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Selection of Design Strengths of Untreated Soil Subgrades and Subgrades Treated with Cement and Hydrated Lime

by

TOMMY C. HOPKINS
Head of Geotechnology

David Q. Hunsucker
Head of Materials

and

Tony L. Beckham
Research Geologist

Kentucky Transportation Center
College of Engineering
University of Kentucky

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Tommy C. Hopkins, **David Q. Hunsucker**, and **Tony L. Beckham**

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16. Abstract
Selection of design strengths of soil subgrades and subgrades treated with cement or hydrated lime is a problem in pavement design analysis and construction because a variety of different types of soils may exist in a highway corridor and a wide range of different strengths may exist after the soils are compacted to form the pavement subgrade. The selected subgrade strength will largely affect the pavement thickness obtained from the design analysis. the future pavement performances. and the overall bearing capacities of the subgrade during construction and the pavement structure after construction. In developing the proposed selection scheme, a newly developed mathematical model, based on limit equilibrium, is used. Relationships between undrained shear strength (and CBR) and tire contact stresses are developed for factors of safety 1.0 and 1.5. The minimum subgrade strength required to sustain anticipated construction tire contact stresses to avoid bearing capacity failures of the subgrade and partially constructed pavements during construction is determined. Also, a criterion is proposed for determining when subgrade stabilization is needed. Methods of selecting the design subgrade strength are examined. A previously published method, based on a least cost analysis, appears to be an appropriate approach as shown by analysis of a case study involving pavement failures during construction. Two case studies show that soaked laboratory strengths appear to be fairly representative of long-term field subgrade strengths. Hence, using soaked laboratory strengths and least cost analysis appears to be reasonable means for selecting the design strength of subgrades for pavement analysis. When chemical stabilization is used, it is suggested that the net strength gain obtained at the end of a 7-day curing period may be used in the pavement design analysis. To avoid failures of chemically stabilized layers, relationships between thicknesses of chemically treated layers and the CBR values of the untreated subgrade for a factor safety of 1.5 are presented. Layer coefficients (a_3), based on 7-day strengths, are also presented for hydrated line-and cement-treated subgrades.

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Subgrade; stabilization; Design; Strengths; Soils; Cement; Hydrated Lime

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in.	inches	25.40000	millimetres	mm	mm	millimetres	0.03937	inches	in.
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mi	miles	1.60934	kilometres	km	km	kilometres	0.62137	miles	mi
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		VOLUME					**VOLUME**		
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yd^3	cubic yards	0.76455	metres cubed	m^3	m^3	metres cubed	1.30795	cubic yards	yd^3
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oz	ounces	28.34952	grams	g	g	grams	0.03527	ounces	oz
lb	pounds	0.45359	kilograms	kg	kg	kilograms	2.20462	pounds	lb
T	short tons (2000 lb)	0.90718	megagrams	Mg	Mg	megagrams	1.10231	short tons (2000 lb)	T
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lbf	pound-force	4.44822	newtons	N	N	newtons	0.22481	pound-force	lbf
psi	pound-force per square inch	6.89476	kilopascal	kPa	kPa	kilopascal	0.14504	pound-force per square inch	psi
		ILLUMINATION					**ILLUMINATION**		
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fl	foot-Lamberts	3.42583	candela/m^2	cd/m^2	cd/m^2	candela/m^2	0.29190	foot-Lamberts	fl
		TEMPERATURE (exact)					**TEMPERATURE (exact)**		
°F	Fahrenheit temperature	5(F-32)/9	Celsius temperature	°C	°C	Celsius temperature	1.8C + 32	Fahrenheit temperature	°F

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

Rev 12/93

INTRODUCTION

Along any highway corridor, before construction, a variety of different soil horizons and soil types are normally encountered and a wide range of bearing strengths may exist when the different types of soils are used to construct pavement subgrades. To avoid bearing capacity failures during construction of the subgrade and placement of the pavement layers, a certain minimum subgrade strength must exist to sustain construction traffic. Hence, the design strength selected for pavement analysis should consider the issue of pavement construction. The method of selecting the design strength is complicated when different subgrade strengths exist along the route to be paved. Additionally, when the design analysis is based on a selected laboratory strength, the question arises whether the laboratory strength is representative of the long-term field strengths existing after paving.

When the actual subgrade strength is lower than the minimum strength required to sustain construction traffic, then it may be necessary to stabilize the subgrade soils with chemical admixtures, such as cement or hydrated lime, or by other means. When chemical stabilization is used, then a design strength of the treated layer as well as a design strength of the untreated layer located below the treated layer must be selected for the design analysis. If the improved strength created by chemical stabilization is ignored, then the pavement thickness obtained from the design analysis may be overconservative. Moreover, the long-term strength gain may be much larger than the subgrade strength existing at the time of construction. Consequently, the issue that arises is whether the stabilized layer should be treated merely as a working platform, with no allowance made in the pavement design analysis for the net strength gain obtained from stabilization, or whether the stabilized layer should be considered an integral part of the pavement structure with the total, or a portion, of the net strength gain considered in the analysis. To examine and analyze the different issues posed, a pavement bearing capacity model (1, 2), formulated on the basis of limit equilibrium, is used. The selection scheme makes use of an approach described by Yoder (3).

