Improved Methods for Selection of *k* Value for Concrete Pavement Design

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The results of research conducted to improve guidelines for *k*-value selection for concrete pavement design are summarized in this paper. The research included a review of the evolution of *k*-value concepts and methods, a review of *k*-value results from several field studies, an examination of the AASHTO Guide's *k*-value methods, and proposed new guidelines for selection of design *k* values by a variety of methods. The *k* value was originally considered a useful and simple parameter for characterizing slab support provided by natural soils of fairly low shear strength. Recognizing that real soils are not true dense liquids, early researchers developed standardized test methods which provided *k* values in good agreement with full-size slab deflections. Later, substantially higher *k* values were attributed to granular and stabilized base layers, based on plate tests on top of bases, although slab tests had shown that such bases did not increase *k* values. Based on the historical review, review of results from several field studies, and a thorough examination of the *k*-value methods introduced in the 1986 AASHTO Guide, it is recommended that *k* values be selected for natural soil materials, and that base layers be considered in concrete pavement design in terms of their effect on the slab response, rather than their supposed effect on *k* value. Improved guidelines were developed for determining *k* value from a variety of methods, including correlations with soil type, soil properties, and other tests; backcalculation methods; and plate-bearing test methods. Guidelines for seasonal adjustment to *k*, and adjustments for embankments and shallow rigid layers were also developed.

In the AASHTO Guide for Design of Pavement Structures, and in all other accepted concrete pavement design procedures, the support a foundation provides a concrete pavement is characterized by a *k* value, which represents a *dense liquid* (elastic springs) foundation. The AASHTO Guide also has a loss of support input which is intended to represent the potential for reduction of support at slab corners over the design life of the pavement. However, the AASHTO Guide's procedures for selecting these inputs do not produce values which are truly representative of the *support* the concrete slab will experience over its design life, and which are truly representative of the effect of support on the performance of the concrete pavement.

Conventional plate-bearing tests were often conducted to determine subgrade *k* values through the 1950s, and even to the 1980s by some agencies. These tests are extremely expensive and time-consuming, and thus are rarely conducted today. Other approaches exist for estimating *k* values for design, including correlation with soil properties and other soil tests, and backcalculation from deflection testing on concrete pavements. The different approaches to selecting *k* values often give different results.

Since the 1960s, *k* values intended to represent plate-bearing test values estimated on top of the base have commonly been used in concrete pavement design. These *top of base* or *composite k* values overestimate the support the slab actually experiences in the field. This is particularly true for stabilized base layers or existing concrete or asphalt pavement structures. The *composite k* concept also does not realistically reflect the effect that a base layer has on stress in a concrete slab due to load, temperature, and moisture influences. A better approach to characterizing concrete pavement support would assign realistic *k* values to natural subgrades and embankments, and account for base effects in slab stressed.

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EVOLUTION OF THE *k* VALUE

A thorough historical review of the *k*-value concepts and methods which have evolved over the last hundred or more years was conducted for this study and yielded valuable insights into the meaning and practical significance of the *k* value. This review is documented in detail in References 1 and 2, and its main findings are briefly summarized below.

Introduction of Dense Liquid Support Model

The concept of a foundation which deflects in proportion to an applied vertical load, without shear transmission to adjacent areas of the foundation, dates back to the nineteenth century. The dense liquid model represents one end of the spectrum of elastic soil response (the other end of the spectrum is the elastic solid model). The elastic response of real soils lies somewhere between these two extremes. Furthermore, the behavior of real soils is not purely elastic, but plastic, and time-dependent as well. The *k* value of saturated cohesive soils may be substantially higher under rapid-loading than under slow-loading, because under slow-loading, primary consolidation occurs gradually as pore water pressures dissipate. In most cases, the deformation of the soil reaches some stable value, but soils may also exhibit secondary (creep) deformation, if the load magnitude exceeds the creep strength of the soil (*3,4*). Consolidation and creep behavior of soils necessitate some standardization of load test methods.

Westergaard's Equations for a Slab on a Dense Liquid

Westergaard presented the first equations for deflection of a concrete slab on a dense liquid foundation, and also introduced the terms *modulus of subgrade reaction* for the spring constant of the subgrade and *radius of relative stiffness* for the stiffness of a concrete slab relative to that of the subgrade (*5*). Westergaard suggested that the subgrade *k* value could be backcalculated from deflections of the slab surface rather than from load tests on the subgrade (*5,6*).

Arlington Road Tests

In the early 1930s, the Bureau of Public Roads conducted extensive field tests to investigate concrete pavement behavior. One of the objectives of these field tests was to verify Westergaard's equations. Among the many valuable findings of the Arlington tests were those concerning measurement of subgrade k values, effects of seasonal moisture variation on k values, effects of slab-curling on corner k values, and effect of subgrade *improvement* on k values (*7*). The Arlington researchers did extensive experimentation to develop methods to determine the subgrade k value from plate load tests and from full-size slab testing. The k values determined from repeated loads on large plates (e.g., at least 30-in [762-mm] diameter) at a deflection at 0.05 in [1.25 mm] yielded k values which agreed well with those backcalculated from deflections induced by loads on top of concrete slabs.

