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

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Selection of design strengths of soil subgrades and subgrades treated with cement or hydrated lime is a problem in pavement design analysis and construction. Different types of soils may exist in a highway corridor and 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, future pavement performance, 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 among undrained shear strength [or California bearing ratio (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 during construction is determined. A criterion is proposed for determining when subgrade stabilization is needed and methods of selecting the design subgrade strength are examined. A least-cost analysis appears to be an appropriate approach as shown by analysis of a case study involving pavement failures. 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 a reasonable means for selecting the design strength of subgrades for pavement analysis. To avoid failures of chemically stabilized layers, relationships among thicknesses of chemically treated layers and the CBR values of the untreated subgrade for a factor safety of 1.5 are presented.

A variety of soils are frequently present along any highway corridor before construction and different bearing strengths exist when the different types of soils are used to construct pavement subgrades. To avoid bearing capacity failures during construction, 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. When the design analysis is based on a selected laboratory strength, the question arises about whether the laboratory strength is representative of the long-term field strengths after paving.

When the actual subgrade strength is smaller than the minimum strength required to sustain construction traffic, it may be necessary to stabilize the subgrade soils with chemical admixtures such as cement or hydrated lime. When chemical stabilization is used, strengths of the treated and untreated layers 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 too conservative. 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*) is used, formulated on the basis of limit equilibrium. The selection scheme makes use of an approach described by Yoder (*3*).

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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 a layered structure have been presented elsewhere (*1*). Each material layer—subgrade, base, and asphaltic layers—in the pavement structure is described in the model using shear strength parameters: the angle of internal friction, ϕ, cohesion, c, and unit weights. Problems involving total stress and effective stress analyses may be solved.

The assumed theoretical failure mass consists of three zones: active and passive wedges connected by a central wedge (Figure 1). The shear surface assumed in the bearing capacity analysis of a homogeneous layer of material consists of a lower boundary, *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 shear surface, *bc*, is determined from the properties of a logarithmic spiral. The shear surface for a layered system is visualized in Figure 2.

The approach is a generalized method of slices and is an adaptation of a slope stability method (*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 (IBM 3090) at the University of Kentucky.

Shear strength of an asphaltic layer varies with temperature and temperature within the asphaltic layer varies with depth. Hence, the shear strength varies with depth. To account for this variation in the model analysis, unconsolidated-undrained triaxial compres-

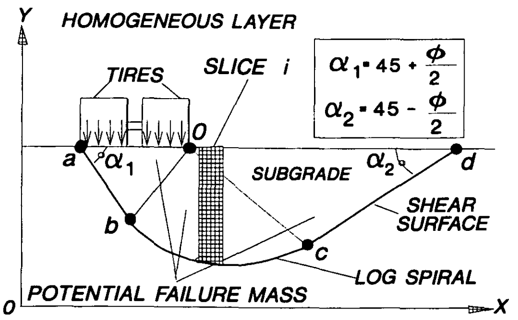

FIGURE 1 Bearing capacity analysis of a homogeneous layer.

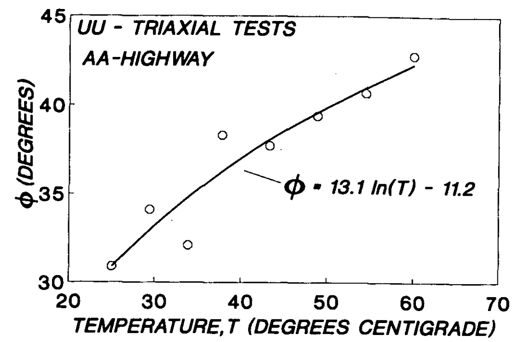

FIGURE 3 Influence of temperature on shear strength parameter ϕ.

sion tests were performed on asphaltic core specimens, which were assumed to be representative of typical flexible pavements, at temperatures ranging from 25 to 60 degrees C. As shown in Figures 3 and 4, ϕ increases and c decreases as the temperature increases. In the analyses of problems involving asphaltic layers, the total thickness of asphaltic pavement is subdivided into finite layers, and a temperature-depth model (*4*) is used to estimate the temperature at the midpoint of each finite layer. Different surface temperatures and average air temperatures may be used in the analysis. On the basis of an estimated temperature and the correlations in Figures 3 and 4, the shear strength parameters, ϕ and c, may be determined for each finite layer. Total stress parameters, ϕ and c, of crushed stone (limestone) bases were assumed to be 43 degrees and 0, respectively (*1*).