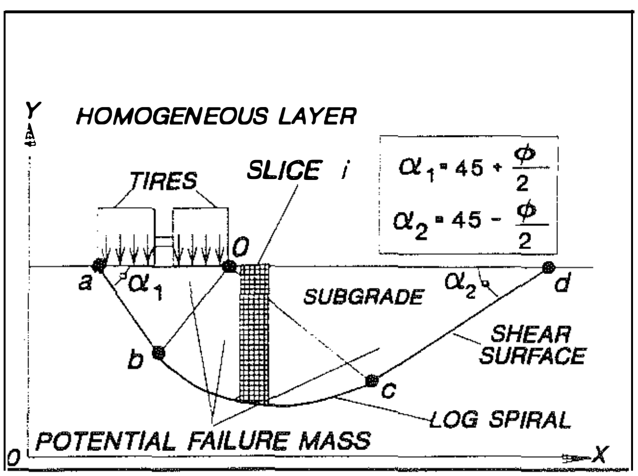

Figure 1. ***Bearing Capacity Analysis of Homogeneous Layer***

BEARING CAPACITY MODEL

The mathematical bearing capacity model used herein is based on limit equilibrium and may be used to calculate the factor of safety against failure. The problems to be analyzed are visualized in Figures 1 and 2. Theoretical considerations and mathematical derivations of limit equilibrium equations for analyzing the ultimate bearing capacity of soil subgrades and partially completed asphaltic pavements, and the extension of those equations to the analyses of asphaltic pavements composed of multiple layers, have been presented elsewhere (1) and are beyond the scope of this paper. Each layer of material--subgrade, base, and asphaltic layers--in the pavement structure are described in the model using shear strength parameters, the angle of internal friction, ϕ, and cohesion, c, and unit weights. Problems involving total stress and effective stress analyses may be solved.

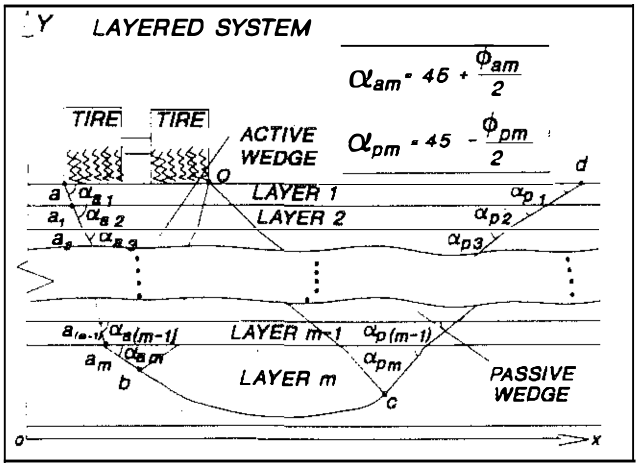

Figure 2. ***Bearing Capacity Analysis of a Layered System***

The assumed theoretical failure mass consists of three zones--active and passive wedges connected by a central wedge whose shear surface is a logarithmic spiral curve. The shear surface assumed in the model analysis for a homogeneous layer of material consists of a lower boundary, identified in Figure 1, as **abcd**. This surface consists of two straight lines, **ab** and **cd**. The portion of the shear surface shown as line **ab** is inclined at an entry angle, α_1. Line **cd** is inclined at an exit angle, α_2. The angles, α_1, and α_2, are defined in Figure 1. The shear surface, **bc**, is determined from the properties of a logarithmic spiral. For a layered system, the shear surface is visualized as shown in Figure.2.

The approach is a generalized method of slices and is an adaptation of a slope stability method developed by Hopkins (2). Vertical, horizontal, and moment equilibrium equations are considered for each slice. In the solution of these equations, the factor of safety appears on both sides of the final equation. Iteration and numerical techniques are used to solve for the factor of safety (1). To facilitate the use of the approach, all algorithms were programmed for the mainframe computer (3090) at the University of Kentucky.

Because the shear strength of asphaltic materials varies with temperature and temperatures within the asphaltic layer vary with depth, the shear strength varies with depth. To account for this variation in the model analysis, unconsolidated-undrained triaxial compression tests were performed on asphaltic core specimens that were assumed to be representative of typical flexible pavements. These tests were conducted for temperatures ranging from 25 to 60 degrees, Centigrade. As shown in Figures 3 and 4, the shear strength parameter, ϕ, increases and the shear strength parameter, c, decreases as the temperature increases. In the analyses of problems involving asphaltic materials, a temperature-depth model (4) is used to estimate the temperature at any depth within a given asphaltic layer. Different surface temperatures and average air temperatures may be used in the analysis. Based on a calculated temperature and the correlations of ϕ and c with temperature, the shear strengths at a given depth within an asphaltic pavement may be determined. and ϕ-

and c-values are assigned to each layer. Total stress parameters, ϕ and c, of crushed stone (limestone) bases were assumed to be 43 degrees and zero, respectively (1).

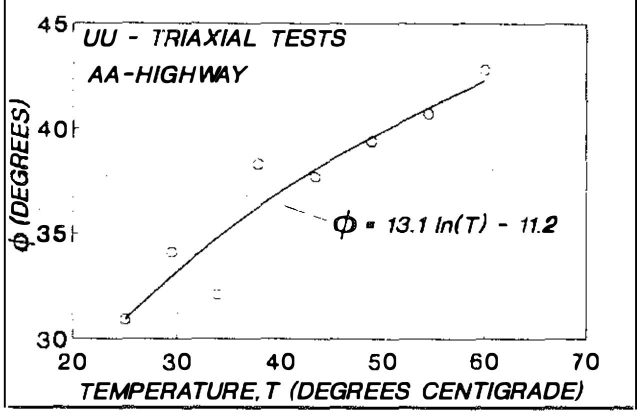

Figure 3. ***Influence of Temperature on Shear Strength Parameter, ϕ***

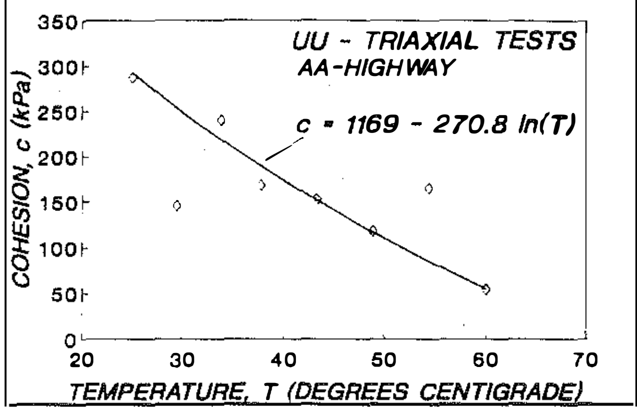

Figure 4. ***Influence of Temperature on Shear Strength Parameter, c***

MINIMUM SUBGRADE STRENGTH

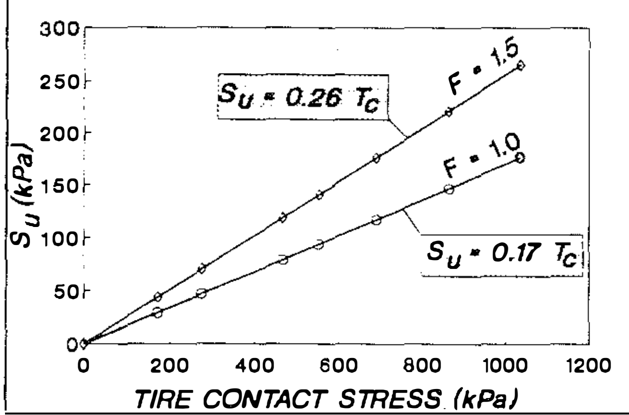

Figure 5. ***Undrained Shear Strength as a Function of Tire Contact Ground Stress***

