	6	7	15	8	.3774	.4766
G	59	8	5	19	10	.4178	.5345
H	54	6	7	25	9	.4721	.6185
I	48	7	5	31	8	.5269	.6981
J	44	8	5	36	7	.5822	.7929
K	40	7	6	41	7	.6398	.8838
L	33	7	6	49	6	.7052	.9948
M	27	7	6	55	5	.7713	1.0971
N	24	3	6	60	7	.8346	1.2086
O	21	0	6	66	8	.9027	1.3214
P	12	3	4	72	9	.9891	1.5227

3. Regional Factor

The Regional Factor (R) is used to account for climatic variability throughout the country. The Regional Factor for the AASHO Road Test in Ottawa, Illinois was set at 1.0. A value of 1.0 is used for pavement design by the SCDOT.

4. Present Serviceability Index

The present serviceability index (PSI) concept was developed during the AASHO Road Test. A maximum PSI rating of 5.0 indicates an absolutely smooth, totally perfect pavement. A rating of 0.0 indicates total impassability at any speed in a wheeled vehicle. Intermediate values are based on levels of roughness and cracking which have been correlated to ratings given by road users. A flexible pavement begins with an initial serviceability of approximately 4.2 and is designed to reach a given serviceability level at failure, called terminal serviceability (p_t). The SCDOT uses a terminal serviceability of 2.5 for high-speed limited-access facilities and 2.0 for all other situations.

5. Coefficients of Relative Strength

Once all factors have been gathered, the equation used for prediction of the number of ESALs required to reach a condition corresponding to p_t for a given pavement is:

$$\log(ESALs) = 9.36\log(SN+1) - 0.20 + \frac{\log[(4.2 - p_t)/(4.2 - 1.5)]}{0.40 + [1094/(SN+1)^{5.19}]} + \log(\frac{1}{R}) + 0.372(SSV - 3.0) \quad \textbf{(1)}$$

where SN = Structural Number. Equation 1 is solved iteratively for an SN value that will provide an appropriate number of ESALs to failure. Equation 1 may also be solved for pt = 2.0 or 2.5 by using AASHTO nomographs shown in Appendix C in Figures 1 and 2.

Once the appropriate SN value is known, the value is converted to an actual pavement structure. Structural number is defined as:

$$SN = \sum_{i=1}^{n} a_i \times h_i \quad \textbf{(2)}$$

where a_i = coefficient of relative strength for the i^{th} layer and h_i = thickness of the i^{th} layer. The assigned coefficients of relative strength for South Carolina paving materials are shown in Figure 3.

The SCDOT has developed a number of different hot mix asphalt types to be used for different conditions. The recommended asphalt types are shown in Table III.

The AASHTO equations shown above cannot be used blindly. For instance, a pavement could be designed using 36 inches of Earth Type Base (often referred to as "Sand Clay" base) covered with a Bituminous Surfacing. This structure might satisfy the design requirement, but would not be suitable for a busy road, regardless of the structural number. Experience and judgment are required to create a combination of layers that will perform satisfactorily. Several "rule-of-thumb" limitations are typically used for flexible pavement design. These are:

1. Granular bases (Macadam, Marine Limestone, Recycled PCC, and Cement Stabilized Aggregate Base) are limited to a maximum thickness of 10 inches and a minimum of 6 inches.

2. Cement-Modified Subbase layers are typically 6 or 8 inches thick.

3. If combined AC Surface and AC Intermediate thickness exceeds 400 psy, the excess over 400 psy is assigned a coefficient of relative strength equal to 0.34.

When designing a pavement, the engineer should also be aware of other placement limitations related to the specific conditions at a given project. For instance, for a project with many tapers and turnouts, it may not be practical to use a design with Cement-Modified Subbase and Graded Aggregate Base. In a case such as this, it may be more practical to provide a full-depth asphalt design, preferably where the base is placed in a single lift.

S.C. Department of Highways and Public Transportation
Coefficients of Relative Strength for
Flexible Pavement Components

July, 2008

Pavement Component

Surface Course	a_1
HMA Surface	0.44[1]
HMA Intermediate	0.44[1]
Open Graded Friction Course	0.44[1]
Bituminous Surfacing	0.35

Old Surface	
Old Asphalt Concrete Surface	0.26
Old Asphalt Concrete Binder	0.26
Old Sand Asphalt	0.16
Bituminous Surfacing	0.21

Base	a_2
Sand-Clay Base	0.12-0.20[2]
Coquina Shell Base	0.12
Graded Aggregate Base	0.18
Cement Stabilized Earth Base	0.25
Asphalt Base, Type D	0.25[2]
Asphalt Base, Type A, B and C	0.34
Cement Modified Recycled Base	0.26
Cement Stabilized Aggregate Base	0.34
Old PCC Pavement	0.40

Subbase	a_3
Soil Aggregate Subbase	0.10
Cement-Modified Subbase	0.15[2]

Note 1. If the combined new HMA Surface and HMA Intermediate course rate exceeds 400 pounds per square yard, then coefficient of the excess material over 400 psy is reduced to 0.34. OGFC is not included in this total.

Note 2. Coefficient dependent on the quality of material available.

Figure 3 - Layer coefficients for South Carolina material

Table III – Guidelines for Hot Mix Asphalt Type Selection

<table>
<tr><th colspan="2"></th><th colspan="5">Type Facility</th></tr>
<tr><th colspan="2"></th><th>Interstate, Intersections, and Problem Areas</th><th>NHS, Primary, and High Volume Secondary (more than 10,000 vpd)</th><th>Primary and High Volume Secondary (10,000 vpd or less)</th><th>Low Volume Primary and High Volume Secondary (5,000 vpd or less)</th><th>Low Volume Secondary (1500 vpd or less)</th></tr>
<tr><td rowspan="4">Asphalt Type</td><td>Surface</td><td>Type A</td><td>Type B</td><td>Type CM</td><td>Type C</td><td>Type D</td></tr>
<tr><td>Intermediate</td><td>Type B
-or-
Type A
(problem areas only)</td><td>Type B
(min. rate 200 psy)</td><td colspan="3">Type C
(minimum rate 200 psy)</td></tr>
<tr><td>HMA Base</td><td colspan="2">Type A or C</td><td colspan="3">Type B, C, or D</td></tr>
<tr><td>Leveling and Build-up</td><td colspan="2">Surface Types B, CM, C, or E
Intermediate Type B
HMA Base Type A or C</td><td colspan="3">Surface Types C, D, or E
Intermediate Type C
HMA Base Type B or D</td></tr>
</table>

B. New Rigid Pavement Design

New rigid pavement design is based on a modified form of the 1998 AASHTO design methodology. Inputs to the rigid pavement design process include Modulus of Subgrade Reaction (k), Equivalent Single Axle Loads (ESALs), Change in Serviceability (ΔPSI), Modulus of Rupture of PCC ($S_c^{'}$), Modulus of Elasticity for PCC (E_e), Load Transfer Coefficient (J), Drainage Coefficient (C_d), Combined Standard Error of Traffic and Performance Prediction (S_0), and Standard Normal Deviate (Z_R). Based on these inputs, the thickness of the slab (D) is provided.

1. Modulus of Subgrade Reaction

Modulus of Subgrade Reaction is equivalent to Westergaard's modulus of subgrade reaction. It represents the load in pounds per square inch on a loaded area divided by the deflection in inches of that loaded area. This modulus has also been referred to as the "dense liquid" constant or the "spring" constant.

Direct determination of k is made by means of a plate-bearing test; a time-consuming and difficult procedure. Due to the difficulty involved, the test is rarely performed. Precise determination of k is made more difficult by the addition of a base course beneath the rigid pavement. The 1998 revisions to the AASHTO design methodology describe in detail the procedures to calculate the combined k of the base and subgrade. However, rigid pavement thickness is relatively insensitive to this parameter and an estimate based on average values is usually sufficient.

2. Traffic

Equivalent Single Axle Loads (ESALs) are defined in the previous section on flexible pavement design. However, different vehicles affect rigid pavement differently from flexible pavements. This results in ESAL factors for rigid pavements that are different and typically higher than those for flexible pavements. It is important to use the ESAL factor appropriate for the type of pavement being designed. See Tables I and II for ESAL factors for rigid pavement.