MINIMUM SUBGRADE STRENGTH

To avoid bearing capacity failures under construction traffic, the subgrade must possess some minimum strength. As the tire contact ground stress increases, the required minimum strength increases. This situation (Figure 1) was analyzed using the bearing capacity model previously described. Dual-wheel tires, a range of tire contact stresses (uniformly distributed), and undrained shear strengths, Su, of the soil subgrade were assumed. The relationships (Figure 5) of undrained shear strength and tire contact ground stresses corresponding to factors of safety of 1.0 (incipient failure) and 1.5 (assumed stable condition) were developed. For a selected tire contact stress and undrained shear strength, the factor of safety was computed. If the anticipated tire contact stress of construction traffic is known, then the required strengths to maintain an incipient failure condition or an assumed stable condition 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 among bearing ratios (ASTM D 1883) and tire contact stresses may be developed using a relationship between bearing ratio and undrained shear strength developed by Hopkins (*1,5*), or

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

or a tire contact stress, Tc, of 552 kPa; the required bearing ratio for incipient failure is about 6.5 and about 10 for an assumed stable condition.

Minimum dynamic modulus of elasticity required to maintain incipient failure and a stable condition may be determined using

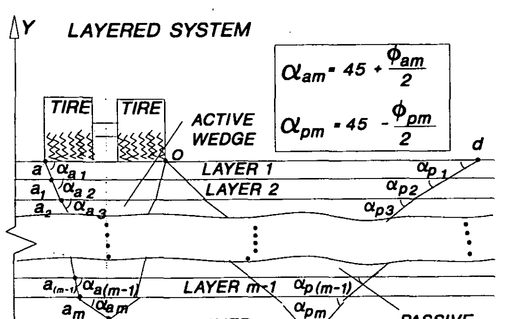

FIGURE 2 Bearing capacity analysis of a layered system.

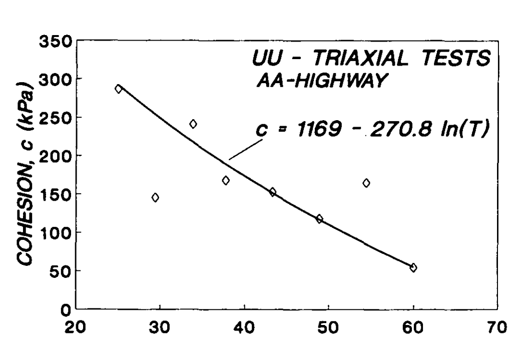

FIGURE 4 Influence of temperature on shear strength parameter c.

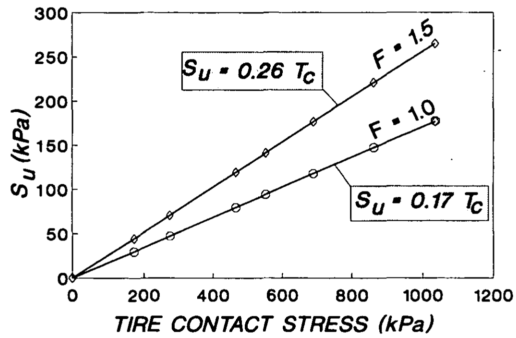

FIGURE 5 Undrained shear strength, Su, as a function of tire contact ground stress, Tc.

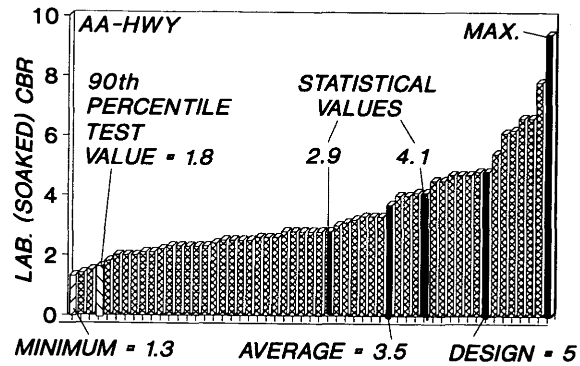

FIGURE 6 Soaked laboratory CBR values of corridor soils.

the relationship developed by Heukelom and Foster (*6*). Reanalysis of those data yields the following expression:

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

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 91,979 Kpa. 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 other types of strength parameters). Some of the approaches include using

- Lowest value,
- Average value,
- Statistical methods of estimating upper and lower values about the average value, or
- Value based on a least-cost analysis (*3*).