In the analyses, each asphaltic pavement was divided into small finite (2.54 cm) layersTo avoid bearing capacity failures under construction traffic and to assure the efficient construction of the pavement, the subgrade must possess some minimum strength. The minimum strength required to maintain stability is a function of the tire contact ground stress. As the contact stress increases, the required strength increases. This situation, as visualized in Figure 1, was analyzed using the bearing capacity model described above. Dual-wheel tires and a range of tire contact stresses (uniformly distributed) and undrained shear strengths of the soil subgrade were assumed. The relationships of undrained shear strength and tire contact ground stresses corresponding to factors of safety of 1.0 (incipient failure) and 1.5 (an assumed stable condition) were developed. For a selected tire contact stress and undrained shear strength, the factor of safety was computed. Relationships developed in this manner are shown in Figure 5. Hence, if the anticipated tire contact stress of construction traffic is known, then the required strengths to maintain an incipient failure condition (F=1.0) or an assumed stable condition (F=1.5) may be determined. For example, if the tire contact stress is 552 kPa, then the undrained shear strength for an incipient failure is 94 kPa and about

144 kPa for an assumed stable condition.

Relationships between bearing ratios (ASTM D 1883) and tire contact stresses may also be developed using a relationship between bearing ratio and undrained shear strength developed by Hopkins (1, 5), or

$$CBR = 0.0649 S_u^{1.014} \quad (kPa) . \tag{1}$$

For a tire contact stress, Tc, of 552 kPa, the required bearing ratio for incipient failure (S_u = 0.17Tc) is about 6.5 and about 10 (S_u = 0.26Tc) for an assumed stable condition.

Required minimum dynamic modulus of elasticity required to maintain incipient failure and a stable condition may be determined using the relationship developed by Heukelom and Foster (6). Re-analyses of those data yields the following expression

$$E_s = 17{,}914 CBR^{0.874} \quad (kPa) . \tag{2}$$

Inserting the CBR values of 6.5 and 10, which correspond to factors of safety of 1 and 1.5, respectively, into Equation 2, the dynamic modulus of elasticity required to maintain an incipient failure state is about 91,979 kPa and for an assumed stable condition the required modulus is 134,031 kPa.

SELECTION OF UNTREATED SUBGRADE DESIGN STRENGTH

Different philosophies exist concerning the method of selecting the subgrade design bearing ratio (or strength parameters from other types of tests). Some of the approaches include using

- the lowest value,
- an average value,
- statistical methods of estimating the average values, or
- a value based on a least-cost analysis.

When the lowest value of bearing ratio of a data set is selected, the pavement may be over designed. If the average value of the data set is selected, approximately one half of the pavement (of a selected route) may be over designed while one half may be under designed (3). Another approach embraces the normal distribution curve and reliability concepts. This concept involves upper and lower limits for the selected confidence interval.

Another approach, based on a least-cost design, has been proposed by Yoder (3) who presented a series of curves that relate percentile test values to soil variability (measured by the coefficient of variance of the test data set), traffic (EAL), and unit cost of the pavement. Unit cost of maintaining a highway is expressed in terms of a cost ratio (CR), or unit maintenance cost divided by the unit initial construction cost. When detailed information is lacking, Yoder suggests using the bearing ratio

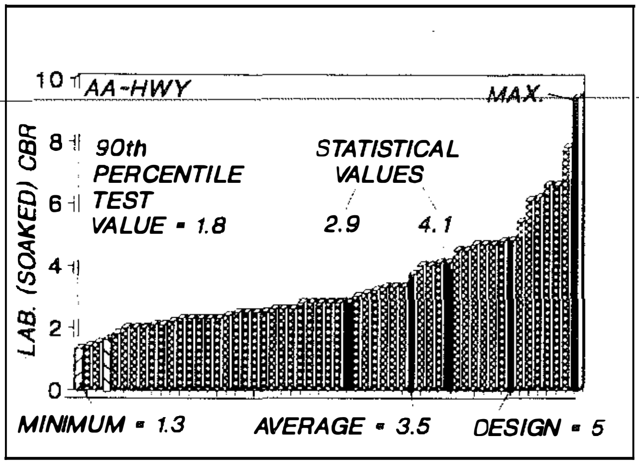

Figure 6. ***Soaked Laboratory CBR Values of Corridor Soils***

occurring at the 80th or 90th percentile test value to obtain an optimum design.

To test and compare the results of the different approaches, an analysis of soaked laboratory CBR values of two adjacent sections of a highway route located in Kentucky was performed. Total length of the two sections was about 12.2 km. The planned pavement structure consisted of 26.7 cm of asphaltic pavement and 10.2 cm of dense graded aggregate. The design CBR and equivalent single-axle load (ESAL) were, reportedly, 5 and 4 million, respectively. During construction the partially completed pavements failed at numerous locations along the two highway sections.

Soaked laboratory values of CBR of corridor soil samples obtained prior to construction are shown graphically in Figure 6. The lowest CBR value of the data set (56 tests) is 1.3 and the average value is 3.4. Based on the assumption that the CBR data set is normally distributed, lower and upper-bound CBR values for a 95 percent confidence interval are 2.9 and 4.1, respectively. Percentile test value (as proposed by Yoder, 3) as a function of the soaked laboratory CBR is shown in Figure 7. Cost ratio for the two highway routes was not available. In this case, as suggested by Yoder, the value of CBR occurring at the 90th or 80th percentile test value may be used. At the 95th, 90th, and 80th percentile test values, the CBR values are 1.4, 1.8, and 2.1, respectively.