3. Change in Serviceability

Terminal serviceability (p_t) for rigid pavement is the same as for flexible pavements (2.5 for limited-access, 2.0 for all others). However, rigid pavements are traditionally assumed by AASHTO to have a slightly higher initial serviceability than flexible pavements (4.5 for rigid versus 4.2 for flexible). Unlike the 1972 equation for flexible pavements, the 1998 equation for rigid pavements asks for a change in serviceability rather than a terminal serviceability. So, the change in serviceability for rigid pavement is 2.0 for limited access and 2.5 for all other conditions and is determined by subtracting the terminal serviceability from the initial serviceability.

4. Moduli of Rupture and Elasticity of PCC

The design equation requires modulus of rupture at 28 days. Based on data collected from 28-day breaks of samples from construction projects, the actual 28-day modulus of rupture is assumed to be 600 psi. However, the design modulus of rupture is assumed to be 75 percent of the actual modulus. Consequently, a value of 450 psi is typically used for design.

Modulus of elasticity is not measured for paving concrete in South Carolina. It is assumed to be equal to 4,200,000 psi, which was the value used in the earlier versions of the AASHTO design equation.

5. Load Transfer Coefficient

The load transfer coefficient (J, also sometimes referred to as the edge protection factor) is used to adjust for the load transfer characteristics for a specific design. This term is related to the shoulder type and the use of dowel bars. Factors recommended for design are given in Table IV. If the outside lanes of a project are designed to keep the edge of the lane at least 2 feet from the edge of the pavement (i.e., a lane width of 14 feet or more, striped for 12 feet), then the coefficients for concrete shoulders may be used regardless of the shoulder type.

Table IV – Recommended load transfer coefficient (J) for various design conditions

Shoulder Type	Asphalt		Concrete	
Dowel Bars	Yes	No	Yes	No
	3.2	4.1	2.7	3.9

6. Drainage Coefficient

The 1998 design equation has an allowance for drainage conditions beneath the slab. The use of this coefficient is not recommended. Consequently, a value of 1.0 is used for all conditions. Although not explicitly included in the design, the importance of drainage beneath the slab cannot be understated. However, the use of a coefficient greater or less than 1.0 implies that adjustments in slab thickness can compensate for good or bad drainage conditions. Concerns over the validity of this concept have precluded its use by SCDOT.

7. Combined Standard Error of Traffic and Performance Prediction

The combined standard error of traffic and performance prediction (S_o) is intended to represent the total error for all aspects of the pavement design. However, SCDOT does not utilize the reliability coefficient in the design and prefers to use individually adjusted input factors. Consequently, the value of S_o is irrelevant because the term of the equation that uses S_o is zero.

8. Standard Normal Deviate

The standard normal deviate (Z_R) is based on the desired reliability for the design. However, SCDOT has not adopted the use of reliability as defined by AASHTO, so the value of Z_R is designated as zero. This corresponds to 50 percent reliability.

9. Slab Thickness

Slab thickness (D) is determined from the AASHTO performance equation. This equation is as follows:

$$\log(ESALs) = Z_R S_o + 7.35\log(D+1) - 0.06 + \frac{\log\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.624x10^7}{(D+1)^{8.46}}}$$

$$+ (4.22 - 0.32 p_t)\log\left[\frac{S_c^{'} C_d (D^{0.75} - 1.132)}{215.63 J\left(D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right)}\right] \qquad \textbf{(2)}$$

No closed-form solution for D exists for this equation. Consequently, this equation must be solved iteratively for different values of D until convergence is reached for the desired value of ESALs. Past experience has shown that rigid pavement thicknesses less than 9 inches are to be used with caution and 8 inches is the minimum allowable rigid pavement thickness. Graphical solutions to this equation are included in the 1993 AASHTO Design Guide, but computer-based solutions are recommended.

10. Other Considerations

Beyond slab thickness, consideration of other factors must be made in the design process. These considerations are:

- Rigid Pavement Type (JP, JR, CR)
- Joint Spacing and Load Transfer
- Base Type
- Subsurface Drainage
- Shoulder Design

The preferred rigid pavement type in South Carolina is a jointed, plain (JP) PCC. This type of pavement has provided outstanding performance, with pavements lasting over 30 years under higher than design traffic loadings without rehabilitation in many cases. Continuously reinforced PCC was used extensively in South Carolina in the 1970's. However, the performance of this type of pavement has been lower than for jointed plain PCC pavement. Jointed reinforced pavement has been used extensively in the northeast and midwest United

States, but has been largely supplanted by the other types of PCC pavement for new construction.

Joint spacing for rigid pavement in South Carolina has ranged from 15 to 30 feet. During the 1960's and 1970's, 25-foot joint spacing was typical. In the 1970's and 1980's SCDOT constructed pavements with a pseudo-random joint spacing of approximately 20 feet. Some pavements in the 1970's (principally portions of I-95 and I-20) were also constructed with a 2:12 skew to reduce the dynamic effect of traffic on the joints. All jointed PCC pavement built in South Carolina prior to 1976 depended on aggregate interlock to provide load transfer.

The purpose of the random joint spacing was to avoid harmonic excitation of automotive suspensions (also known as "freeway hop" or "pogoing"). Some 1950's automobiles with extremely soft suspensions would exhibit severe harmonic excitation when driving on faulted concrete pavement with 15-foot joint spacing. Consequently, research by General Motors indicated that joint spacing divisible by 7.5 feet was to be avoided. However, improvements in automotive design have largely eliminated this problem in modern cars, rendering random joint spacing and avoidance of multiples of 7.5 feet unnecessary.

Research conducted on SHRP data and other sources have indicated a strong correlation between increased pavement life and shorter joint spacing. This relationship is not reflected in the 1998 AASHTO design equation, but will be included in future design equations. Until a better relationship is developed, it is recommended that a uniform 15-foot joint spacing be used on all new concrete pavements. Any spacing used should be evenly divisible by 2.5 feet to ensure compatibility with the tie bar spacing.

Aggregate interlock was the method used in South Carolina until the mid-1970's to provide load transfer across pavement joints. While working well initially, this type of joint loses its interlock with time, causing the pavement to fault and develop unacceptable ride quality. Due to this problem, positive load transfer through dowelled joints is recommended for all new rigid pavements. The diameter of the dowel bar is typically 1¼ inch on pavement ten inches thick or less and 1½ inch on pavements up to twelve inches. For pavements greater than twelve inches, consult the SCDOT Pavement Design Engineer for specific recommendations.

Pumping of base material leading to faulting of concrete pavement joints has been a major distress for South Carolina's rigid pavements. In addition to positive load transfer, adequate design is required to retard these problems and improve performance. Even with highly effective maintenance, any jointed rigid pavement will be subject to intrusion of surface water. To extend pavement life, positive drainage may be incorporated into all rigid pavement designs. This will typically consist of a base material specifically engineered for mechanical stability and high hydraulic conductivity. The permeable material must be connected to a series of drains with pipe-type outfalls. Daylighting of base material has not performed well previously and is not considered acceptable.

Although it is a generally accepted principle that improved drainage will extend the life of a concrete pavement, the degree of actual life extension provided by a positive drainage system is not well defined. Additionally, it is questionable in the very long term whether the drainage system will continue to perform properly or will eventually clog and become a ready sump to hold water. It is unknown whether maintenance forces will be consistently available in the future to maintain the outlets and ensure that the drains flow freely. More recent research indicates that pumping at pavement joints is considerably reduced by the use of shorter slab spacing, positive load transfer, and erosion-resistant base materials. Consequently, each project needs to be evaluated individually to determine whether a drainage system is desirable.

A key factor in the performance of concrete pavements is the selection of base material type. Unlike asphalt pavements, concrete pavement thickness is relatively insensitive to the overall stiffness of the base and subgrade. Prior to WWII, when concrete was the standard paving material for primary pavements, the pavement was frequently placed directly on compacted subgrade without a base course. Given the low traffic and axle loads of the time, pavement constructed in this manner performed quite well and many of these pavements continue to perform under a thin lift of asphalt after 70 or more years.