When the lowest bearing ratio of a data set is selected, the pavement may be overdesigned. If the average value is selected, approximately one half of the pavement may be overdesigned, whereas one half may be underdesigned (*3*). A statistical approach involving upper and lower limits for a selected confidence interval has also been proposed.

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 (equivalent axle load), and unit cost of the pavement. Unit cost of maintaining a highway is expressed in terms of a cost ratio, or unit maintenance cost divided by the unit initial construction cost. When detailed information is lacking, Yoder suggests using the bearing ratio at the 80th to 90th percentile test value.

To test and compare the results of the different approaches, an analysis of soaked laboratory CBR values of two adjacent sections (12.2 km) of a highway route located in Kentucky was performed. 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 5 and 4 million, respectively. During construction, the partially completed flexible pavements failed at numerous locations along the two highway sections.

Soaked laboratory CBR values of corridor soil samples obtained before construction are shown in Figure 6. The lowest CBR value is 1.3 and the average is 3.5. Lower- and upper-bound CBR values for a 95 percent confidence interval are 2.9 and 4.1, respectively. Percentile test value (*3*) as a function of the soaked laboratory CBR is shown in Figure 7. Cost ratio for the two highway routes was not available. Therefore, the CBR at the 90th to 80th percentile test value may be considered. At the 95th, 90th, and 80th percentile test values, the CBR values are 1.4, 1.8, and 2.1, respectively.

To compare the different CBR selection approaches, factors of safety of the design pavement section were computed using the bearing capacity model already described. Surface and air temperatures at the time of the failures were 60 and 26.7 degrees C, respectively. A temperature-depth model (*1,4*) was used to estimate the temperatures at the midpoint of each 2.54-cm asphaltic layer. Using these estimated temperatures, ϕ and c values for each

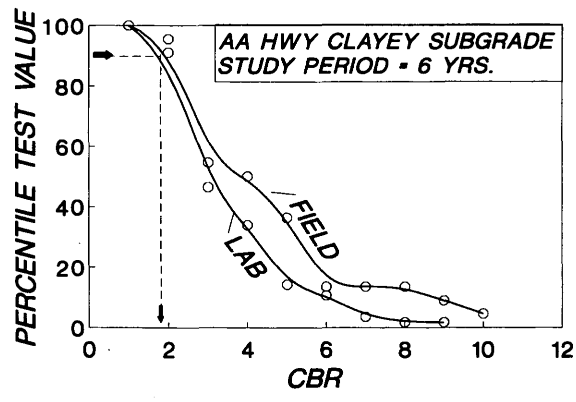

FIGURE 7 Field and laboratory percentile test values as a function of CBR: AA Route.

finite layer were estimated from the curves shown in Figures 3 and 4. CBR values were converted to undrained shear strengths using the relationship given by Equation 1. A uniformly distributed tire-contact stress of 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 is assumed to be the correct value, a factor of safety of 1.33 is obtained. If the assumption 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 CBR values obtained from statistical theory are used, then factors of safety of 1.22 and 1.43 are obtained. In each of these three cases, the factor of safety is much greater than 1. However, because the pavements failed, the factor of safety should be near 1. On the basis of CBR values equal to 1.4, 1.8, and 2.1, factors of safety of 0.91, 1.00, and 1.07 are obtained, respectively. The CBR value of 1.8 (90th percentile test value), yields a factor of safety of 1 and represents 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. On the basis of an analyses (*1*) of 237 asphaltic pavement sections of the American Association of State Highway Officials' (AASHO) Road Test (*7*), an approximate relationship (serviceability index = 2.5) between factor of safety and (weighted) ESAL was developed, or

$$F = (0.095)\ \mathrm{Ln(ESAL)} - 0.005 \tag{3}$$

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 and design factor of safety was obtained from the bearing capacity model by iteration. Thickness of the pavement is varied until the factor of safety is equal to the selected design factor of safety obtained from Equation 3. 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 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 CBR value (1.3) is assumed, then a total thickness of 53.1 cm is required. This thickness is 16.3 cm larger than the planned thickness. If the average CBR value (3.5) is used, then a thickness of 40.1 cm is obtained, which is only 3.3 cm larger than the original planned thickness. A CBR of 3.5 corresponds to a percentile test value of only about 40 (Figure 7). Accordingly, numerous portions (spot-to-spot) of the pavement would require future maintenance. Required thicknesses obtained from the upper- and lower-bound values of CBR are only 0.25 cm to 2 cm greater, respectively, than the original design section. If the CBR of 1.8 is assumed, then a thickness of 50 cm is obtained—a thickness that is 13.2 cm greater than the original planned section. As shown in Figure 6, CBR values less than 1.8 occur at only about 10 percent of the sampling sites.