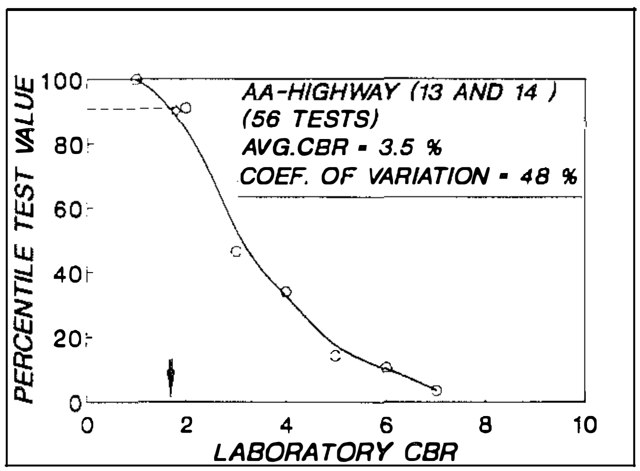

Figure 7. ***Percentile Test Value as a Function of Soaked Laboratory CBR -- AA Route***

To compare the different CBR selection approaches, factors of safety of the planned pavement section were computed using the bearing capacity model described above. Surface and air temperatures at the time of the failures were, reportedly, 60 and 26.7 degrees Centigrade, respectively. A temperature-depth (4) model (1) was used to estimate the temperatures at each midpoint of each 2.54-cm asphaltic layer. Using these estimated temperatures, ϕ-and c-values for each layer were estimated from the curves shown in Figures 3 and 4. Values of CBR were converted to undrained shear strengths using the relationship given by Equation 1. A uniformly distributed, tire contact stress of

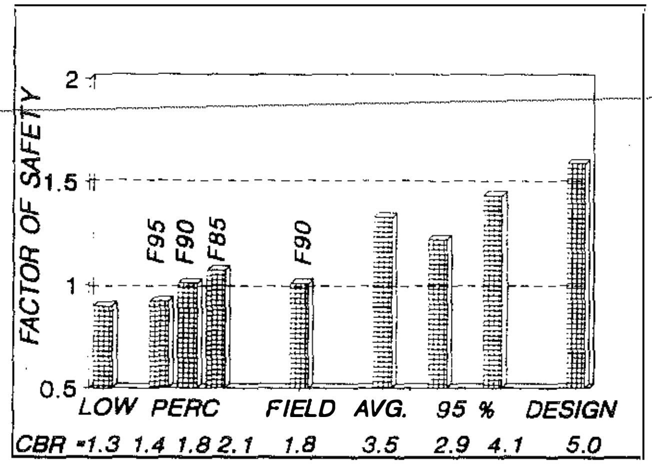

Figure 8. ***Factors of Safety for Different CBR Design Values***

552 kPa (dual wheels) was assumed in the analysis.

Factors of safety, based on different CBR design assumptions, are compared in Figure 8. When the average CBR value of the data set is assumed to be the correct value, a factor of safety of 1.33 is obtained. If it is assumed that the CBR (equal to 5) used in the original design is correct, then a factor of safety of about 1.59 is obtained. If the CBR values obtained from reliability theory at a confidence interval of 95 percent are used, then factors of safety of 1.22 and 1.43 are obtained. This approach yields an unsafe design. In each of these three cases, the factor of safety is much greater then one. However, since the pavements failed, the factor of safety should be near one. Based on values of CBR (1.4, 1.8, and 2.1) corresponding to percentile test values of 95, 90, and 85, factors of safety of 0.91, 1.00 and 1.07 are obtained, respectively. The CBR value (1.8) corresponding to the 90th percentile test value, which yields a factor of safety of one, appears to be an appropriate design choice.

The problem of selecting a design CBR value may be illustrated in another manner using model analysis to determine the required thickness for a given design factor of safety. Based on an analyses (1) of some 237 asphaltic pavement sections of the AASHO Road Test (7), an approximate relationship, corresponding to a serviceability index of 2.5, between factor of safety and (weighted) equivalent single-axle load (ESAL) was developed, or

$$F = (0.095)\, Ln(ESAL) - 0.005 \qquad (3)$$

Inserting the design ESAL of 4 million into Equation 3, the design factor of safety is 1.44. The total pavement thickness corresponding to a selected subgrade CBR value and design factor of safety was obtained from the bearing capacity model by iteration. The thickness of the pavement is varied until the factor of safety is equal to the selected design factor of safety obtained from Equation 3. The thickness of the DGA (10.2 cm) was held constant so that the various thicknesses (based on different assumed CBR design values) could be compared to the thickness of the pavement sections after overlays were constructed.

Thicknesses obtained from the various analyses, based on different assumed design values of CBR and corresponding to a factor of safety of 1.44, are shown in Figure 9. If the lowest value of CBR (1.3) is assumed to be the correct design value, than a total thickness of 53.1 cm is required. This thickness is some 16.3 cm larger than the planned thickness. If the average value of CBR (3.4) is used, a thickness of 40.1 cm is obtained. The average CBR value yields a thickness that is only 3.3 cm greater