Although the pavement thickness is relatively insensitive to the base stiffness, the performance of concrete pavement is still strongly affected by the base selection. To perform well, the base material must be insensitive to moisture and resistant to erosion. Additionally, while the total stiffness of the base is not critical, the uniformity of support is very important. If the base has areas that are less stiff than the surrounding material, undesirable stress concentrations are created in the concrete slab. These stresses will lead to early failure of the slab and cracking. Bases that are too stiff may also lead to undesirable concentration of stress when the concrete slabs warp and curl due to temperature differentials between the top and bottom of the slab.

Consequently, concrete (including Roller Compacted and Lean) and cement stabilized aggregate bases are not recommended due to their high stiffness. Earth Type (Sand Clay) Base, Marine Limestone Base, Coquina, Sand Asphalt, and Cement Stabilized Earth Base are not recommended directly beneath the slab due to their erosion characteristics. Macadam Base made entirely from crushed granitic aggregate and Asphalt Aggregate Base are acceptable base materials for lower volume concrete pavements. For interstates and other high volume routes, a course of Asphalt Surface – T1 is strongly recommended directly beneath the slab. However, any of the bases types listed above may be used if they are capped by a course of asphalt surface.

C. Shoulder Design Guidelines for New Pavement

In general, it is recommended that the shoulder pavement type be the same as the mainline type. For new flexible pavements, shoulders may be designed using the same structural techniques as for the mainline pavement. The design inputs for flexible shoulders are the same as for flexible mainline pavement, except for the traffic level. For shoulders, the traffic is some fraction of the mainline traffic and represents the percentage of encroaching and parked traffic using the shoulder. Studies conducted in other states indicate that one to eight percent of trucks will encroach upon the shoulder at any given point. An additional one-half percent of trucks will use the shoulder for parking. Consequently, it is recommended that a flexible shoulder be structurally designed to withstand three to five percent of the mainline traffic. If the shoulder is to be used as a traffic lane in the future, this should also be considered in the design process.

Shoulders for new rigid pavements should be tied to the mainline pavement using tie bars. The shoulder may be constructed out of a leaner concrete mixture than the mainline pavement, but should be the same depth. Where applicable, the structural benefits of tied shoulders may be achieved by widening the lane two feet, them marking the pavement to discourage traffic from using the additional width. This is only recommended in areas where the pavement will never be widened to include another lane, because the traffic in the future lane would be operating on top of the old lane/shoulder joint. If a fourteen-foot outer lane is used, the remaining shoulder thickness may be constructed using asphalt designed for two to three percent of the mainline traffic.

D. Selection of Pavement Type

The selection of pavement type is not an exact, objective process, but one in which the pavement designer must make judgments on many varying factors. The pavement type selection may be dictated by an overriding consideration for one or more of these factors. The predominant factors in the selection process are given below.

The selection process may be facilitated by comparison of alternate structural designs for one or more pavement types using theoretical or empirically derived methods. However, such methods are not so precise as to absolutely guarantee a certain level of performance from any one alternate or comparable service for all alternates.

Comparative cost estimates can be applied to alternate pavement designs to aid in the decision-making process. The cost for the service of the pavement would include not only the initial cost but also subsequent costs to maintain the service level desired. It should be noted that these procedures are also imprecise due to the lack of information on costs attributable to future events such as maintenance, salvage value, and the value of reduced service to the road user.

Even if structural design and cost comparison procedures were perfected, by their nature they would not encompass all factors that should be considered in pavement type selection. Such a selection should properly be one of professional engineering judgment based on the consideration and evaluation of all factors applicable to a given highway section.

Beyond economic analysis, a variety of factors affect the pavement type selection process. Some of these factors are:

- *Construction Considerations*: Staged construction of the pavement structure may dictate the type of pavement selected. Other considerations such as speed of construction, accommodating traffic during construction, safety of traffic during construction, ease of replacement, anticipated future widening, seasons of the year when construction must be accomplished, and others might have a strong influence on paving type selections in specific cases.

- *Initial Cost*: While it is desirable to compare pavement costs on the basis of the entire life-cycle, it must be recognized that available resources are finite. In cases where a pressing need for construction exists, deferring needs until adequate resources are available to build a more expensive structure may not be an option. In these cases, first cost becomes an overriding concern in the selection process.

- *Adjacent Existing Pavement*: Provided there is no major change in conditions, the choice of a pavement type may be influenced by adjacent existing sections that have given adequate service. The resultant continuity of pavement type serves to simplify maintenance and rehabilitation activities.

- *Stimulation of Competition*: It is desirable that monopoly situations be avoided and that improvement in products and methods be encouraged. These goals are aided by healthy competition among industries involved in the production of paving materials.

- *Ease of Maintenance*: Certain pavement alternatives may provide a superior life-cycle cost, but may also entail frequent or complex maintenance activities. While SCDOT strives to provide excellent maintenance for its facilities, there is no assurance that additional resources may be available for options that require unusual levels of maintenance. Consequently, pavement designs should be considered realistically when their future performance is based on critical maintenance activities.

- *Local Preference and Recognition of Local Industry*: While these considerations may seem to be outside the realm of pavement design, highway administrators cannot always ignore them. This is especially true when many other factors involved are indecisive with respect to the selection process.

SCDOT has a formal pavement type selection procedure detailed in Engineering Directive Memorandum 15.

E. Selection of Pavement Design Life

Selection of pavement design life should be coordinated with a Pavement Management System. In South Carolina the traditional design life for new flexible and rigid pavement has been 10 and 20 years respectively. The lesser design life for flexible pavement is in recognition of flexible pavement's historical tendency to deteriorate after approximately 12 to 15 years regardless of traffic. The activities necessary to address this deterioration also provide an incremental structural increase sufficient to extend the pavement life to 20 years or more. Also, flexible pavements are easy to improve structurally with overlays while rigid pavements are not easily improved once constructed. For pavement designs using Superpave mixes and modified binders, a 20-year design life is used in the hope that the improved binders will provide a longer functional life than traditional asphalt mixes. A 10-year design life is used for pavements with conventional mixes.

Rigid pavement constructed with "modern" features such as positive load transfer at joints, slab lengths less than 20 feet, edge support from tied concrete shoulders or widened lanes, and subdrainage systems are designed for at least 30 years. Since 30-year traffic estimates are not available, the designs are based on double the 20-year traffic estimate.

Observations of in-service pavements indicate that the pavement design procedures used by the SCDOT are quite conservative. Generally, it appears that a typical 10-year flexible pavement design will last approximately 12 to 15 years before requiring rehabilitation. Typical jointed concrete 20-year designs, have typically lasted 25 to 35 years.

IV. Rehabilitation of Existing Pavement

A. Introduction

Much of the major highway mileage in South Carolina has been constructed over the last 45 years. In recent years, the overall emphasis is turning from construction of new roadways to the preservation and improvement of the existing system. A large portion of the system is well past the end of its initial design life and will be required to carry traffic well into the future.

Nationally, the pavement design community is beginning to look at longer-term rehabilitation than has traditionally been performed. This is based on the developing awareness that, for instance, if all pavement rehabilitation activity in a system is designed for a 10-year life, eventually 10 percent of the highway network will require rehabilitation every year. In order to develop a network condition that is sustainable, pavement rehabilitation must be coordinated with pavement management to develop an appropriate strategy for selection of rehabilitated pavement design life.

Pavement rehabilitation activities fall into two broad classes. The first type of activity is structural, that is, it increases the load carrying capability of a pavement structure. Examples of this type of rehabilitation are asphalt overlays for asphalt pavements and the addition of tied PCC shoulders for rigid pavements. The second type of activity is non-structural. This type of activity is performed when the pavement is structurally sound, but no longer provides adequate serviceability. Examples of this type of activity are microsurfacing of flexible pavements and diamond grinding of rigid pavement. Many structural rehabilitation activities also provide a functional improvement, but not vice-versa.

B. Rehabilitation of Flexible Pavement

Flexible pavement structural rehabilitation is based on AASHTO's "Thickness Deficiency" approach. This approach assumes that a pavement in need of rehabilitation is structurally deficient. It further assumes that as a pavement ages and wears, its effective thickness decreases. So, by increasing the pavement's thickness, the structural deficiency may be relieved.