Approximately one half of the total length of the highway sections were repaired using an overlay thickness of about 12.7 cm. Total thickness of the pavement at those locations was about 49.5 cm—a value that is nearly identical to the thickness (50 cm) obtained when the 90th percentile CBR 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*) and a CBR of 1.8, a thickness of 47 cm is obtained. Proper selection of a subgrade design CBR is vital to avoid construction failures and to ensure good pavement performance.

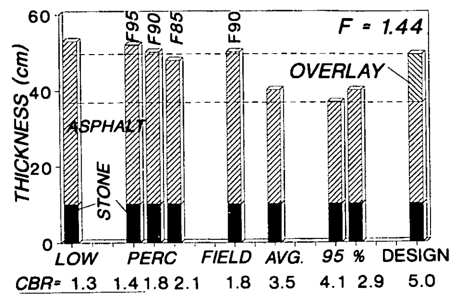

FIGURE 9 Flexible pavement thicknesses obtained for different CBR design values.

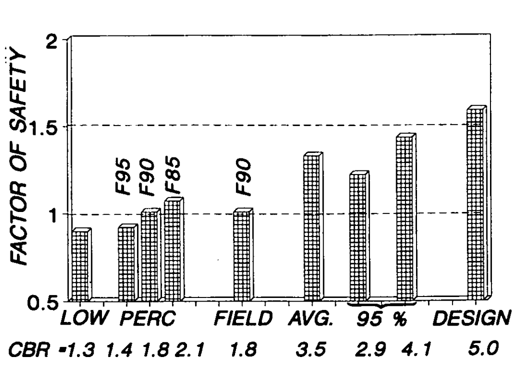

FIGURE 8 Factors of safety for different CBR design values.

EFFECT OF MOISTURE ON SOIL SUBGRADES

Subgrades built with 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 strength. Bearing strength of completed subgrades depends on long-term density and moisture. Clayey soils tend to absorb water and increase in volume. As volume increases, the density decreases and the shear strength available to resist failure decreases. Differences in bearing strength of compacted soils in soaked and unsoaked states may readily be illustrated by analyzing the results of 727 laboratory CBR tests (*1*). Each specimen was penetrated before 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.

Field observations show that bearing strength 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 was performed after exposure of the subgrade to one winter season. Values ranged from about 1 to 4—a dramatic decrease in bearing strength. Hence, as noted by Yoder and Witczak (*10*), pavement design analysis should be based on the characteristics of the completed subgrade. In areas where water infiltrates the subgrade from surface and subsurface sources, the design should be based on the strength of the soaked condition of the completed subgrade.

Many projects are scheduled years in advance and it may not be convenient, or the opportunity may not be available, to perform field tests on a subgrade 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, a question arises concerning the similarity of field and laboratory strengths.

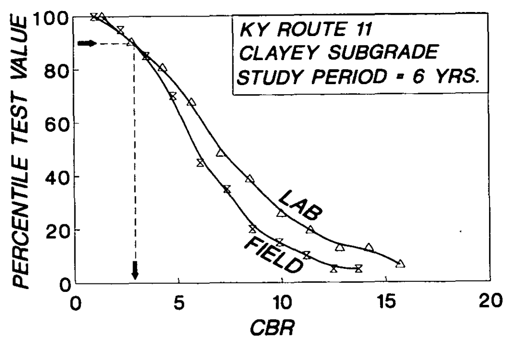

FIGURE 10 Field and laboratory percentile test values as a function of CBR: KY-11.

COMPARISON OF FIELD AND LABORATORY SUBGRADE STRENGTHS

To determine the similarity of laboratory and long-term field strengths, two highway routes were selected where numerous 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 6 years. Testing did not begin 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. Soil classification of these residual soils were A-6 and A-7, or, according to the Unified Soil Classification System, CL (clay) and CH (inorganic clay). A comparison of percentile test values as a function of laboratory and field CBR values is presented in Figure 7. 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 80 and about 10 percent, the laboratory CBR was about 90 to 70 percent of the field CBR.