Corps of Engineers Field Studies

In the 1940s, the U.S. Army Corps of Engineers conducted load tests on subgrades and concrete slabs at Wright Field in Ohio and several other airfields. One of the objectives of the Wright Field slab tests was to develop a standard test method for determining subgrade k values (*8*). The k values obtained at a deflection of 0.05 in [1.25 mm] using a 30-in. [762 mm] diameter plate consistently yielded subgrade k values in close agreement with volumetric k values obtained from test on concrete slabs (calculated by dividing the load by the volume of the deflection basin) (*8,9,10*). "The only exception to this pattern is the high k value obtained on moderate base course thicknesses which generally must be adjusted downward to match full-size slab performance" (*8*). The Corps of Engineers' test method for k became the basis for the ASTM and AASHTO standard test methods developed later.

Correlation of k Value and CBR and Soil Classification

In 1942, Corps of Engineers researchers published perhaps the first chart correlating k value to California Bearing Ratio (CBR) and the Unified and Public Roads (now AASHTO) soil classification groups (*10*). This chart became the basis for correlation charts and tables later published in concrete pavement design manuals by the U.S. Army and the Portland Cement Association.

Effect of Base Layers on k

In the 1940s, numerous reports appeared in the literature concerning plate load tests on subgrades and on base layers. These studies contributed to a trend to quantifying k value increases as a function of base thickness and base material. The Corps of Engineers also apparently changed its position on the effect of base layers on k value, but apparently did not attempt to validate its design curves for "base k value" with deflection tests on concrete pavements (*11*).

ASTM Plate-Bearing Test Methods

The first American Society for Testing and Materials (ASTM) test methods for plate-bearing tests on soils were published in 1952: D 1195, Repetitive Static Plate Load Test, and D 1196, Nonrepetitive Static Plate Load Test. These were based on the Corps of Engineers test methods and have changed very little since they were originally published. Neither of the ASTM test methods gives any guidance on calculation of the subgrade k value from the test results, unlike the Corps of Engineers test method, and the AASHTO test methods T221 and T222, which were not standardized until the 1960s.

AASHO Road Test

Plate load, California Bearing Ratio (CBR), moisture content, and density tests were made on the subbase and the embankment at the AASHO Road Test. At the time of the Road Test, AASHO did not have standard test methods for plate-bearing tests, and the test procedure used did not conform to the then-current ASTM or Corps of Engineers standards. The procedure used was similar to that used at the Arlington Road Test, and involved cycling loading and unloading at three load levels using a 30-in. [762-mm] diameter plate. An average elastic k value was determined by dividing each of the individual loads by the elastic deformations they produced and a gross k value was determined for each load level by dividing the load by the total deformation produced, including permanent deformation. The elastic k values averaged 77 percent greater than the corresponding gross k values (*12*).

A k value of 60 psi/in. [16 kPa/mm] was used to represent AASHO Road Test conditions in the development of the AASHO rigid pavement design equation (*13*). This is the mean springtime gross k value from tests on top of the subbase. It is almost as conservative a value as could possibly have been picked to represent the Road Test conditions. The only more conservative value would have been the slightly lower springtime gross k value of 49 psi/in. [13 kPa/mm] on top of the subgrade. Why the subbase gross k was selected rather than the subgrade k is not documented.

The Corps of Engineers conducted load tests on top of the existing slabs at the AASHO Road Test site in 1962 and calculated volumetric k values between 25 and 92 psi/in. [7 and 26 kPa/mm] from the slab deflection basins (*14,15*). Loop 1 of the AASHO Road Test was tested using a Falling Weight Deflectometer for this study in May 1992. Care was taken in the analysis to account for the effects of temperature, load transfer, slab size, and concrete compressibility. The mean backcalculated dynamic k of 148 psi/in. [40 kPa/mm], when divided by 2, yields an estimated static k of 74 psi/in. [20 kPa/mm], which is within the range obtained from plate load tests on the subgrade and also within the range obtained by the Corps of Engineers from static tests on top of the slabs.

Portland Cement Association

In the 1960s, the Portland Cement Association (PCA) conducted plate tests on subgrade soils, untreated gravel and crushed stone bases, cement-treated subbases, and soil-cement pavements. The tests on the granular bases yielded slightly higher k values than the subgrade k values, but the tests on the cement-treated bases yielded considerably higher k values. Tests on concrete slabs constructed on the cement-treated bases showed decreases in maximum edge and interior deflections with increasing base thickness (*16*). The PCA used these results to develop curves for top-of-base k values for granular and cement-treated bases, which were incorporated in PCA's concrete pavement design procedures.

In this study, the slab deflection data reported by PCA were used to backcalculate k values, and the values obtained were much more similar to those reported for the subgrade plate tests than to those reported for the plate tests on the cement-treated base.

1972 AASHO Interim Guide

The 1972 Interim Guide (*17*) recommended the use of the subgrade gross k value, and provided a nomograph to determine a composite k value on top of a subbase. The 1972 Guide also suggested that an adjustment to the k value might be warranted to reflect loss of support. Both the subbase adjustment and the loss of support adjustment are inconsistent with the derivation of the AASHTO rigid pavement model, which was already calibrated to the subbase k value of the Road Test, and was also calibrated to the performance (including the effect of substantial pumping and loss of support) of the AASHO Road Test's granular-base concrete pavement sections.