Typically, flexible pavement rehabilitation involves the placement of an asphaltic concrete overlay. Other techniques, such as in-place recycling and rigid (PCC) pavement overlays, may be considered on a special, case-by-case basis.

For structural evaluation of flexible pavement, it is necessary to determine the effective structural value of the existing pavement. Once the effective structural value of the existing pavement is known, it may then be compared to the structural value needed to carry future traffic. The difference between the two values determines the appropriate overlay thickness.

Structural evaluation of existing pavement may be done by two methods. These are the Coefficient Depreciation Method and the Direct Measurement Method. Direct Measurement requires the use of nondestructive testing (NDT) to determine the pavement strength.

The Coefficient Depreciation Method requires engineering judgment to make assumptions based on the visual condition and age of a pavement. If distress appears to be limited to the surface, with only slight to moderate cracking, then it is assumed that distress is occurring primarily in the upper pavement layers. In this case, the surface and binder courses are depreciated to 60 percent of their original structural value while other pavement components are given their original structural value. This results in an AC surface and binder structural coefficient of 0.26 versus the original value of 0.44. If more than 4 inches of surface and binder are present in the pavement, the additional depth is considered part of the base and given a coefficient of 0.34. If cracking is severe, then it is assumed that the distress has extended through the AC-bound material in the pavement and the AC-bound portion of the base is depreciated to 70 percent of its original structural value. This results in a structural coefficient of 0.24 versus an original value of 0.34. If the pavement exhibits rutting and distress indicative of shear failure in the base material, then the damage to the pavement is assumed to extend to the subgrade. This results in depreciating the base material to 80 percent of its original value. These judgements are highly subjective and require personnel familiar with various modes of pavement failure. When properly applied, however, this technique has proven to give an excellent indication of needed overlay thickness.

The Direct Measurement Method is preferred for the determination of flexible pavement structural capacity. This method uses a Falling Weight Deflectometer (FWD) to measure the pavement's deflection response under load. This response is then converted to structural capacity through the backcalculation of pavement properties. The techniques used to convert response to structural capacity are complex and require the use of computer software for analysis. A report titled Design of Flexible Pavement Overlays by Dynamic Deflections (HPR #544) details the procedures used for this type of analysis. The results of this type of analysis are the same as for the Coefficient Depreciation Method, a reduced structural number indicating the current in situ condition.

Once the in situ structural number for a project has been determined, the required structural number is determined exactly as for new flexible pavement. The difference between the new and existing structural numbers is the structural requirement for the overlay. It is recommended that both techniques for structural evaluation be performed. If a large difference exists between the two methods, then the source of the discrepancy should be investigated.

In many cases, the structural capacity will be as high as or higher than the required structural number, indicating no structural need for an overlay. In these instances, restoring adequate serviceability will control the selection of rehabilitation activities. The use of pavement milling to remove aged or unstable material should be examined on a case-by-case basis.

C. Rehabilitation of Rigid Pavement

A variety of techniques are available to rehabilitate rigid (PCC) pavement. Unlike flexible pavement, the in situ structural capacity of rigid pavements is not easy to estimate. Even if the structural capacity of the pavement is known, increasing that capacity to meet future needs is not easily accomplished. The two predominant types of rigid pavement in South Carolina are Plain Jointed and Continuously Reinforced. These pavements function differently and must be rehabilitated with different techniques.

1. Plain Jointed PCC Pavement Rehabilitation

Typically, plain jointed rigid pavement in South Carolina begins to exhibit unacceptable serviceability after twenty to thirty years of service. Aggregates in South Carolina are primarily non-reactive, so the durability of PCC does not present a problem. Rehabilitation activities for plain jointed rigid pavement may include:

Grinding
Retrofit Pavement Drainage
Pressure Grouting
Slab Replacement and Patching
Joint Cleaning and Sealing
Retrofit Concrete Shoulders
Asphaltic Overlay
Crack and Seat
Reconstruction
Rubblizing

Grinding: The most severe problem typically encountered with rigid pavement in South Carolina is faulting. Prior to the mid-1970s, rigid pavement was constructed with undowelled joints and relied on aggregate interlock to provide load transfer between slabs. However, aggregate interlock has been found to be inadequate to withstand the high levels of traffic encountered on Interstate highways. As a result, the pavement faults through differential slab motion and associated pumping of fine material from the base course. Once interlock has been lost, there is currently no cost-effective way to restore it. If this type of distress is encountered, it is usually remedied by grinding the pavement to restore an appropriate profile. Pumping of fines from the base will continue, resulting in the recurrence of unacceptable faulting levels after approximately three to eight years. Grinding can only be applied to severe faulting once or twice before the structural integrity of the slab is compromised. However, when applied to pavements which are otherwise structurally sound, grinding may significantly extend the useful life of rigid pavement.

Retrofit Pavement Drainage: To reduce the rate of pumping, provisions to provide drainage of water from beneath the pavement may be beneficial. However, it is critical that the drainage outlets from such a system be maintained properly. If water is unable to exit these pavement drains, the entrapped moisture can greatly accelerate the distresses the drains were installed to alleviate. Since retrofit pavement drains may be harmful to the pavement structure under some conditions, the SCDOT now only recommends the use of such drainage where clear evidence of moisture related distress is noted.

Pressure Grouting: Another rigid pavement rehabilitation activity to be applied with extreme caution is the use of pressurized grout or expanding urethane foams (sometimes referred to as undersealing) to fill voids beneath slabs. Although pressure grouting can give good results if it is applied with great skill, many times too much grout is applied. The excess volume of grout tends to lift the slab and create a new void behind the original void. Therefore, the use of this technique is only recommended in special cases.

Slab Replacement and Patching: Frequently, pavement distress will appear in a localized area surrounded by otherwise sound pavement. Typically, this type of distress is caused by inadequate subgrade preparation or problems with paving materials. If the conditions are sufficiently localized, then complete removal and replacement of distressed slabs may be an adequate rehabilitation activity. If the distress is even smaller, such as an isolated corner break due to pumping, then a patch may also be acceptable. Due to superior performance, patches should be full-depth.

Joint Cleaning and Sealing: Although ideally a maintenance activity, in South Carolina most jointed pavement in need of rehabilitation is also in need of joint cleaning and sealing. This activity is highly recommended, even if the pavement is to be overlaid with asphalt concrete.

Retrofit Concrete Shoulders: Most of the early concrete pavements built in South Carolina were constructed with very thin asphaltic shoulders. Research has shown that tied concrete shoulders significantly reduce edge stresses in the mainline pavement, resulting in improved pavement fatigue life. However, the addition of tied concrete shoulders to older pavements is not always cost-effective. Generally, if a pavement has undergone significant traffic loading, then the increase in fatigue life is small. As a rule of thumb, tied concrete shoulders are added only when 5 percent or less of the slabs in a pavement are broken. Beyond this level, it is assumed that the pavement is worn to the point that the increase in fatigue life is negligible. Also, the addition of tied concrete shoulders was previously thought to reduce slab faulting. However, it has been found that retrofit tied shoulders do not reduce faulting appreciably.

Asphaltic Overlay: If the concrete pavement is showing signs of fatigue failure such as transverse mid-slab cracking, one rehabilitation technique is to clean and seal the pavement joints, repair severe distress, and then overlay with approximately 4 inches of asphalt. Although these overlays have performed within expectations, several problems tend to occur. Joints in the concrete pavement will cause reflective cracks in the asphalt overlay. Once the asphalt becomes cracked, water infiltrates the pavement structure and becomes trapped at the asphalt/PCC interface and may also penetrate to the PCC pavement's base through unsealed joints. The water at the asphalt/PCC interface develops high hydraulic pressures under wheel loads. The high pressures can, in turn, frequently cause the asphalt to debond from the aggregate in the paving mixture and become unstable. This is referred to as stripping. These cracks also may spall under traffic loading. Research is ongoing to reduce these problems, but little success has been achieved to date. The most successful technique for reflective crack control has been to saw and seal the asphalt overlay. Use of hydrated lime and crushed aggregates in asphalt concrete, both of which are now required, should reduce the level of stripping encountered.