Comparison of laboratory and field CBR values of the second highway route is shown in Figure 10. Soil classifications ranged from A-4 to A-7 and ML-CL (inorganic silt-clay) to CL (clay). Between percentile test values of 90 and 10, the field value is some 100 to 75 percent of the laboratory CBR. On the basis of these comparisons, soaked laboratory CBR values provide a reasonable representation of the field CBR values of the completed subgrade after sufficient time has elapsed for soaking conditions to develop. Therefore, design strength of the untreated subgrade may be based on the soaked laboratory CBR test.

DESIGN STRENGTHS OF CHEMICALLY STABILIZED SUBGRADES

Selection of a design strength of a subgrade 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. If the subgrade strength at the end of a selected curing period can be estimated with confidence, then that strength may be used in the pavement design analysis. In Kentucky, treated subgrades are allowed to cure for 7 days and substantial strength gains occur during the curing period. Optimum percentages, as determined by testing (*1*), of cement or hydrated lime are used in treating the subgrades.

General guidelines for selecting design strengths of hydrated lime- and cement-treated subgrades were developed on the basis of 7-day strengths. Several highway routes were selected and core specimens of the hydrated lime- or cemented-treated subgrades were obtained at about the end of the 7-day curing period. Numerous soils, ranging from A-4 to A-7, were used in constructing the subgrades of those routes. Unconfined compression tests were performed on the core specimens. Bag samples of the untreated soil subgrades were obtained at several locations along each route of the completed subgrade before treatment. Soil specimens 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. Unconfined compression tests were performed after aging the sealed specimens for 7 days.

Field and laboratory unconfined compressive strengths of the cement- and hydrated lime-treated specimens, as a function of percentile test values, are compared in Figures 11 and 12.

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. 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 3, respectively. Assuming that the 90th percentile test value is a reasonable working value, unconfined compressive strengths of about 333 kPa and 707 kPa (S_u = 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 bearing ratios, 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.

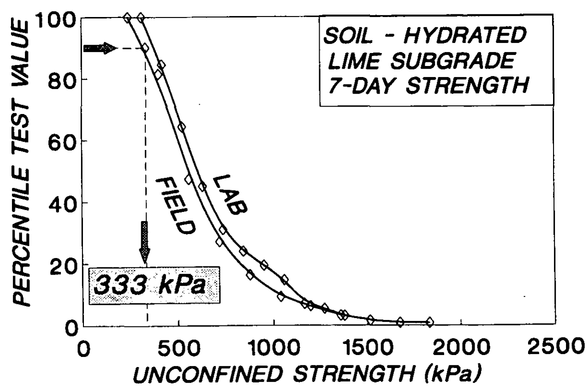

FIGURE 11 Field and laboratory percentile test values as a function of CBR: soil-hydrated lime subgrades.

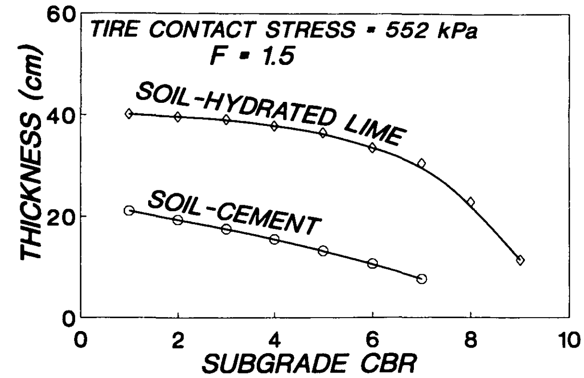

FIGURE 13 Thickness of treated subgrades as a function of untreated subgrade CBR values.

APPROXIMATE REQUIRED THICKNESS OF TREATED SUBGRADES

By using 7-day strengths, some portion of the strength gain of the chemically treated subgrade may be considered in the pavement design analysis. Bearing capacity of the treated layer is a function of thickness of the treated layer and bearing strength of the underlying untreated layer. To estimate thickness required to maintain an assumed stable condition ($F = 1.5$), bearing capacity analyses were performed. In the analysis of this two-layered problem, a tire contact stress of 552 kPa was used. The unconfined compressive strength at the 90th percentile test value (Figures 11 and 12) was assumed for the treated layer. Bearing ratio of the untreated layer ranged from 1 to 9 (or S_u - values ranging from 15 kPa to 130 kPa). Required thickness, (Figure 13), of hydrated lime-treated subgrades ranged from about 40 cm to 11 cm for CBR values of the untreated layer ranging from 1 to 9. For cement-treated subgrades and CBR values ranging from 1 to 7, required thickness ranged from about 21 cm to 7.6 cm.