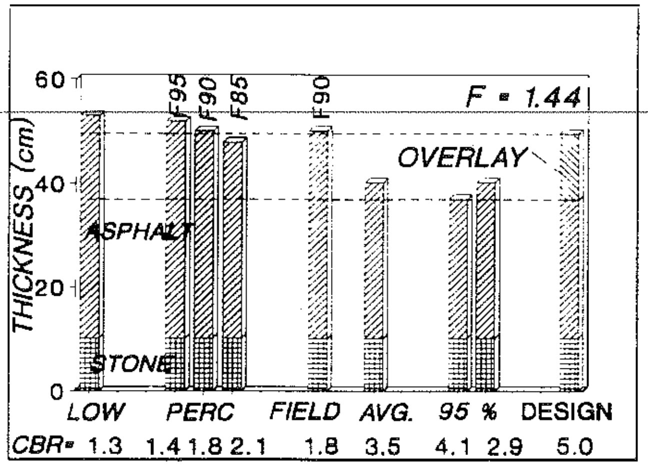

Figure 9. Flexible Pavement Thicknesses obtained for Diggerent CBR Design Values

than the original planned thickness. A value of 3.4 corresponds to a percentile test value of only about 40 to 50 (Figure 7). Accordingly, numerous portions (spot-to-spot) of the pavement would require future maintenance. Required thicknesses obtained when the upper and lower bound values of CBR obtained from reliability theory are only 0.25 cm to 2 cm, respectively, greater than the original design section, which failed. If the CBR value (1.8) occurring at the 90th percentile is assumed to be the correct design value, then a thickness of 50 cm is obtained -- a thickness that is some 13.2 cm greater than the original planned section. As shown in Figure 6, values of CBR less than 1.8 occur at only about ten percent of the sampling sites.

Approximately 50 percent of the total length of the highway sections was repaired using an overlay thickness of about 12.7 cm. Total thickness of the pavement at those locations after overlaying was about 49.5 cm -- a value that is nearly identical to the thickness (50 cm) obtained when the value of CBR at the 90th percentile test value is used. The method proposed by Yoder appears to be a reasonable approach to the problem of selecting a design subgrade strength as strongly indicated by this case history analyses. Using the 1981 Kentucky design curves (8), a thickness of 47 cm is obtained. Proper selection of a subgrade design value of CBR (or other strength parameters) is vital to avoid construction failures and to insure good pavement performance.

EFFECT OF MOISTURE ON SOIL SUBGRADES

Subgrades built of clayey soils and compacted according to standard compaction specifications generally possess large bearing strengths immediately after compaction. However, there is no assurance that the subgrade soils will retain their original strengths. Bearing strengths of the completed subgrade depend on the long-term density and moisture. The original compactive state of clayey soils is very likely to change with increasing time and load applications. Clayey soils tend to absorb water and increase in volume. With an increase in volume, the shear strength available to resist failure decreases. The differences in bearing strengths of compacted soils in soaked and unsoaked states may readily be illustrated by analyzing the results of some 727 laboratory CBR tests (1). Each specimen of the group of tests was penetrated before soaking and after soaking. Before soaking, and immediately after compaction, bearing ratios of 95 percent of the specimens were greater than 6. After soaking, the bearing ratio of only 54 percent of the specimens exceeded 6. As shown by the theoretical analysis, bearing capacity failures may occur in the subgrade when the CBR is less than about 6.5 and the tire contact stress of construction vehicles is about 552 kPa.

Field observations also show that bearing strengths of clayey subgrades may decrease significantly after construction (1, 9). Field CBR tests were performed on a clayey subgrade, at a highway construction site in Kentucky, immediately after compaction. Values of CBR ranged from about 20 to 40. A second series of field CBR tests were performed after the subgrade has been exposed to a winter season. Values ranged from about 1 to 4 -- a dramatic decrease in bearing strengths. Hence, as noted by Yoder and Watczik (10), pavement design analysis should be based on the characteristics of the completed subgrade. In areas where water may infiltrate the subgrade from surface and subsurface waters, the design should be based on the strength of the soaked condition of the completed subgrade. The strength may be very large if field tests are performed on the subgrade immediately after compaction. When sufficient time has elapsed between the completion of subgrade compaction and paving, and the subgrade has been exposed to wetting conditions, then using the field strengths of the soaked subgrade may represent a valid approach. However, when the pavement is placed immediately after compaction, then the field strengths may be too large to assume for design purposes. Moreover, many projects are scheduled years in advance and it may not be convenient, or the opportunity may not be available, to perform field tests in a soaked condition before the design analysis. Hence, the design analysis should be based on the soaked strengths of laboratory tests. When the design is based on laboratory tests, then a question arises concerning the similarity of field and laboratory strengths.

COMPARISON OF FIELD AND LABORATORY SUBGRADE STRENGTHS

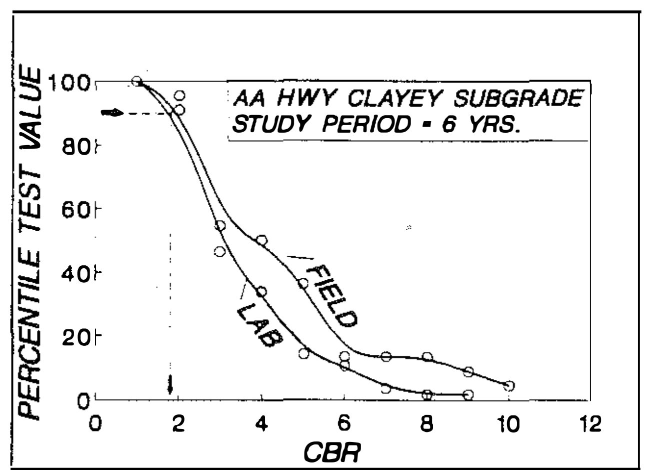

Figure 10. ***Field and Laboratory Percentile Test Values as a Function of CBR -- AA Route***

To determine the similarity of laboratory and long-term field strengths, two highway routes were selected where a number of laboratory (soaked condition) bearing ratios had been performed on the corridor soils. Field bearing ratio tests were performed through core holes on top of the untreated subgrades over a period of 5.5 years. Testing did not commence until the pavement had been placed and at least one winter and spring season had passed. Because it was not certain where particular corridor soils would be placed in the subgrades of each route, curves of percentile test value as a function of laboratory and field bearing ratios were developed and compared. Soils of the first route (identified as AA route) are residual soils of the Kope Geological Formation (clayey shales). Classification of these soils ranged from A-6 to A-7 and CL to CH. A comparison of percentile test values as a function of laboratory and field values of CBR of this route is presented in Figure 10. Average values of laboratory and field CBR were 3.5 (56 tests) and 4.1 (22 tests), respectively. At the 90th and 80th percentile test values, the laboratory strength is about 90 percent of the field CBR. Between 60 to about ten percent, the laboratory CBR value was about 90 to 70 per cent of the field value. Hence, there was reasonable agreement between the laboratory and field percentile test curves.