Crack and Seat: Crack and seat is a technique to reduce reflective cracking. It involves cracking the concrete slabs into pieces approximately 1½ to 3 feet long, seating the cracked pieces with a heavy roller, and placing an asphaltic overlay. By reducing the slab length, the slab action of the PCC pavement is reduced, resulting in reduced reflective cracking. This technique is only performed in cases where extensive distress makes pre-overlay repair more expensive than cracking and seating. It is important to note that the cost of cracking and seating not only includes the cracking operation, but also more extensive traffic control and a greater asphaltic overlay thickness.

Reconstruction: If pavement distress has become extremely severe, an ultimate rehabilitation option is reconstruction of the pavement. For rigid pavement, reconstruction is defined as the total removal and replacement of at least the PCC layer, and possibly some base layers. The great advantage of this strategy is that design changes, such as the addition of subsurface drainage and load transfer, may be made as part of the reconstruction.

Although it is technically feasible to remove a rigid pavement and replace it with a flexible pavement, in this instance rubblizing (described below) would be a more logical approach since removal of the concrete would involve the loss of any structural contribution of the concrete and entail considerable transportation and disposal costs.

The two main disadvantages of reconstruction is the cost, as compared to the other strategies, and the difficulty of traffic control during construction. However, as our rigid interstate pavements progress into their fifth decade of service and beyond, this strategy may become more commonplace assuming issues such as traffic control can be addressed.

Rubblizing: Another extreme option in rigid pavement rehabilitation is rubblizing. Unlike cracking and seating, which seeks to induce closely spaced cracks in the old rigid pavement, the intent of rubblizing is to reduce the PCC in the pavement to the equivalent of an unbound aggregate base. This totally eliminates the problem of reflective cracking in the overlaid pavement.

The disadvantage of rubblizing is that extensive traffic control is needed during construction, and the structural capacity of the underlying concrete is greatly reduced. Because of the lower structural capacity, typically a very thick overlay is required to achieve structural adequacy. In addition to the cost of the asphaltic material, considerable grading may be required to adjust the shoulders and median to the new pavement grade. Also, reconstruction near bridges or bridge jacking may be needed to achieve adequate overhead clearance.

2. Rehabilitation of Continuously Reinforced Concrete Pavement

The interconnected nature of continuously reinforced concrete pavement (CRCP) requires different techniques for rehabilitation from jointed plain concrete pavement (JPCP). Since CRCP does not have pavement joints, faulting is not a problem. However, through the mid-1970's when most of South Carolina's CRCP was designed, it was believed by pavement designers that CRCP could be built approximately 2 inches thinner than an equivalent jointed pavement. This has had severe consequences for the longevity of CRCP. Examination of the AASHTO pavement design equation shows that for each inch of thickness added to a rigid pavement, the pavement's life is roughly doubled. Therefore, cutting pavement thickness by 2 inches effectively reduces the pavement life by a factor of four.

Because of earlier pavement design practices, much CRCP in South Carolina is exhibiting fatigue-related distresses, primarily longitudinal wheelpath cracks and punchouts. Since these distresses are signs of structural inadequacy, simple repair of punchouts is frequently not sufficient because the punchouts will continue to occur at an accelerating rate. At this stage, some form of structural improvement is needed. Although PCC overlays have been tried in other states, South Carolina has no experience with this type of repair and problems encountered in neighboring states tend to preclude using bonded PCC overlays.

Experience in South Carolina as well as in other states has shown that asphaltic overlays are excellent at arresting punchout development. However, the mechanics of punchout reduction for this repair strategy are not well understood. Linear elastic analysis of asphalt over concrete pavement systems indicates that a 4 inch (10 cm) asphaltic layer over a CRCP has little or no effect on the stress developed within the CRCP layer. It is theorized that the asphaltic overlay serves to reduce temperature-induced stresses, preventing the formation of harmful tensile stresses in the CRCP.

APPENDIX A
DETERMINATION OF SOIL SUPPORT VALUE

APPENDIX A.

Preliminary Site Investigation for Pavement Design and Assignment of Soil Support Value

The objective of the preliminary site investigation for pavement design is to become familiar with the topographic and geological conditions of the proposed highway location. During this investigation, the preliminary plans are studied, if available, and a visual inspection of the proposed location is usually made to determine the type of drilling equipment needed and resolve any problems the drilling crew may encounter during drilling operations.

1. Soil boring and sampling

Usually, soil borings are made along the centerline of the proposed location at approximately 500-foot intervals. In the case of rehabilitation of existing pavement, if the original soil boring information is unavailable, borings may be made in the shoulder. The borings are extended to a depth of five feet below the proposed grade line in cut sections and five feet below the existing ground line in fill sections. Additional borings are usually made in potential problem areas such as deep cuts, swamps or marshes, and section where high groundwater table and/or rock is encountered. Soil samples are taken from each stratum at each boring site. A boring log is maintained for each boring site indicating the depth, texture and color of each soil sample. The approximate depths to rock and the water table are also recorded when encountered.

Soils samples are taken from the full depth of the borings, even in areas where cut is planned. Although the soils in the strata above the proposed grade will not underlie the pavement at the location of the boring, it is likely that this material will be used as fill elsewhere along the project. Consequently, all soil samples are evaluated unless it is clearly specified that the cut material will be hauled off the project and wasted. Otherwise, all samples taken during the boring and sampling operation are identified by location, depth, and color and classified in the laboratory.

At boring sites where groundwater is encountered, the approximate elevation of the groundwater table is established by leaving the bore hole open and checking the depth to water after a minimum of four hours in the case of cohesionless soils and after a minimum period of 24 hours for cohesive soils. Although this information is recorded, the user of this information is strongly warned that the groundwater table may vary seasonally and in response to climatologic change, such as drought. Consequently, water table location may be prone to change between the time of investigation and construction.

2. Soil Support Value

The design method outlined in the SCDOT Pavement Design Guide was developed from tests and measurements on experimental sections of highway built at the original AASHO Road Test in Ottawa, Illinois and supplemented by a program of investigation at Clemson University and the University of South Carolina. To allow application of the AASHTO design method at locations outside of Ottawa, the Soil Support Value (SSV) scale was developed. In order for this method to be used by the SCDOT for design purposes, a correlation has been established between SSV and values obtained for soil testing procedures conducted by the SCDOT Office of Materials and Research.

The Soil Support Value is a dimensionless number indicating the quality of the soils at a given location relative to the original Road Test. Values greater than 3.0 are indicative of soils with roadbed support capabilities superior to those in Ottawa. However, when the original pavement design methodologies were being developed, no consensus on a standardized test could be reached because different areas of the county have different roadbed support issues. Consequently, each state was left to develop a SSV scale based on their experience and testing preference as part of the overall calibration process necessary to adopt the AASHTO methodology in a given location.

As a result, methods and values assigned to given soil types vary widely around the country. Generally, these differences are offset by differences in design life, terminal serviceability, and other inputs to the design procedure. The outcome of this process is that **SSV CORRELATIONS DEVELOPED IN OTHER STATES ARE NOT VALID WHEN USED WITHIN THE OVERALL PAVEMENT DESIGN PROCESS IN SOUTH CAROLINA!!! TO MAINTAIN THE ORIGINAL ASSUMPTIONS INHERENT IN THE EMPIRICAL**

<u>DESIGN PROCESSES, THE CORRELATIONS GIVEN IN THIS REPORT MUST BE USED. VALUES FROM NORTH CAROLINA, GEORGIA, OR TEXAS, AS WELL AS OTHER STATES, ARE NOT VALID SINCE OTHER FACETS OF THEIR DESIGN METHODOLOGIES ALSO VARY FROM SCDOT PRACTICE.</u>

3. <u>AASHTO classification</u>

Soil samples are prepared for testing at the laboratory in accordance with AASHTO T 87, "Dry Preparation of Disturbed Soils and Soil Aggregate Samples for Test". The soils are then tested using procedures as set forth in SC-T-34, "Mechanical Analysis of Soils (Elutriation Method)"; AASHTO T 89, "Determining the Liquid Limit of Soils"; and AASHTO T 90, "Determining the Plastic Limit and Plasticity Index of Soils". After these tests are completed, each soil sample is classified according to AASHTO M 145, "The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes". The result of all the tests are added to the soil boring log to show the AASHTO classification of each soil sample along with rock and/or groundwater information and the logs are distributed to field and road design personnel for their use.