Correlation of k to Soil Type and Degree of Saturation

The 1977 Zero-Maintenance study proposed that the k value in various seasons could be estimated from its AASHTO classification and the degree of saturation in the upper 5 ft [1.5 m] of soil (*18*). The curves developed for k value were obtained using correlations between resilient modulus, static elastic modulus, and degree of saturation which were developed from an extensive field and laboratory study of Illinois soils (*19*).

Backcalculation Methods

In the last 15 years, several methods were developed for efficient estimation of k values from deflection test data. These methods used finite element programs or Westergaard's equations to determine the subgrade k value as a function of the deflection basin measured by a Falling Weight Deflectometer or similar device. Nomographs and equations for backcalculation of concrete elastic moduli and foundation k values and concrete E values for concrete or composite pavements were incorporated in the overlay design procedures in Part III of the 1993 AASHTO Guide (*20*).

The dynamic k values obtained from FWD data are typically about twice as high as the static k values which would be expected for the same soils in standard plate bearing tests. The rule of thumb gives reasonable results, as numerous field studies reviewed for this research have shown. It is very difficult to provide a more sophisticated method for converting dynamic k values to static k values, due to the complexities of modelling dynamic soil behavior and the sparsity of side-by-side comparisons of dynamic and static soil response. Nonetheless, it may be true, and future research may show, that the relationship between dynamic and static k varies in a predictable way as a function of soil properties, loading characteristics, or other factors.

AASHTO GUIDE k VALUE METHODS

The 1986 version of the AASHTO Guide (*21*) contained five modifications to the k value guidelines of the 1972 Interim Guide:

1. An equation was provided for k value for an unprotected subgrade;
2. A revised nomograph for composite (top-of-base) k was provided;
3. An adjustment for depth to a rigid foundation was introduced;
4. A seasonal adjustment procedure for k was provided; and
5. A loss-of-support procedure was provided.

k Equation for Unprotected Subgrade

A simple linear relationship described as a "theoretical relationship between k values from a plate bearing test and elastic modulus of the roadbed soil" was presented in the 1986 AASHTO Guide:

$$k = \frac{M_R}{19.4} \tag{1}$$

Part II of the AASHTO Guide makes no distinction between the laboratory-measured resilient modulus of a soil sample (M_R) and the in situ elastic modulus of a subgrade soil mass (E). The relationship between k and resilient modulus given in the Guide's Appendix HH was derived using an elastic layer computer program to model a circular load on an elastic half-space. Because an elastic layer program cannot model rigid plate loading, k was not computed as pressure divided by deflection, but rather as load divided by deflection volume. These two definitions for k are equivalent only when the total deflected volume is equal to the plate deflection times the contact area. However, in the derivation of Equation 1, the k corresponding to each input E was computed by dividing the plate load by only the portion of the deflected volume within the radius of the load plate.

In a real plate load test on a natural subgrade material, the shear stress at the edge of a flexible load plate is equal to the applied pressure. The shear stress at the edge of a rigid load plate is considerably higher. If this shear stress exceeds the shear strength of the soil, the plate will punch down into the soil and relatively little deflection will occur outside the load plate. To the extent that this happens, the real soil's response approaches that of an ideal Winkler foundation. However, an elastic layer program is not capable of reproducing the type of discontinuous deflected shape of the subgrade surface which would really occur in plate tests on most natural soils of relatively low shear strength.

If an elastic layer program is used to model a concrete slab on subgrades with the set of E values used to derive Equation 1, the k values which may be backcalculated from the slab deflections are substantially less than those indicated by Equation 1. A real example of this is the AASHO Road Test soil itself: the laboratory resilient modulus of the Road Test soil was about 3000 psi [20.7 MPa] for springtime moisture conditions. Dividing 3000 by 19.4 yields a k value of about 155 psi/in. [42 kPa/mm], nearly twice as high as the springtime elastic subgrade k value, 86 psi/in. [23 kPa/mm].

Composite k Nomograph for Base and Subgrade

The 1986 AASHTO Guide presented a nomograph for determining a composite k as a function of subgrade resilient modulus and the thickness and elastic modulus of a base layer. The development of this nomograph was documented in the Guide's Appendix LL. Again, the subgrade's laboratory resilient modulus was presumed equal to the in situ elastic modulus. The nomograph was developed by simulating plate load tests with an elastic layer program. A k

value was calculated as the plate pressure divided by the maximum deflection under the plate. One anomaly of the AASHTO composite k nomograph is that, although it yields very high k values for base layers, in some cases these k values are lower than the k values that would be assigned to the subgrade if the base were not present. Furthermore, elastic layer analyses conducted in this study showed that the Guide's composite k nomograph yielded substantially higher k values than those backcalculated from slab deflections.

Adjustment to k for Shallow Rigid Foundation

The 1986 Guide introduced a nomograph for adjustment to the composite k value when a rigid foundation was present at a depth within 10 ft [3 m]. The basis for the AASHTO Guide's nomograph is not documented, although it is presumed to have been developed using elastic layer simulation in a manner similar to the development of the composite k nomograph.

Seasonal Adjustment Procedure for k

The 1986 Guide provided a method for determining a design k value which represents the range of k values expected in various seasons, weighted with respect to the relative damage done to the pavement in those seasons. The relative damage is calculated using the AASHTO rigid pavement design equation. This damage-weighted seasonal adjustment is reasonable in concept, although the nomograph provided suggests that relative damage is sensitive to slab thickness, and close examination of the nomograph and equations reveals that slab thickness has little or no effect. Another inconsistency of the seasonal adjustment procedure is that the design equation itself is not calibrated to a seasonal average k for the AASHO Road Test site, but rather the springtime k value. This inconsistency, by the way, is present in the flexible pavement design procedure as well, where the impact on required pavement thickness is much more dramatic.