Comparison of laboratory and field values of CBR of the second highway route (KY Route 11) is shown in Figure 11. Classifications of the soils on this route ranged from A-4 to A-7 and ML-CL to CL. From percentile test values of 100 to 90, the field and laboratory values are essentially the same. Between percentile test values of 90 and 10, the field value is some 100 to 75 percent of the laboratory CBR. At the 90th and 80th percentile test values, the field and laboratory values of CBR are nearly identical. Based on these comparisons, laboratory CBR values appear to provide a reasonable representation of the field CBR values of the completed subgrade after sufficient time has elapsed for soaking conditions to develop. Consequently, design strength of the untreated subgrade may be based on the soaked laboratory CBR test.

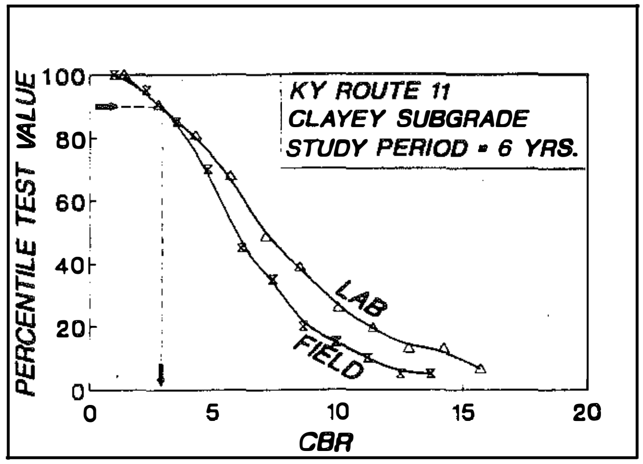

Figure 11. Field and Laboratory Percentile Test Values as a Function of CBR -- Ky Route 11

STABILIZATION REQUIREMENTS

As shown by the theoretical analyses, Figure 5, bearing capacity failures may occur in the subgrade during construction when the CBR value is below about 6.5 and the tire contact stress is 552 kPa. Consequently, to avoid bearing capacity failure of the completed subgrade during construction, to provide a firm platform for paving, and to insure efficient construction, it may be necessary to stabilize the soils using chemical admixtures, such as cement or hydrated lime. Considering that a variety of strengths may exist in the completed subgrade, subgrade stabilization should be considered when the CBR value occurring at about the 80th or 90th percentile test value is below about 6.5-10, although the value of the design CBR may be selected at some percentile test value that is smaller than the 80th or 90th percentile test value if cost ratios are used. By using the CBR value at the 80th or 90th percentile test value, adequate subgrade stability should be available to maintain efficient construction throughout.

DESIGN STRENGTHS OF CHEMICALLY STABILIZED SUBGRADES

Selection of the design strength of subgrades treated with cement or hydrated lime will be controlled by the time allowed for curing. At the end of the curing period, sufficient strength must exist to withstand construction traffic loadings and to avoid bearing capacity failures. If the strength existing at the end of a selected curing period can be estimated with some degree of confidence, then that strength may be used in the pavement design analysis. For example, in Kentucky, treated subgrades are allowed to cure for seven days and substantial strength gains occur in the treated layer during the curing period. This specified curing period appears to be acceptable to sponsoring agencies and contractors. Optimum percentages, as determined from testing (1), of cement or hydrated lime are used to treat the subgrades.

General guidelines for selecting the design strengths of hydrated lime- and cement-treated subgrades were developed on the basis of strengths of the treated layers existing at the end of a seven-day curing period. Several highway routes were selected and core specimens of the hydrated lime- or cemented- treated subgrades were obtained at about the end of the seven-day period. Numerous types of soils, ranging from A-4 to A-7, were used to construct the subgrades at the selected routes. Unconfined compression tests were performed on the core specimens. Bag samples of the untreated soil subgrades were obtained at several, equally spaced, locations along each route of the completed subgrade before treatment. Specimens of these soils were remolded to optimum moisture content and 95 percent of maximum dry density (AASHTO T 99). Optimum percentages of chemical admixture were used in remolding the specimens. After aging the sealed specimens for 7 days, unconfined compression tests were performed.

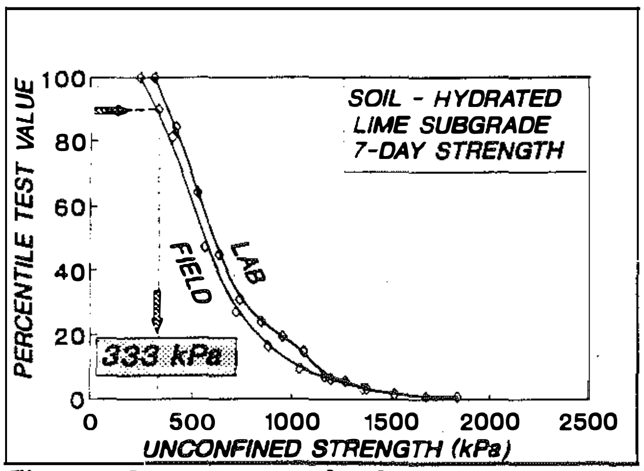

Figure 12. ***Field and Laboratory Percentile Test Values as a Function of CBR -- Soil-Hydrated Lime Subgrades***

Field and laboratory unconfined compressive strengths of the cement- and hydrated lime- treated specimens, as a function of percentile test values, are shown in Figures 12 and 13. Unconfined compressive strengths of the field, hydrated lime-treated specimens were about 85 to 90 percent of the unconfined compressive strengths of the laboratory specimens for percentile test values ranging from 100 to about 10. This indicated that the hydrated lime and clayey soils were mixed very well in the field and also indicated that the hydrated lime penetrated the clayey clods. Unconfined compressive strengths of the field, cement-soil core specimens ranged from about 75 to 50 percent of laboratory unconfined compressive strengths for percentile test values ranging from 100 to 0, respectively. Assuming that the 90th percentile test value is a reasonable working level, unconfined compressive strengths