4. <u>CBR testing</u>

The number of soil samples taken for CBR testing depends upon soil conditions. However, as a general rule, one CBR sample is taken per mile of roadway. These samples are usually taken from the soil stratum at or near the proposed subgrade elevation and representative of the predominant soil types encountered.

Soil samples for CBR tests are prepared and tested initially from AASHTO classification as outlined previously. AASHTO T 99, "The Moisture-Density Relations of Soils Using a 5.5-pound Rammer and a 12-inch Drop", is used to determine the maximum dry density and optimum moisture content. Method B is used for samples on which CBR tests are to be conducted.

After the maximum dry density and optimum moisture content have been determined, test specimens are molded at the optimum moisture content and 95% of the maximum dry density. CBR test specimens are moded, cured, and tested in accordance with AASHTO T 193, "The California Bearing Ratio", except that only two specimens are molded and tested. The actual density of the two specimens should bracket 95% maximum dry density. A linear interpolation is used to determine and report the CBR value at 95% density.

5. Triaxial testing

In addition to testing specimens for strength using CBR, static triaxial testing may also be used to determine the specimen properties for SSV. Static triaxial testing was the basis for the original calibration conducted to implement the AASHO design methods in the 1960's. However, because CBR is easier to determine, this method is predominantly used at present. For further information on triaxial methodologies for SSV, the Geotechnical Materials Engineer at the SCDOT Office of Materials and Research should be contacted.

6. Analysis to establish SSV

The SSV for an individual sample is determined from Figure A-1, which has been prepared to show the relationship between CBR value and SSV. A comparison of CBR and triaxial tests on a large number of samples was used as the basis of this relationship. Curves were developed for the Piedmont and Coastal Plain regions of the state.

A tabulation of AASHTO classification of all soil samples from the project is made by listing the samples in order of their occurrence along the centerline. Samples that have actual soil support values derived from CBR or triaxial testing are also listed. Historical data for similar soils nearby, where available, is consulted to assign other samples an individual SSV. Lacking historical data, additional CBR or triaxial tests may be required.

A chart is then prepared showing SSV versus the percentage of soils having a SSV equal to or greater than the SSV. The SSV recommended for pavement design purposes is that value which 70% to 90% of the soils on the project equal or exceed. It is not economically feasible to design a pavement structure based on the poorest quality soils encountered on a project. Soils with SSVs somewhat less than the design SSV are usually removed and replaced with better quality materials during construction.

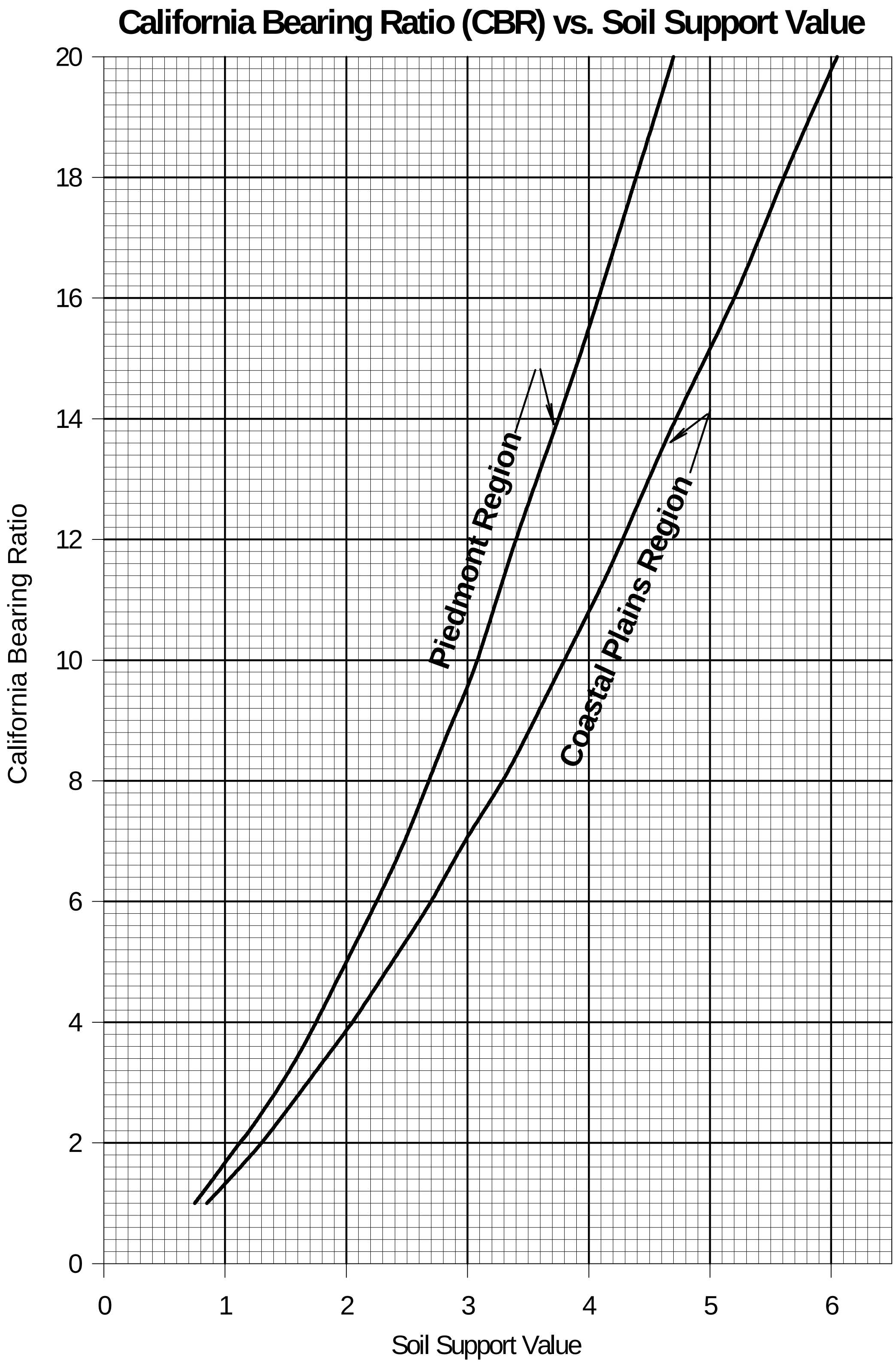

Figure A-1, CBR versus Soil Support Value

The exact SSV assigned to a given project is also a function of the engineer's assessment of the potential for variation within a project. Typically, it is not feasible to have multiple SSV's or multiple pavement designs within a single project. As a result, the engineer must also consider the potential sources for variation in subgrade support within that project. For instance, a large project with substantial fill, such as might be encountered in the flat, coastal regions, might derive much of its subgrade material from multiple borrow sources. While it is generally assumed that the soil samples obtained along the centerline of the project would be representative of sources of borrow, this is not actually known at the time of SSV assignment. Additionally, mixing of borrow sources may result in unpredictable soil properties on the roadway. Accordingly, the engineer would typically choose a lower SSV to account for this potential variation.

On the other hand, a smaller project with little or no borrow being mostly constructed at-grade has a lower potential for variability. Consequently, the engineer may choose to assign a higher SSV on the basis of similar test results solely on the basis of an estimation of variability. No "plug-and-chug" rule applies for selection of SSV for an entire project. An engineer experienced in the assignment of SSV, fully aware of the historical and empirical aspects of this value, and informed of the nature of the particular project should make this final selection.

APPENDIX B
PAVEMENT DESIGN EXAMPLE

January 3, 2003

MEMORANDUM

To: Mr. Derek J. Piper, Program Manager, CRM East

From: Dr. Andrew Johnson, State Pavement Design Engineer

SUBJECT: PAVEMENT DESIGN RECOMMENDATIONS, SC-6/60, FROM S-32-175 TO US-378, LEXINGTON COUNTY
FILES 32.147B, 32.148B

As requested, we have reviewed conditions at the subject site in order to provide pavement design recommendations.