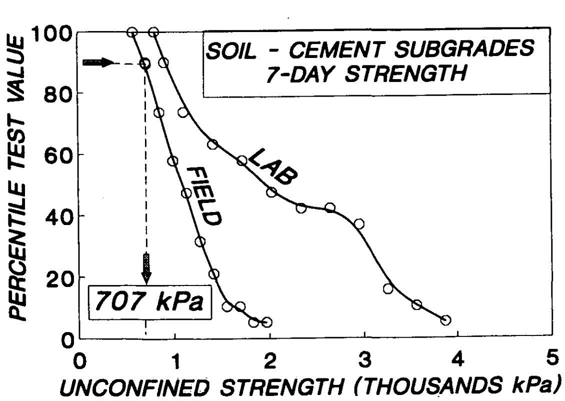

FIGURE 12 Field and laboratory percentile test values as a function of CBR: soil-cement subgrades.

SIGNIFICANCE OF TREATED SUBGRADES TO PAVEMENT STRUCTURE

Use of hydrated lime or cement increases the shear strength of a soil subgrade and improves the overall bearing capacity of a flexible pavement. This may be demonstrated by a design example. Assume that a flexible pavement is to be designed for an ESAL value of 18 million and 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 [Table 2 of the 1960 AASHO Road Test (7)] 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 consists of one-third asphaltic concrete and two-thirds crushed stone. Layer 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 (2.5 or S_u = 36.7 kPa), model analysis yields a factor of safety of 1.29. From Equation 3, the estimated ESAL is only 800,000—much lower than the design ESAL of 18 million. If the average CBR (3.6) is used, then a factor of safety of 1.55 is obtained. The estimated ESAL equals 16 million, which is near the design ESAL of 18 million. However, the percentile test value is only about 40. Hence, much maintenance may be required if the average CBR is used.

Because the CBR values at the 90th and 40th percentile test values are below a CBR of 6.5, stabilization of the soil subgrade should be considered. Moreover, difficulties may be encountered during placement of the first lift of crushed stone base if treatment is not performed. Bearing capacity analysis of the untreated soil subgrade (CBR equal 2.5) yields a factor of safety of only 0.46. Using the average CBR, the factor of safety is only 0.65. If the subgrade soils remained free of water 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 and 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 at the 90th percentile test value was used. A strength of 36.7 kPa (CBR = 2.5) was used for the underlying untreated layer. For an assumed thickness of 30.5 cm, a factor of safety of about 1.36 was obtained. This factor of safety should be sufficient to avoid bearing capacity failures and deep rutting during construction. A factor of safety of 1.35 is obtained when a 12.7-cm soil-cement layer is analyzed.

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, undrained shear strengths of the treated and untreated layers at the 90th percentile test value 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 greater than 18 million. Similarly, when a 12.7-cm layer of cement-treated subgrade is used, a factor of safety of 1.85 is obtained. Based on Equation 3, a design factor of safety of 1.57 is required. Accordingly, thickness of the asphaltic layers could 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 are 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 previously described.

Structural number, SN, is defined as

$$SN = a_1d_1 + a_2d_2 + a_3d_3, \tag{4}$$

where a_1, a_2, a_3 equals layer coefficients representative of surface, base, and subbase (in this case, the treated layer), respectively, and d_1, d_2, d_3 equals actual thickness of surface, base, and subbase courses, respectively.

Because 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 strengths of untreated soil subgrades and subgrades treated with cement or hydrated lime were proposed. Theoretical bearing capacity analysis shows that a minimum subgrade strength must exist to avoid bearing capacity failures during construction. To maintain an incipient failure state (F = 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 are 92 mPa and 134 mPa. Based on a case history involving the failure of a partially completed pavement, the method proposed by Yoder (*3*) is a reasonable approach for 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 or mechanical stabilization 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 because a CBR of 2.5 is less than a CBR of 6.5. However, to avoid bearing-capacity problems during construction, the subgrade CBR should be greater than about 6.5.

Field CBR values of untreated clayey subgrades obtained at two highway sites during a period of about 6 years were compared with 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 strength provides a reasonable approach for selecting design CBR strengths of clayey subgrades.

Strengths of subgrade core specimens mixed with hydrated lime were about 85 to 90 percent of laboratory remolded strengths. Strengths of soil-cement cores were about 75 to 50 percent of laboratory strengths for percentile test values ranging from 100 to 10. Seven-day unconfined compressive strengths of 333 kPa and 707 kPa indicate 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 those values, increased the overall bearing capacity of flexible pavements substantially.

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