Loss of Support Adjustment to k

The nomograph introduced in the 1986 Guide for reducing the k value for potential loss of support due to base erodibility produces dramatic reductions in k values for erodible bases. Little or no adjustment is applied when the base is a relatively inerodible stabilized material. This loss of support adjustment is a major inconsistency in the AASHTO design procedure, because the rigid pavement design equation is based on the performance of AASHO Road Test pavements which had granular bases and experienced substantial loss of support. The loss of support adjustment is also inconsistent with the k value of 60 psi/in. [16 kPa/mm] embedded in the rigid pavement design equation. According to the loss of support nomograph, the granular base at the AASHO Road Test site would be assigned a loss of support factor from 1.0 to 3.0, which would reduce the k value to between 6 and 22 psi/in. [1.6 and 6 kPa/mm].

IMPROVED METHODS FOR DETERMINING k VALUE

The elastic k value on top of the subgrade or prepared embankment is the recommended design input. Only the elastic component of deformation is considered representative of the response of the subgrade to traffic loads on the pavement. Three categories of methods were compiled in this study for estimating the elastic k value of the subgrade for a pavement design project: correlation methods, backcalculation methods, and plate-testing methods.

Correlation Methods

Guidelines were developed for selecting an appropriate k value based on soil classification, moisture level, density, California Bearing Ratio (CBR), Hveem Stabilometer data (R-value), or Dynamic Cone Penetrometer (DCP) data. These correlation methods are anticipated to be routinely used for design. k values obtained from correlation methods may need adjustment for embankment above the subgrade or a shallow rigid layer beneath the subgrade.

k Value Correlations for Cohesionless Soils (A-1 and A-3)

A cohesionless material may be characterized by its shear modulus, which is fairly insensitive to moisture variation and is predominantly a function of void ratio and overall stress state (which in turn are functions of dry density and depth). Recommended k value ranges for A-1 and A-3 soils are given in Table 1.

k Value Correlations for A-2 Soils

Soils in the A-2 class are all granular materials falling between A-1 and A-3. Although it is difficult to predict the behavior of such a wide variety of materials, the available data indicates that in terms of bearing capacity, A-2 materials behave similarly to cohesionless materials of comparable density. Recommended k value ranges for A-1 and A-3 soils are given in Table 1.

k Value Correlations for Cohesive Soils (A-4 through A-7)

Some characteristics of the various classes of cohesive soils are summarized in Table 1. The bearing capacity of these cohesive soils is strongly influenced by their degree of saturation, which is a function of moisture content, dry density, and specific gravity. Recommended k values for each cohesive soil type as a function of degree of saturation are shown in Figure 1. Each line represents the midrange of reasonable values for k. For any given soil type and degree of saturation, the range of reasonable values is about + 40 psi/in. [11 kPa/mm]. So, for example, an A-6 soil might be expected to exhibit k values between about 180 and 260 psi/in. [49 and 70 kPa/mm] at 50 percent saturation, and k values between about 5 and 85 psi/in. [1 and 23 kPa/mm] at 100 percent saturation. Note that two different types of materials can be classified as A-4. The line labeled A-4 in Figure 1 is representative of predominantly silty materials (at most 25 percent retained on the #200 sieve) with densities between about 90 and 105 lb/ft^3 [14300 and 16700 N/m^3] and CBRs between about 4 and 8. Mixtures of silt, sand, and gravel (up to 64 percent retained on the #200 sieve) can also be classified as A-4, but have densities between about 100 and 125 lb/ft^3 [15900 and 19900 N/m^3] and CBRs between about 5 and 15. The line labeled A-7-6 is more representative of this latter group.

TABLE 1 Recommended *k* Value Ranges for Various Soil Types

AASHTO Class	Description	Unified Class	Dry Density (lb/ft^3)	CBR (percent)	k value (psi/in)
Coarse-Grained Soils:					
A-1-a, well graded	gravel	GW, GP	125 - 140	60 - 80	300 - 450
A-1-a, poorly graded			120 - 130	35 - 60	300 - 400
A-1-b	coarse sand	SW	110 - 130	20 - 40	200 - 400
A-3	fine sand	SP	105 - 120	15 - 25	150 - 300
A-2 Soils (Granular Materials with High Fines):					
A-2-4, gravelly	silty gravel	GM	130 - 145	40 - 80	300 - 500
A-2-5, gravelly	silty sandy gravel				
A-2-4, sandy	silty sand	SM	120 - 135	20 - 40	300 - 400
A-2-5, sandy	silty gravelly sand				
A-2-6, gravelly	clayey gravel	GC	120 - 140	20 - 40	200 - 450
A-2-7, gravelly	clayey sandy gravel				
A-2-6, sandy	clayey sand	SC	105 - 130	10 - 20	150 - 350
A-2-7, sandy	clayey gravelly sand				
Fine-Grained Soils:					
A-4	silt	ML, OL	90 - 105	4 - 8	5 - 165 *
	silt/sand/ gravel mixture		100 - 125	5 - 15	40 - 220 *
A-5	poorly graded silt	MH	80 - 100	4 - 8	5 - 190 *
A-6	plastic clay	CL	100 - 125	5 - 15	5 - 255 *
A-7-5	moderately plastic elastic clay	CL, OL	90 - 125	4 - 15	5 - 215 *
A-7-6	highly plastic elastic clay	CH, OH	80 - 110	3 - 5	40 - 220 *