Project Overview
The project consists of widening SC-6 from US-378 to SC-60 and widening SC-60 from SC-6 to S-175. The intersection of SC-6 and SC-60 will be replaced with a grade-separated interchange. It is assumed that the areas away from the Lake Murray Dam will be widened mostly about the centerline and the pavement in the vicinity of the SC-6 and SC-60 intersection will be replaced as part of the intersection reconstruction.

The traffic across the dam will be separated. Traffic heading from Irmo to Lexington will use the existing road on the crest of the dam, while traffic heading towards Irmo will use a new roadway constructed between the existing dam and the new dam. Pedestrian and bicycle facilities will be added to the existing roadway at the crest of the dam.

Soil Conditions
Mr. Piper provided soil information to us. Based upon a review of this information and previous investigations in this area, an SSV of 1.3 is recommended for SC-60 and and an SSV of 1.6 is recommended for SC-6.

Traffic Conditions
From US-378 across the dam, the current ADT is estimated to be 16,400 with 10 percent of the traffic being trucks. This is estimated to increase to 26,800 ADT in 10 years and 36,000 ADT in 20 years. For pavement design, this results in a 10-year ESAL estimate of 3,011,000 ESALs and a 20-year estimate of 6,977,340 ESALs.

For SC-60 from the SC-6 intersection to S-32-175, the current ADT is estimated to be 10,800 with 10 percent of the traffic being trucks. This is estimated to increase to 26,300 in 10 years and 35,300 in 20 years. This results in a 10-year ESAL estimate of 1,780,000 ESALs and a 20-year estimate of 4,420,000 ESALs.

Existing Pavement Conditions

The pavement throughout the project was originally concrete. Since original construction, probably in the early 1930's, the concrete was overlaid and widened using asphalt and has been overlaid multiple times. The original PCC appears to be in good condition with the exception of several blow-ups resulting from inadequate joint cleaning prior to original overlay. The pavement joints have reflected through the asphalt, but faulting is of low severity. Maintenance forces have previously attempted to seal cracks in the asphalt to extend the pavement life.

The previous widening is in poor condition. Moderate subsidence of the widening was noted throughout both projects as well as numerous areas of moderate to high severity distress and spalling. The pavement across the dam had numerous blow-ups, but was otherwise in good condition.

Pavement Design Recommendations

Based on the information provided, a pavement SN of 4.96 is recommended for SC-60. For SC-6 from the dam to US-378, a pavement SN of 4.91 is recommended. As is typical for all designs, these are based on ten-year traffic projections. However, because of the limited access to the pavement across the dam in the future, a 20-year design is recommended for this area. Consequently, for the pavement across the dam, a pavement SN of 5.49 is recommended. The following pavement structures are recommended:

SC-60 from S-175 to SC-6 and SC-6 from dam to US-378:	200 psy AC Surface – T1C
	200 psy AC Binder – T1
	1000 psy Asphalt Aggregate Base – T1

This pavement has a SN of 4.92 versus target values of 4.96 and 4.91.

SC-6 across dam:	200 psy AC Surface – T1C
	200 psy AC Binder
	750 psy Asphalt Aggregate Base – T1
	8 inches Graded Aggregate Base

This pavement has a SN of 5.55 versus a target value of 5.59. An additional 450 psy AAB – T1 may be used in lieu of the GAB for tapers and other areas where GAB placement is not feasible.

For repair of existing pavement, with the exception of the pavement across the dam, it is recommended that the previous widening be removed, any blow-ups be spot milled to restore a smooth surface, and the pavement be overlaid with 200 psy AC Surface – T1C and 200 psy AC Binder – T1.

For the pavement across the dam, we recommend that the existing pavement be removed and replaced with the new pavement structure recommended above.

Concrete Pavement
In accordance with SCDOT's recently approved pavement type selection policy, a life cycle cost analysis was performed for the pavement over the Lake Murray dam because the pavement is on new location and the required structural number is over 4.5. This analysis is attached. Concrete pavement was found to be 113% of the cost of asphalt and 98% of the cost of asphalt in long-term cost. Because of the short length of the project and to maintain continuity, asphalt pavement is recommended; even though concrete is approximately the same cost in the long-term.

Please let us know if we may provide further information.

Andrew M. Johnson

AMJ/amj
File RM:MLC

cc: Mr. Al Barwick, Construction
Mr. Jim Cagney, District 1
Mr. Robert Pratt, Preconstruction

Flexible Pavement Design
SCDOT Research and Materials Laboratory

Road:	SC-60
County:	Lexington
Location:	From: SC-6 To: S-32-175
Street:	
File Number:	32.147B

Traffic Data	Year	Years in Period	Estimated ADT
Begin Design Period	2001		10,800
End Intermediate Design Period	2011	10	26,300
End Ultimate Design Period	2021	20	35,300

Structural Number Determination					
Design Period	5	10	15	20	10-20
Terminal Serviceability	2.0	2.0	2.0	2.0	2.0
Soil Support Value	1.3				
Structural Number	4.41	4.96	5.32	5.59	5.23
Regional Factor	1.0	1.0	1.0	1.0	1.0
Weighted Structural Number	4.41	4.96	5.32	5.59	5.23

Remarks:

Prepared by: Andrew Johnson
Title: State Pavement Design Engineer
Date: December 31, 2002

Flexible Pavement Design
SCDOT Research and Materials Laboratory

Road:	SC-6
County:	Lexington
Location:	From: Lake Murray Dam To: US-378
Street:	
File Number:	32.148B

Traffic Data	Year	Years in Period	Estimated ADT
Begin Design Period	2001		16,400
End Intermediate Design Period	2011	10	26,800
End Ultimate Design Period	2021	20	36,000

Structural Number Determination					
Design Period	5	10	15	20	10-20
Terminal Serviceability	2.0	2.0	2.0	2.0	2.0
Soil Support Value	1.6				
Structural Number	4.41	4.91	5.24	5.49	5.09
Regional Factor	1.0	1.0	1.0	1.0	1.0
Weighted Structural Number	4.41	4.91	5.24	5.49	5.09

Remarks:

Prepared by: Andrew Johnson
Title: State Pavement Design Engineer
Date: December 31, 2002

Conversion of Pavement Design to Thickness
Project: SC-6/60, Lexington County

1. Surface and Binder Courses

Material	Rate (psy)	Thick. (in)	New a_1	Dep A_1	SN Year 1-10	SN Year 11-20
AC Surface – T1C	200	1.91	0.44		0.84	
AC Binder – T1	200	1.91	0.44		0.84	
SUBTOTAL					1.68	

2. Base Courses

Material	Rate (psy)	Thick. (in)	New a_2	Dep A_2	SN Year 1-10	SN Year 11-20
Asphalt Aggr. Base – T1	1000	9.52	0.34		3.24	
					3.24	

3. Subbase Courses

Material	Rate (psy)	Thick. (in)	New a_3	Dep A_3	SN Year 1-10	SN Year 11-20
SUBTOTAL					0.00	0.00

TOTAL SN	4.92	
TARGET SN	*	

Remarks: *Target is 4.91 for SC-6 and 4.96 for SC-60

Conversion of Pavement Design to Thickness
Project: SC-6/60, Lexington County

1. Surface and Binder Courses

Material	Rate (psy)	Thick. (in)	New a_1	Dep A_1	SN Year 1-10	SN Year 11-20
AC Surface – T1C	200	1.91	0.44			0.84
AC Binder – T1	200	1.91	0.44			0.84
SUBTOTAL						1.68

2. Base Courses

Material	Rate (psy)	Thick. (in)	New a_2	Dep A_2	SN Year 1-10	SN Year 11-20
Asphalt Aggr. Base – T1	750	7.14	0.34			2.43
Graded Aggr. Base		8	0.18			1.44
						3.87

3. Subbase Courses

Material	Rate (psy)	Thick. (in)	New a_3	Dep A_3	SN Year 1-10	SN Year 11-20
SUBTOTAL					0.00	0.00
TOTAL SN						5.55
TARGET SN						5.49