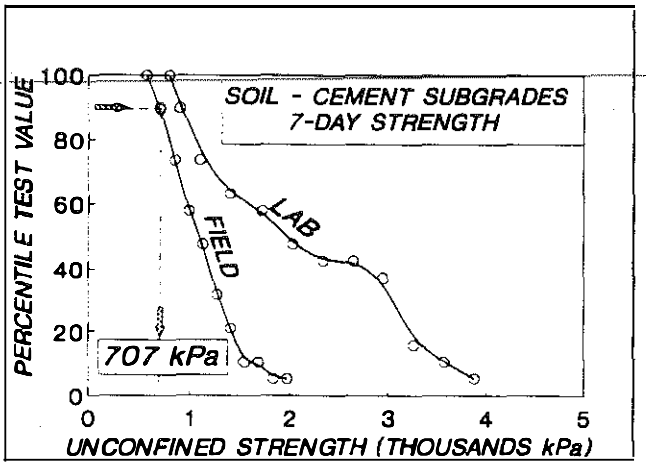

Figure 13. ***Field and Laboratory Percentile Test Values as a Function of CBR -- Soil-Cement Subgrades***

of about 333 kPa and 707 kPa (undrained shear strengths, S_u, of 167 and 354 kPa, respectively) appear to be reasonable values to assume in the design of hydrated lime- and cement-treated soil subgrades, respectively. Corresponding values of bearing ratio, estimated from Equation 1, are about 11.6 and 24.9, respectively. Estimated values of dynamic modulus of elasticity (Equation 2) are about 152,590 and 297,489 kPa, respectively.

APPROXIMATE REQUIRED THICKNESSES OF TREATED SUBGRADES

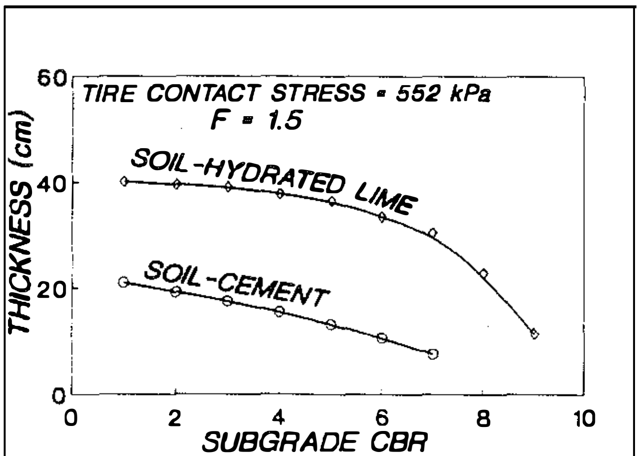

Figure 14. ***Thicknesses of Treated Subgrades as a Function of Untreated Subgrade CBR Values***

By using the seven-day strengths, some portion of the total strength gain of the hydrated lime- or cement- treated subgrade may be considered in the pavement design analysis. However, use of these strengths does not, necessarily, assure that bearing capacity failure of the treated layer will not occur. The bearing capacity of the chemically treated layer is a function of the thickness of the treated layer and the bearing strength of the untreated layer located below the treated layer. To estimate thicknesses required to maintain an assumed stable condition (say, F=1.5), bearing capacity analyses were performed using the model described above. In the analysis of this two-layered problem, the tire contact stress was assumed to be 552 kPa, the unconfined compressive strengths occurring at the 90th percentile test value (Figure 10) were assumed for the treated layers, and the bearing ratio of the untreated layer was ranged from 1 to 9 (or unconfined strength ranging from 15 kPa to 130 kPa). Thicknesses, as shown in Figure 14, of hydrated lime-treated subgrades required to maintain a factor of safety of 1.5 range from about 40 cm to 11 cm for values of CBR of the untreated layer ranging from 1 to 9. For cement-treated subgrades and for CBR values ranging from 1 to 7, required thicknesses range from about 21 cm to 7.6 cm.

SIGNIFICANCE OF TREATED SUBGRADES TO PAVEMENT STRUCTURE

Use of hydrated lime or cement not only increases the shear strength of a soil subgrade but it also improves the overall bearing capacity of a flexible pavement. The value of stabilizing subgrades with hydrated lime or cement may readily be demonstrated by an example design problem. Assume, for instance, that a flexible pavement is to be designed for an equivalent single-axle load (ESAL) of 18 million and the subgrade soils are the same as those used at the 1960 AASHO Road Test (7). Percentile test values as a function of field CBR values (from the trenching program --Table 2 of Reference 7) for spring and summer seasons were determined. At the 90th percentile test value, bearing ratios, corresponding to spring and summer are 2.5 and 3.0, respectively. Average CBR values are 3.6 and 5.3, respectively. The design is to consist of one-third asphaltic concrete and two-thirds crushed stone. Coefficients, a_1 and a_2, are 0.44 and 0.14, respectively, terminal serviceability index is 2.5, and tire unit contact stress is 466 kPa. The soil support value is 3.

The structural number, SN, is 5.6. Total pavement thickness is 59.2 cm -- 19.8 cm of asphaltic concrete and 39.4 cm of crushed stone base. Using the CBR of the untreated subgrade occurring at the 90th percentile (2.5 or an undrained shear strength of 36.7 kPa), model analysis yields a factor of safety of 1.29. If the average CBR (3.6) is used, then a factor of safety of 1.55 is obtained. From Equation 3, the estimated ESAL is only 800,000 -- a value that is much lower than the design ESAL of 18 million. If the average CBR of 3.6 is used in the analysis, then the estimated ESAL value is about 16 million, which is near the design value of 18 million. However, the percentile test value, is only about 40. Hence, if the value of 3.6 is used, much maintenance would be required.