*** k value of fine-grained soils is dependent on degree of saturation. See Figure 1.**
$1\ lb/ft^3 = 159\ N/m^3$, $1\ psi/in = 0.27\ kPa/mm$

Correlation of k Value to Other Tests

Correlations were also developed in this study to estimate *k* value from California Bearing Ratio (CBR), R-value, and Dynamic Cone Penetrometer (DCP) penetration rate and are presented in Reference 2.

Backcalculation Methods

Backcalculation methods are suitable for determining *k* value for design of overlays of existing pavements, or for design of reconstructed pavements on existing alignments, or for design of similar pavements in the same general location on the same type of subgrade. An agency may also use backcalculation methods to develop correlations between nondestructive deflection testing results, and subgrade types and properties. Cut-and-fill sections are likely to yield different *k* values. No embankment or rigid layer adjustment is required for backcalculated *k* values if these characteristics are similar for the pavement being tested and the pavement being designed, but backcalculated dynamic *k* values need to be reduced by a factor of approximately two to estimate a static elastic *k* value for use in design.

AREA Methods

Equations and nomographs for backcalculation of concrete elastic moduli and subgrade *k* values for concrete and composite pavements are provided in Part III of the 1993 AASHTO Guide. This

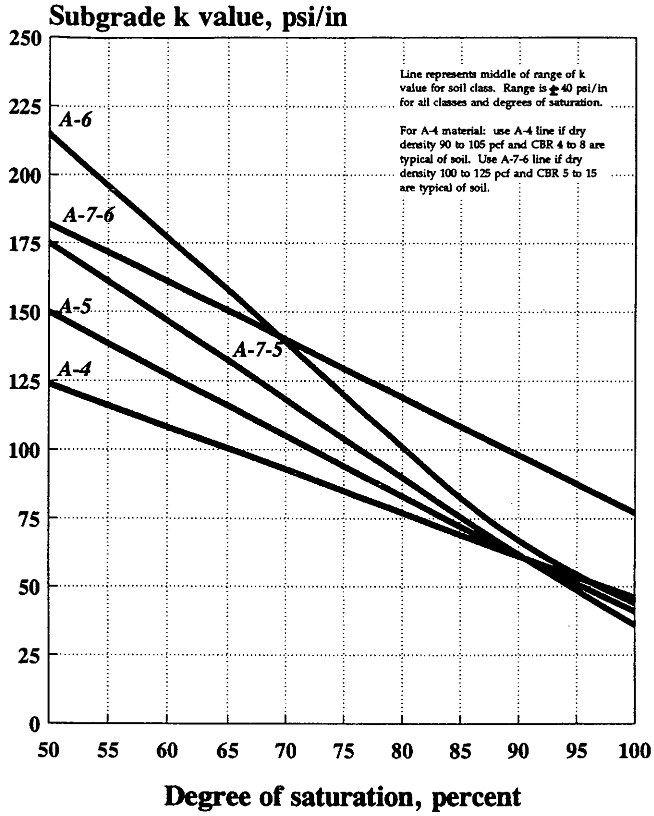

FIGURE 1 ***k* Value versus degree of saturation for cohesive soil classes.**

solution method is based on deflection of an infinite slab, and produces a dynamic elastic k value, which should be reduced by a factor of two to estimate a static elastic k value for design. This procedure is also based on a four-sensor definition of the deflection basin. For the SHRP sensor positions of 0,8,12, 18,24,36, and 60 inches [0, 203,305,457,601,914, and 1524 mm] from the load center, AREA is calculated from the following equation:

$$\text{AREA}_{\text{SHRP}} = \frac{4\,d_0 + 6\,d_8 + 5\,d_{12} + 6d_{18} + 9\,d_{24} + 18\,d_{36} + 12\,d_{60}}{d_0} \tag{2}$$

The radius of relative stiffness is calculated from the following equation:

$$\ell = \left[\frac{\ln\left(\dfrac{60 - \text{AREA}_{\text{SHRP}}}{289.708}\right)}{-\,0.698}\right]^{2.566} \tag{3}$$

$\text{AREA}_{\text{SHRP}}$ values between 35 and 50 correspond to typical l values of 25 to 55 for concrete highway pavements. (The corresponding range of AREA values according to the four-sensor definition would be 27 to 33). In theory the two AREA definitions should yield the same ℓ, but in practice the two may give different ℓ values, primarily because $\text{AREA}_{\text{SHRP}}$ includes a deflection at a much greater distance from the load.