Remarks:

APPENDIX C
AASHTO PAVEMENT DESIGN NOMOGRAPHS

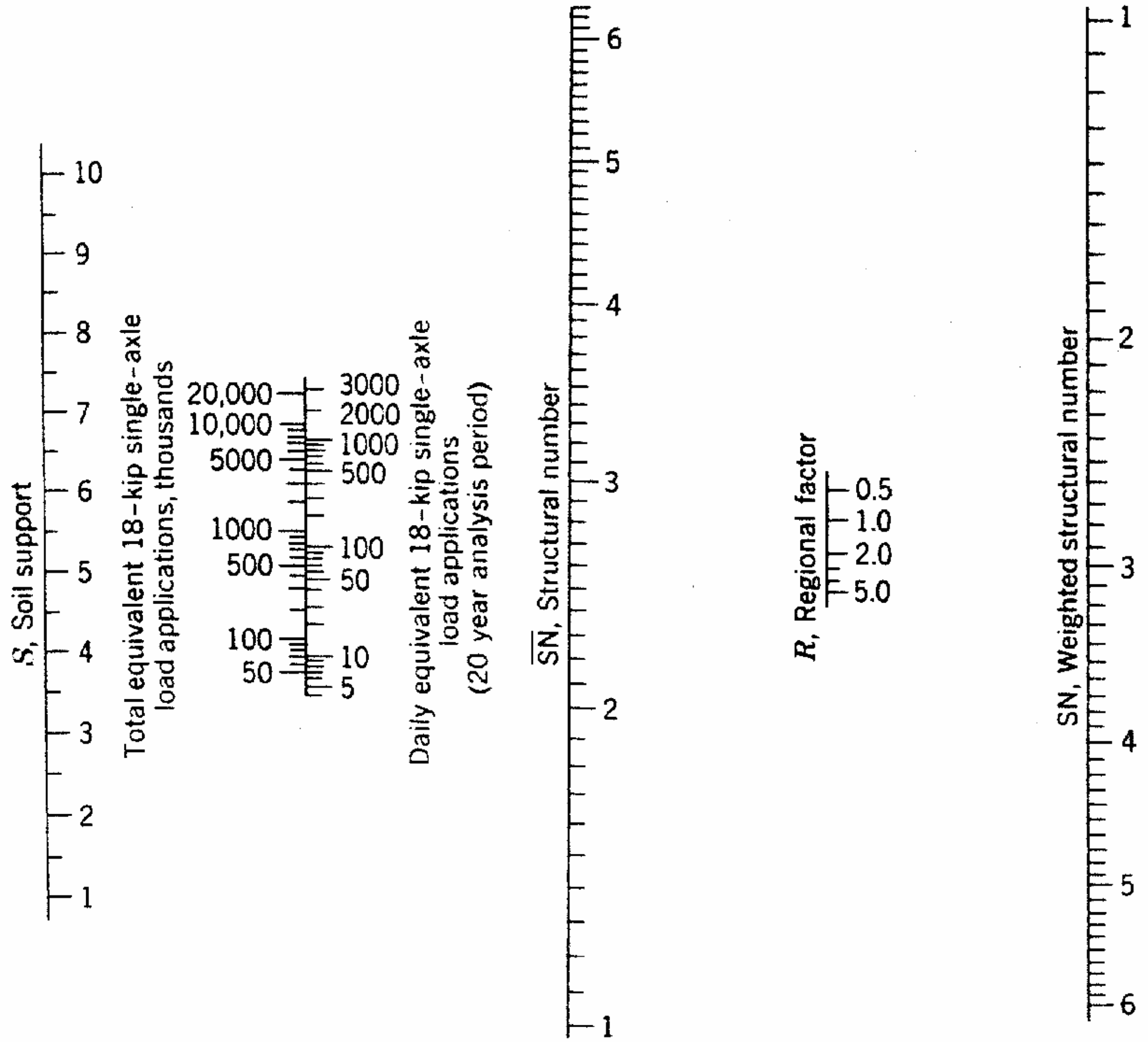

Figure 1 - AASHTO Flexible Pavement Design Nomograph, $p_t = 2.0$

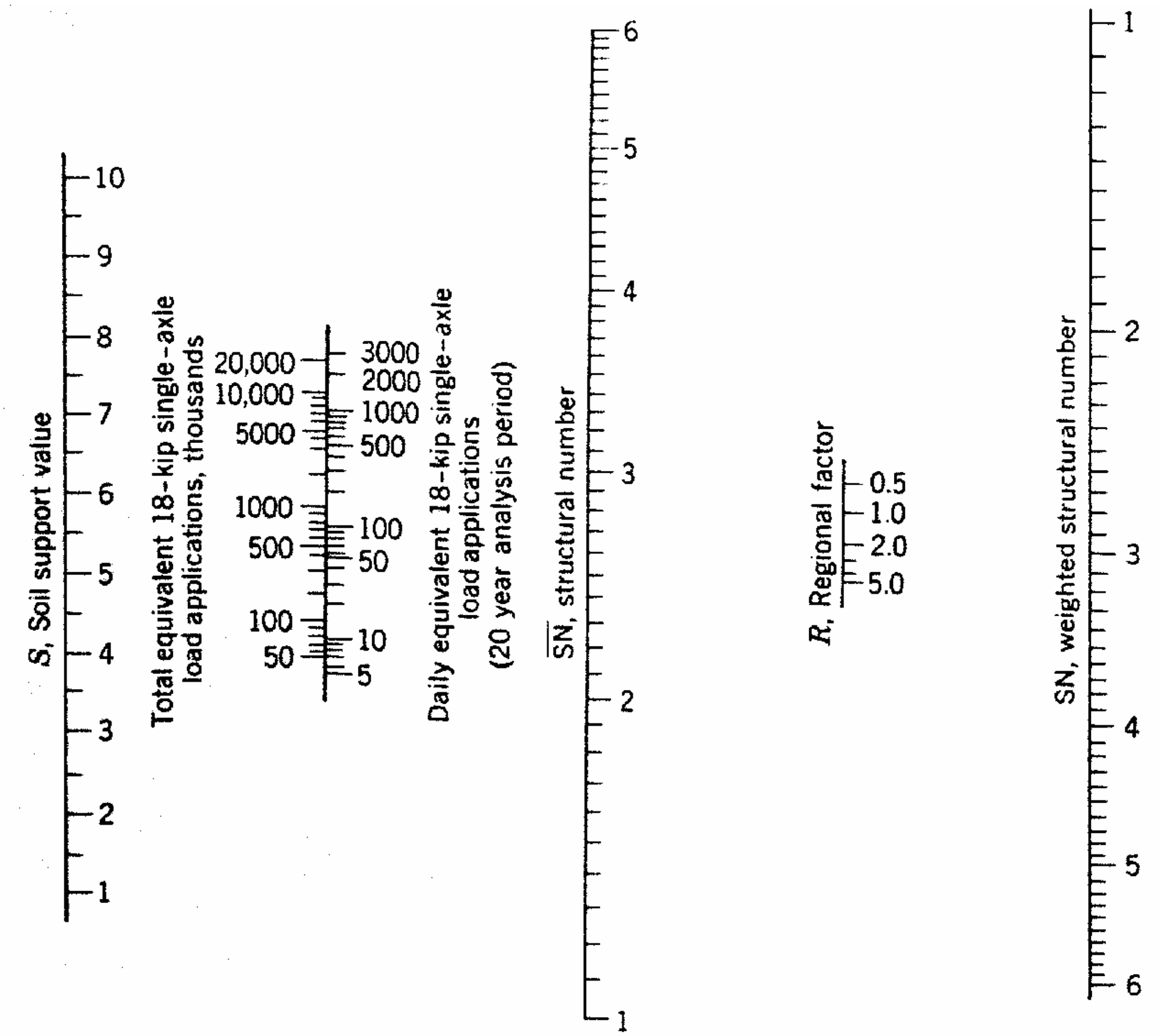

Figure 2 - AASHTO Flexible Pavement Design Nomograph, $p_t = 2.5$