Lime-Stabilized Soil Properties and the Beam Action Hypothesis

JOAKIM G. LAGUROS, Associate Professor of Civil Engineering, University of Oklahoma

The stabilizing effects of hydrated lime and chemical additives (sodium hydroxide, sodium orthosilicate, and calcium chloride) on a Texas montmorillonitic clayey soil were studied and measured by means of compressive strength, durability, consistency, consolidation and permeability tests. The strength beneficiation obtained from the addition of lime to the soil was substantial. Although further improvement with chemical additives was minimal for strength and consistency, it was considerable when viewed on the basis of consolidation, durability, and permeability test results. Also, the modulus of elasticity of the lime-established soil was determined and compared with that of the same soil stabilized with cement. Based on Winkler's model, a hypothesis was advanced to calculate the deflection and stresses in a stabilized soil layer, wherever such a layer is used as a pavement component.

•STABILIZATION OF in-place soils by admixtures has become an increasingly accepted method for improving the bearing capacity of either the substructure or some other component of the pavement. To measure the amount of beneficiation which weak soils derive from the incorporation of admixtures, the consistency limits, the unconfined compressive strength and the durability tests have been widely used. Of great importance to load-sensitive soils are other properties such as compressibility, permeability, and confined compressive strength. The extent of the stabilizing effect may also be evaluated by measuring the static modulus of elasticity, which is an important parameter in the determination of load-stress-deflection relationship, when it is hypothesized that a stabilized soil base course behaves as a semirigid finite beam resting on an elastic foundation.

In this paper the properties of a montmorillonitic clayey soil stabilized with hydrated lime and with lime and chemicals are evaluated. In addition, the modulus of elasticity of the lime-stabilized soil is compared with that of the cement-stabilized soil and, based on Winkler's model, the finite beam approach is suggested.

MATERIALS

The stabilizing agents used for the poorly reactive soil, whose properties are given in Table 1, were a high-calcium hydrated lime, sodium hydroxide, sodium orthosilicate, and calcium chloride. A preliminary investigation (10) indicated that these chemicals were most effective.

PREPARATION OF MIXES

The soil was air dried, pulverized, and screened through a U. S. No. 10 sieve. Each batch of soil was first hand mixed with lime, then part of the compaction water

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TABLE 1

PROPERTIES OF SOIL

Property	Value
Sample designation	Texas clay
Sampling location	Harris Co., Texas
Parent material	Coastal Plain deposit, largely deltaic
Great soil group	Grumusol
Soil series	Lake Charles
Horizon	C
Sampling depth, in.	39-144
Textural composition:[1]	
Gravel (> 2 mm), %	0.0
Sand (2 - 0.074 mm), %	3.0
Silt (0.074 - 0.005 mm), %	36.0
Clay (< 0.005 mm), %	61.0
Clay (< 0.002 mm), %	40.0
L.L., %[2]	65
P.L., %[3]	18
P.I.	47
CEC, meq/100 g[4]	27.30
Carbonates, %[5]	16.20
pH[6]	8.2
Organic matter, %[7]	0.13
Non-clay minerals[8]	Quartz, feldspar
Predominant clay mineral[8]	Montmorillonite
Classification:	
Textural (Triangular chart, BPR)	Clay
Engineering (AASHO M 145-491)	A-7-6(20)
Unified	CH

[1]Textural gradation tests performed on soil fraction passing No. 10 sieve; ASTM Method 422-54T.
[2]ASTM Method D 423-54T.
[3]ASTM Method D 424-54T.
[4]Cation exchange capacity determined by ammonium acetate (pH = 7) method on soil fraction less than 0.42 mm.
[5]Versenate method for total 1 N HCl-soluble calcium.
[6]Glass electrode method using suspension of 15 g soil in 30 cc distilled water.
[7]Potassium dichromate method.
[8]X-ray diffraction analysis.

(that amount of water to give optimum moisture content for near standard Proctor density) was added and the mixture was hand mixed. Whenever chemicals were used, they were dissolved or dispersed in the compaction water.

METHODS OF EVALUATION

Preliminary Investigation

From mixes containing various amounts of lime and from those with various amounts of lime and chemicals, and on the basis of 28-day unconfined compressive strength, the optimum amounts (by ovendry weight of soil) of lime and chemicals were established as:

Lime,	6 percent;
Sodium hydroxide,	0.25 percent;
Sodium orthosilicate,	0.50 percent; and
Calcium chloride,	0.50 percent.

Consistency Tests

When mixing was completed, 2-in. high by 2-in. diameter specimens were molded in a drop hammer molding apparatus to near standard Proctor density, wrapped well in waxed paper, and cured in a chamber at 95 ± 5 percent RH and 70 ± 5 F for 28 days. At the end of the curing period the specimens were pulverized and their consistency limits were determined by the ASTM standard methods. The results are given in Table 2.

Unconfined Compressive Strength

Following the procedure explained under the consistency tests, specimens were molded and cured for 28 days. Then they were unwrapped and immersed in distilled water for 24 hr, after which they were tested to failure to determine their unconfined compressive strength. These results are also given in Table 2.