The subgrade k value and concrete E value may be calculated from Westergaard's deflection equation and definition of radius of relative stiffness, or may be calculated from any of the sensor deflections, using the following equations:

$$k = \frac{P\,d_r^*}{d_r\,\ell^2} \tag{4}$$

$$E = \frac{12\,(1 - \mu^2)\,P\,\ell^2\,d_r^*}{d_r\,h^3} \tag{5}$$

where

P = load magnitude
d_r = measured deflection at radial distance r
h = slab thickness
μ = slab Poisson's ratio
D = slab bending stiffness:

$$D = \frac{E\,h^3}{12\,(1 - \mu^2)} \tag{6}$$

d_r^* = nondimensional deflection coefficient for radial distance r:

$$d_r^* = ae^{[-be(-c\ell)]}. \tag{7}$$

The values for the $a, b,$ and c constants in this equation are given in Table 2. Note that k and E values computed from two or more sensor deflections should not be considered independent estimates, because they are all derived from a common ℓ value which was determined from the AREA computed from all of the deflections. Note also that these equations were developed for the FWD load plate radius of 5.9 inches [150 mm], although they are not very sensitive to load size. Similar equations for the large FWD load plate have also been developed.

Solution for Any Arbitrary Sensor Arrangement

The backcalculation methods based on any given sensor arrangement are limited in application to data collected with that sensor arrangement. It is also possible to solve for l from any two deflections at any two radial distances greater than 0, because for a given load plate size, the nondimensional deflection coefficient is a function of a single parameter, the radial distance normalized to the radius of relative stiffness, as shown in Figure 2. The following equation was obtained for this curve:

$$d^* = 0.12497e^{-0.46308\left(\frac{r}{\ell}\right)^{1.55212}} \tag{8}$$

Any two deflections d_x and d_y measured at radial distances x and y (both greater than 0 and x greater than y) may be used to solve for ℓ:

$$\ell = \sqrt[1.55212]{\frac{-0.46308\,(x^{1.55212} - y^{1.55212})}{\ln\left(\dfrac{d_x}{d_y}\right)}} \tag{9}$$

Edge and Corner Solutions and Slab Size Effects

As Crovetti (*22*) has shown, Westergaard's equations for maximum edge, interior, and corner deflection may be represented as quadratic

TABLE 2 Regression Coefficients for d Versus ℓ Relationships

Radial distance (in)	a	b	c
0	0.12450	0.14707	0.07565
8	0.12323	0.46911	0.07209
12	0.12188	0.79432	0.07074
18	0.11933	1.38363	0.06909
24	0.11634	2.06115	0.06775
36	0.10960	3.62187	0.06568
60	0.09521	7.41241	0.06255

$R^2 \geq$ 99.7 percent (predicted versus actual values) for all models.
$\sigma_Y \leq 0.01$ for all models.
1 in = 25.4 mm

equations of the following form (note that a, b, and c are quadratic equation constants and a_r is the load radius):

$$\frac{d_0 D}{P\ell^2} = x_1 + x_2\left(\frac{a_r}{\ell}\right) + x_3\left(\frac{a_r}{\ell}\right)^2 \quad (10)$$

The equations for interior, edge, and corner loading become (*22*):

$$\text{Interior: } \frac{d_0 D}{P\ell^2} = 0.1253 - 0.008\left(\frac{a_r}{\ell}\right) - 0.028\left(\frac{a_r}{\ell}\right)^2 \quad (11)$$

$$\text{Edge: } \frac{d_0 D}{P\ell^2} = 0.4311 - 0.707\left(\frac{a_r}{\ell}\right) - 0.2899\left(\frac{a_r}{\ell}\right)^2 \quad (12)$$

$$\text{Corner: } \frac{d_0 D}{P\ell^2} = 1.148 - 1.50\left(\frac{a_r}{\ell}\right) - 0.6565\left(\frac{a_r}{\ell}\right)^2 \quad (13)$$

Each of these equations can be rearranged to isolate ℓ on the right side and solve for ℓ as a root of the quadratic equation:

$$\ell = \frac{b - \sqrt{b^2 - 4ac}}{2a} \quad (14)$$

where, for example, for the interior deflection equation:

$$a = 0.1253$$
$$b = -0.008a_r$$
$$c = -0.028\,a_r^2 - (d_0\,D/P)$$

This approach to determining ℓ permits backcalculation of edge and corner k values if the concrete E is assumed or is backcalculated from interior deflection basins (e.g., by the AREA method). Again, adjustment may be required for slab size.

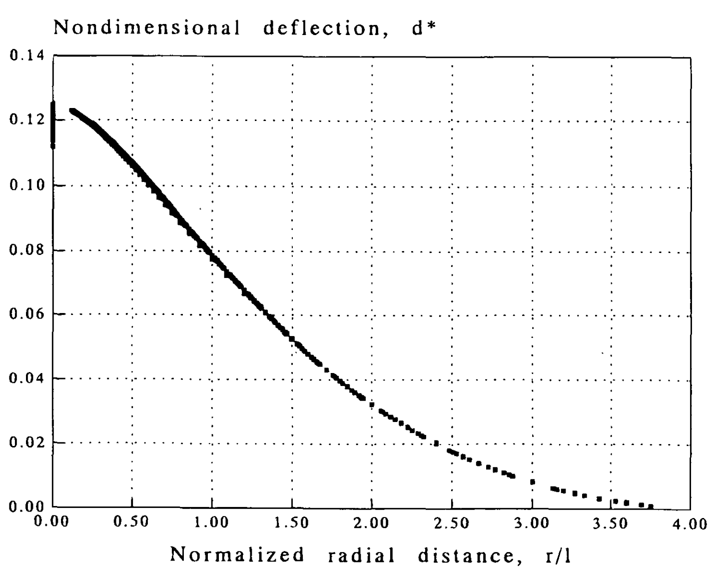

FIGURE 2 Nondimensional deflection coefficient versus normalized radial distance.