Since the CBR value occurring at the 90th percentile test value (as well as the average CBR occurring at the 40th percentile test value) is below a CBR of 6.5, stabilization of the soil subgrades should be considered to avoid bearing capacity failures. Moreover, difficulties may be encountered during placement of the first lift of crushed stone base if treatment was not performed. Bearing capacity analysis of the untreated soil subgrade based on the undrained shear strength occurring at the 90th percentile test value yields a factor of safety of only 0.46. Using the average CBR value of 3.6 (or an undrained shear strength of 49 kPa), the factor of safety is only 0.65. Now if the subgrade soils remained free of water (an unsoaked condition) during construction, then the CBR strength may be greater than 6.5 and construction difficulties would not be encountered during paving. The designer cannot rely on this unlikely condition. Subgrade stabilization should be performed.

In the design analysis, both hydrated lime- and cement-treated subgrade layers were considered. For the hydrated lime-treated subgrade, an undrained shear strength occurring at the 90th percentile test value was used in the analysis. A strength value of 36.7 kPa (CBR=2.5) was used for the underlying untreated layer. For an assumed thickness of 30.5cm, a factor of safety of about 1.36 was obtained. This level of factor of safety should be sufficient to avoid bearing capacity failures and deep rutting during construction. If a 12.7-cm subgrade layer of soil-cement is assumed, then a factor of safety of abut 1.35 is obtained.

Model analyses were performed to determine the factor of safety of the full 59.2 cm of pavement resting on the 30.5-cm layer of hydrated lime-treated subgrade or the 12.7-cm layer of cement-treated subgrade. In both cases, the values of undrained shear strength for the treated and untreated layers occurring at the 90th percentile test values were used. When the lime-treated layer is included in the design, a factor of safety of 1.85 is obtained. Hence, the factor of safety increases from 1.29 (no treatment) to 1.85, or about 31 percent. Predicted values of ESAL (Equation 3) are much in excess of 18 million. Similarly, when a 12.7-cm layer of cement-treated subgrade is used, a factor of safety of 1.85 is also obtained. Based on Equation 3, a design factor of safety of 1.57 is required. Hence, according to this approach the thicknesses of the asphalt layer and crushed stone could be reduced. Thickness of the asphaltic layers can be reduced from 19.8 cm to 12.7 cm and the crushed stone thickness could be reduced from 39.4 cm to 25.4 cm when a 30.5 cm layer of hydrated lime-treated subgrade or 12.7-cm layer of soil-cement is used. In both cases, the factor of safety is about 1.57 -- the required value that satisfies equation 3.

LAYER COEFFICIENTS OF HYDRATED LIME- AND CEMENT- SOILS

The coefficient, a_3, may be estimated for the hydrated lime-treated subgrade and the soil-cement layer for the example described above.

The structural number, SN, is defined as

$$SN = a_1 d_1 + a_2 d_2 + a_3 d_3, \tag{4}$$

where

a_1, a_2, a_3 = layer coefficients representative of surface, base, and subbase (in this case, the treated layer), respectively, and

d_1, d_2, d_3 = actual thicknesses, centimeters, of surface, base and subbase courses, respectively.

Since a_1 and a_2 are equal to 0.44 and 0.14, respectively, the structural number is 5.6, the thickness of the asphalt is 12.7cm (or d_1=5 in.), the crushed stone thickness is 25.4 cm, and the hydrated-lime layer is 30.5 cm (or d_3=12 in.), a_3 equals 0.17. Similarly, a_3 equals 0.34 when the 12.7 cm layer of soil-cement is considered.

SUMMARY AND CONCLUSIONS

Guidelines for selection of design strength of untreated soil subgrades and subgrades treated with cement or hydrated lime were proposed. Theoretical bearing capacity analysis showed that a minimum subgrade strength must exist to avoid bearing capacity failures during construction. To maintain an incipient failure state (factor of safety, F, equal 1.0) and an assumed stable state (F = 1.5), the undrained shear strength should be 94 kPa and 144 kPa, respectively. These values correspond to CBR values of about 6.5 and 10, respectively. Corresponding values of dynamic modulus of elasticity were 92 mPa and 134 mPa. Based on a case history involving the failure of a partially completed pavement, the method proposed by Yoder (3) in 1969 appears to be a reasonable approach for analyzing strength data of corridor soils and in selecting design strengths on the basis of percentile test values.

It was proposed that if the minimum strength for a selected percentile test value is less than the minimum strength required to avoid bearing capacity failures during construction, then chemical stabilization (or other stabilization methods) of the subgrade should be considered. For example, if the tire contact stress of construction equipment is 552 kPa and the CBR is 2.5 at a selected percentile test value, then subgrade stabilization should be performed since the CBR strength of 2.5 is less than the CBR strength of 6.5 required to maintain an incipient failure condition. However, to avoid bearing capacity problems during construction, the subgrade CBR strengths should generally be greater than about 6.5.

Field CBR values of untreated subgrades obtained at two highway sites over a period of about 5.5 years were compared to soaked laboratory CBR values of corridor soils. Soaked laboratory CBR strengths appeared to represent reasonably well the long-term field CBR strengths of the clayey subgrades of the two routes. Use of soaked laboratory CBR strengths appears to provide a reasonable approach for selecting design CBR strengths of clayey subgrades.

Unconfined compressive strengths of core specimens of several soil subgrades treated with hydrated lime and cement were compared to strengths of laboratory specimens that had been mixed with hydrated lime and cement for percentile test values ranging from 100 to 10. Strengths of core specimens mixed with hydrated lime were about 85 to 90 percent of the laboratory strengths. Strengths of soil-cement cores were about 75 to 50 percent of laboratory strengths for percentile test values ranging from 100 to 0. Based on a 7-day curing period and strengths of core specimens occurring at the 90th percentile test value, unconfined compressive strengths of about 333 kPa and 707 kPa appear to be reasonable values to assume in the design of hydrated lime- and cement-treated soil subgrades, respectively. Corresponding CBR values are 11.6 and 25. Dynamic modulus of elasticity is 152 mPa and 298 mPa, respectively. Bearing capacity model analysis of an example problem showed that treated subgrades, based on these values, increased the overall bearing capacity of flexible pavement substantially.

ACKNOWLEDGEMENTS

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