Durability Tests

The natural and stabilized soils were evaluated by the Iowa freeze-thaw (6) and the standard test (1). In both methods the specimens were cured for 7 days. Results are given in Tables 3 and 4.

TABLE 2

CONSISTENCY LIMITS AND UNCONFINED COMPRESSIVE STRENGTH OF NATURAL AND STABILIZED SOIL

Specimen	L.L. (%)	P.L. (%)	P.I.	Comp. Strength (psi)
Natural soil	65	18	47	0
Soil + 6% lime	30	15	15	300
Soil + 6% lime + 0.25% NaOH	28	15	13	320
Soil + 6% lime + 0.50% Na_4SiO_4	30	15	15	300
Soil + 6% lime + 0.50% $CaCl_2$	28	15	13	330

Triaxial Compression Tests

From each mix batch, nine cylindrical specimens were molded in the Harvard miniature compaction apparatus which gives a compacted sample at approximately standard Proctor density. All specimens were cured for 28 days as previously explained. At the end of the curing period

TABLE 3

DURABILITY INDICES FOR NATURAL AND STABILIZED SOIL BY IOWA TEST[a]

Specimen	P_f (psi)	P_{c_f} (psi)	R_f (%)	P_i (psi)	P_{c_i} (psi)	R_i (%)
Natural soil	0	0	-	0	0	-
Soil + 6% lime	0	396	0	360	448	75
Soil + 6% lime + 0.25% NaOH	320	400	80	382	490	78
Soil + 6% lime + 0.50% Na_4SiO_4	284	379	75	336	412	89
Soil + 6% lime + 0.50% $CaCl_2$	383	488	78	396	531	75

[a]Symbols used are as follows:

P_f = unconfined compressive strength of freeze-thaw specimen;
P_{c_f} = unconfined compressive strength of control specimen;
$R_f = \left(R_f/P_{c_f}\right) \times 100$;
P_i = unconfined compressive strength of immersed specimen;
P_{c_i} = unconfined compressive strength of control specimen; and
$R_i = \left(P_i/P_{c_i}\right) \times 100$.

the specimens were immersed for 24 hr in distilled water, drained naturally in the curing chamber, and tested in a triaxial compression machine under undrained conditions. Three specimens were used for each of the three lateral pressures of 10, 20, and 30 psi. The cohesion (c) and angle of internal friction (ϕ) of any particular mix were determined graphically using the Coulomb-Mohr theory given by the formula:

$$s = c + p \tan \phi \qquad (1)$$

Results are given in Table 5.

TABLE 4

SUMMARY OF STANDARD FREEZE-THAW TESTS

Specimen	Init. Density (pcf)	Init. Moisture (%)	No. Cycles[a]	After Cycling: Density (pcf)	After Cycling: Moisture (%)	Soil Volume: Change (%)	Soil Volume: Loss (%)
Natural soil	102.9	22.3	0	129.4	35.1	1.1	10.2
Soil + 6% lime	118	23.0	3	124.1	30.9	1.0	9.2
Soil + 6% lime + 0.25% NaOH	118.5	23.5	5	123.1	31.0	1.0	9.7
Soil + 6% lime + 0.50% Na_4SiO_4	118.5	23.5	5	123.2	31.1	1.0	9.8
Soil + 6% lime + 0.50% $CaCl_2$	118.5	23.5	5	123.2	31.0	1.0	9.7

[a]Source: ASTM Standards (1, p. 34).

Permeability Tests

The coefficient of permeability (k) for the five different mixtures (Table 5) was determined by using a falling-head miniature permeameter (Soiltest Model K-620). The test specimens, identical in form and method of preparation with those for the triaxial compression test, were cured for 28 days in their molds and then saturated and tested.

Consolidation Tests

The apparatus used in this phase of the study was the fixed type consolidometer (Soiltest Model C-252). Through trial tests the correct amount of soil to give a 1-in. high specimen and the static load required to give standard Proctor density were established. The specimens, 4 7/16 in. in diameter and 1 in. high, were compacted by the static load in the specimen ring, and the assembly was cured for 28 days. The two-way drainage test was run after saturating the specimen in the ring. Figure 1 shows the consolidation characteristics of the natural and stabilized soil through the e-log p curves and c_v values of the test specimens. For the soil-lime-chemical mixtures, the differences in the e-log p and c_v values were extremely small and, therefore, a typical curve was drawn.