Seasonal Variation in Backcalculated k Values

The *k* value determined from backcalculation represents the *k* value for the season in which the deflection testing was conducted. An agency may wish to conduct deflection testing on selected projects in different seasons of the year to assess the seasonal variation in backcalculated *k* values for different types of subgrades.

Plate-Bearing Test Methods

The most direct method of determining *k* is by repetitive or nonrepetitive static plate-loading tests (AASHTO T 221 or T 222, ASTM D 1195 or D 1196) on a prepared section of the subgrade or embankment. Because these tests are costly and time-consuming, it is not anticipated that they will be conducted routinely. AASHTO T 221 and T 222 specify that if the pavement is to be built on an embankment, the plate-bearing tests should be conducted on a test embankment.

In the repetitive test, the elastic *k* value is determined from the ratio of load to elastic deformation (the recoverable portion of the total deformation measured). In the nonrepetitive test, the load-deformation ratio at a deformation of 0.05 in [1.25 mm] is considered to represent the elastic *k* value, according to the research by the Corps of Engineers. Note also that a 30-in. [762-mm] diameter plate should be used to determine the elastic static *k* value for use in design. Smaller-diameter plates will yield much higher *k* values which are inconsistent with slab behavior under load.

Assignment of *k* Values to Seasons

A season is defined as a period of time within a year which can be characterized by some set of climatic parameters. Among the factors which should be considered in selecting seasonal *k* values are the seasonal movement of the water table, seasonal precipitation levels, winter frost depths, number of freeze-thaw cycles, and the extent to which the subgrade will be protected from frost by embankment material. A *frozen k* may not be appropriate for winter, even in a cold climate, if the frost will not remain in a substantial thickness of the subgrade throughout the winter. If it is anticipated that a substantial depth (e.g., a few feet) of the subgrade will be frozen, a *k* value of 500 psi/in. [135 kPa/mm] would be a reasonable *frozen k.*

The seasonal variation in degree of saturation is difficult to predict, but in locations where a water table is constantly present at a depth of less than about 10 ft [3 m], it is reasonable to expect that fine-grained subgrades will remain at least 70 and 90 percent saturated, and may be completely saturated for substantial periods in the spring. The highest position of the water table, but not its annual variation, can be determined from county soil reports.

A seasonally adjusted *effective k* value may be obtained by combining the seasonal *k* values. The *effective k* value is essentially a weighted average based on some performance measure such as fatigue damage. The *effective k* value results in the same performance over the entire year that is caused by the seasonally varying *k* value. Determination of a seasonally adjusted *effective k* value within the context of any specific design procedure must be done using the performance model intrinsic to that procedure. In this study, an improved seasonal adjustment procedure was developed for the AASHTO Guide, using a proposed revised performance model calibrated to the seasonally adjusted *k* value of the AASHTO Road Test site, as described in Reference 1.

Adjustment to *k* for Fill Thickness and Rigid Layer

The nomograph shown in Figure 3 was developed for adjustment of the seasonally adjusted effective subgrade *k* value if (a) fill material will be placed above the natural subgrade, and/or (b) a rigid layer (e.g., bedrock or hard clay) is present at a depth of 10 ft [3 m] or less beneath the existing subgrade surface. Note that the rigid layer

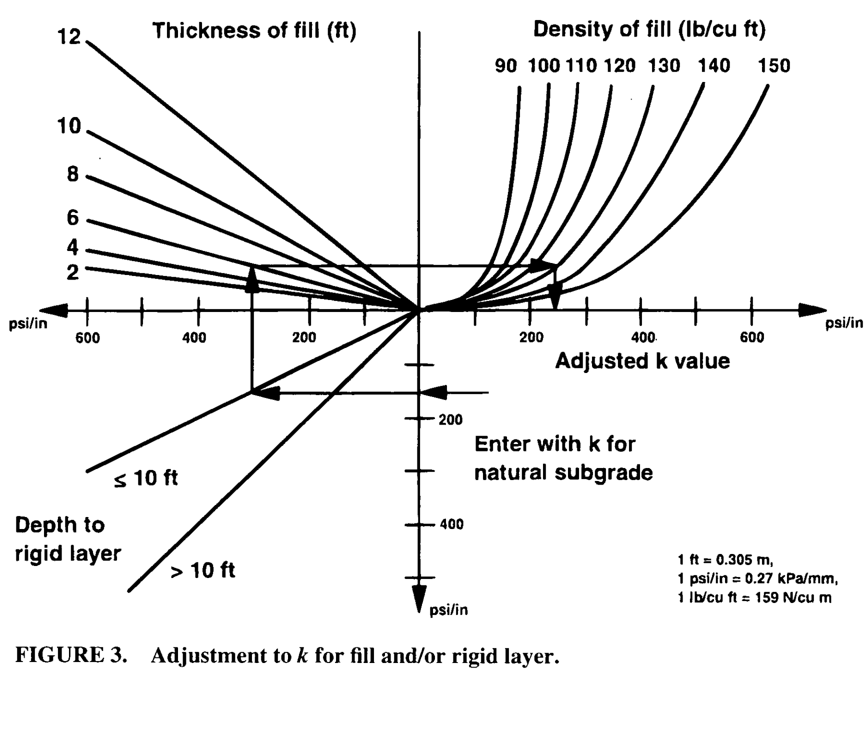

FIGURE 3. Adjustment to *k* for fill and/or rigid layer.

adjustment should only be applied if the subgrade k was determined on the basis of soil type or similar correlations. If the k value was determined from nondestructive deflection testing or from plate-bearing tests, the effect of a rigid layer is already represented in the k value obtained.

CONCLUSIONS AND RECOMMENDATIONS

This paper presents the results of research conducted to develop improved guidelines for k value selection for concrete pavement design. The research included a review of the evolution of k value concepts and methods, a review of k value results from several field studies, an examination of the AASHTO Guide's k value methods, and proposed new guidelines for selection of design k values by a variety of methods. These include correlations with soil type, soil properties, and other tests; deflection testing and backcalculation methods; and plate-bearing test methods. Guidelines for seasonal adjustment to k and adjustments for embankments and shallow, rigid layers were also developed.

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REFERENCES

1. Darter, M. I., K. T. Hall, and C.M. Kuo. Support Under Portland Cement Concrete Pavements, *NCHRP Report 372*. TRB, National Research Council, Washington, D.C., 1995.
2. Darter, M. I., K. T. Hall, and C. M. Kuo. Support Under Portland Cement Concrete Pavements, *Appendices to NCHRP Project 1-30 Final Report*. TRB, National Research Council, Washington, D.C., 1995.
3. Selvadurai, A. P. S. Elastic Analysis of Soil-Foundation Interaction. *Developments in Geotechnical Engineering*. Vol. 17, Elsevier Scientific Publishing Company, The Netherlands, 1979.
4. Peck, R. B., W. E. Hanson, and T. H. Thornburn. *Foundation Engineering*, 2nd ed. John Wiley and Sons, New York, 1974.
5. Westergaard, H. M. Computation of Stresses in Concrete Roads. *Proceedings*. HRB, National Research Council, Washington, D.C., 1925. Also published as Stresses in Concrete Pavements Computed by Theoretical Analysis. *Public Roads*. Vol. 7, No. 2, 1926.
6. Westergaard, H. M. Theory of Stresses in Road Slabs, *Proceedings, Fourth Annual Meeting of the Highway Research Board*. HRB, National Research Council, Washington, D.C., 1925.
7. Teller, L. W., and E. C. Sutherland. The Structural Design of Concrete Pavements, Part 5, An Experimental Study of the Westergaard Analysis of Stress Conditions in Concrete Pavements of Uniform Thickness. *Public Roads*. Vol. 23, No. 8, 1943.
8. Sale, J. P. Rigid Pavement Design for Airfields. *Proceedings, First International Conference on Concrete Pavement Design*, Purdue University, West Lafayette, Indiana, 1977.
9. Phillippe, R. R. Field Bearing Tests Applied to Pavement Design. *Symposium on Load Tests of Bearing Capacity of Soils*, American Society for Testing and Materials, Special Technical Publication No. 79, 1947.
10. Middelbrooks, T. A., and G. E. Bertram. Soil Tests for Design of Runway Pavements. *Proceedings*, HRB, National Research Council, Washington, D.C., 1942.
11. Ahlvin, R. G. Origin of Developments for Structural Design of Pavements. *Technical Report GL-91-26*, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, 1991.
12. Highway Research Board. The AASHO Road Test, Report 5, Pavement Research, *Special Report 61E*, 1962.
13. Langsner, G., T. S. Huff, and W. J. Liddle. Use of Road Test Findings by AASHO Design Committee, from The AASHO Road Test, Proceedings of a Conference Held May 16–18, 1962, St. Louis, Mo. *HRB Special Report No. 73*, National Research Council, Washington, D.C., 1962.
14. U.S. Army Corps of Engineers. Results of Modulus of Subgrade Reaction Determination at the AASHO Road Test Site by Means of Pavement Volumetric Displacement Test. Ohio River Division Laboratories, April 1962.
15. Vesic, A. S., and S. K. Saxena. Analysis of Structural Behavior of AASHO Road Test Rigid Pavements, *NCHRP Report No. 97*, HRB, National Research Council, Washington, D.C., 1970.
16. Childs, L. D. Cement-Treated Subbases for Concrete Pavements. *Record No. 189*, HRB, National Research Council, Washington, D.C., 1967.
17. American Association of State Highway Officials. *Interim Guide for Design of Pavement Structures*. Washington, D.C., 1972.
18. Darter, M. I. Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume 1—Development of Design Procedures. *Report No. FHWA-RD-77-111* University of Illinois at Urbana-Champaign, 1977.
19. Thompson, M. R., and Q. L. Robnett. Resilient Properties of Subgrade Soils. *Illinois Cooperative Highway Research Program, Report No. UILU-ENG-76-2009*, University of Illinois and Illinois Department of Transportation, 1976.
20. American Association of State Highway and Transportation Officials, *Guide for Design of Pavement Structures*, Washington, D.C., 1993.
21. American Association of State Highway and Transportation Officials, *Guide for Design of Pavement Structures*, Washington, D.C., 1986.
22. Crovetti, J. A. Evaluation of Jointed Concrete Pavement Systems Incorporating Open-Graded Subbases. Ph.D. thesis. University of Illinois at Urbana-Champaign, 1994.

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