TABLE 5

RESULTS OF TRIAXIAL COMPRESSION TESTS AND k VALUES

Specimen	c (psi)	ϕ (deg)	$s = c + p \tan \phi$	k (cm/sec)
Natural soil	25	20	$s = 25 + 0.36p$	8.6×10^{-6}
Soil + 6% lime	28	49	$s = 28 + 1.15p$	6.8×10^{-5}
Soil + 6% lime + 0.25% NaOH	43	42	$s = 43 + 0.90p$	3.9×10^{-5}
Soil + 6% lime + 0.50% Na_4SiO_4	42	41	$s = 42 + 0.87p$	4.8×10^{-5}
Soil + 6% lime + 0.50% $CaCl_2$	43	44	$s = 43 + 0.97p$	3.5×10^{-5}

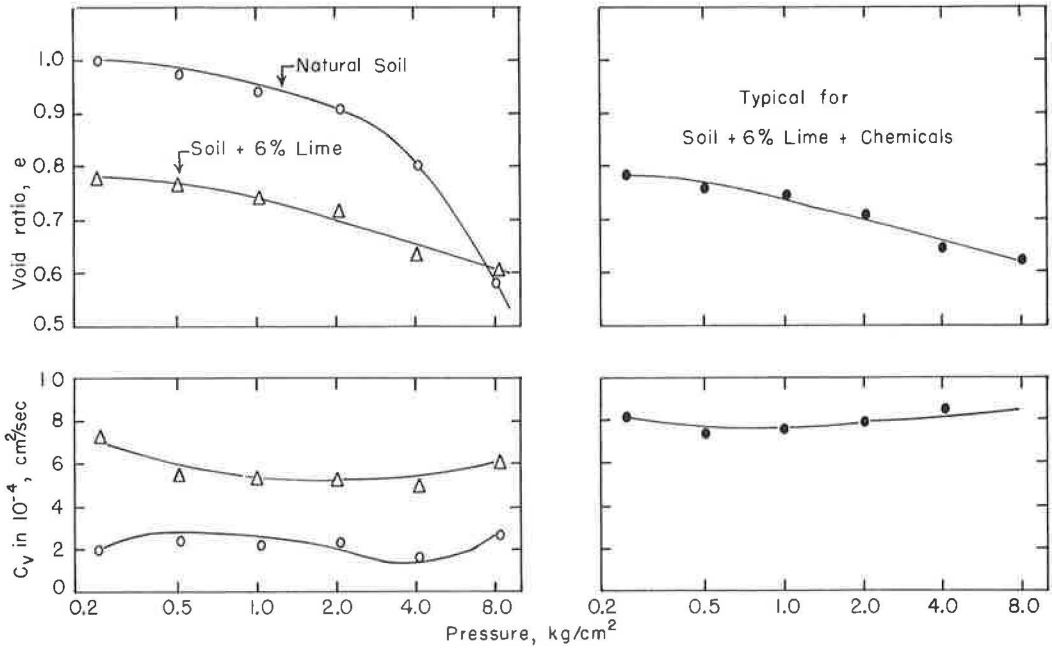

Figure 1. Consolidation characteristics of test specimens.

Static Modulus of Elasticity

Specimens were molded in the Harvard miniature compaction apparatus and cured as for the triaxial compression tests. At the end of a 7-day curing period, they were unsealed, immersed in water for 24 hr, and tested in unconfined compression to failure at a rate of 0.05-in./min strain. For reasons of comparison, specimens stabilized with 12 percent portland cement and with 12 percent cement plus 0.25 percent sodium hydroxide were prepared and cured in the same manner. The lime-stabilized specimens showed so little strength gain at the end of the 7-day curing period that their modulus of elasticity could not be well defined. Therefore, the curing period was extended to 28 days. However, the cement-stabilized specimens showed marked gain in strength at the end of the 7-day curing period. This is supported by previous experience (11, 13) which indicates that the 7-day unconfined wet compressive strength is a fairly reliable measure for soil-cement. Table 6 presents a summary of pertinent data rela-

TABLE 6

STRENGTH—E_{sc} RESULTS

Specimen	Max. Dry Density (pcf)	Opt. Moisture Content (%)	After Curing and 24-Hr Immersion: Density (pcf)	Moisture Content (%)	Unconfined Compressive Strength (psi)	Avg. E_{sc}, (psi)
Texas clay + 6% lime	118	23.0	127.2	24.8	300	8.8×10^3
Texas clay + 6% lime + 0.25% NaOH	118.5	23.5	127.8	25.1	320	8.9×10^3
Texas clay + 12% cement	102.5	22.6	122.1	26.3	350	9.4×10^3
Texas clay + 12% cement + 0.25% NaOH	102.5	22.6	123.2	26.6	420	1.9×10^4

tive to the test specimens. The E_{sc} (static modulus of elasticity under compression) values were calculated from the stress-strain measurements of each specimen by fitting a tangent to the straight portion of the stress-strain curve. This straight portion extended over a section approximately 30 to 60 percent of the maximum strength.

In investigating the elastic behavior of soil-cement, triaxial loading has been used (2). Although this type of testing is desirable in many instances, for cohesive materials such as the ones used in this study, it does not seem to be absolutely essential. Under triaxial loading the E_{sc} values are expected to be higher. It is questionable, however, if at the shoulder pavement interface, where conditions are critical, the confinement pressure will be of a magnitude to increase significantly the supporting power of a base course.

Since it is proposed to use the stabilized soil layer as a beam, the use of the unconfined compression test in lieu of a beam test may invite some argument. It must be remembered, however, that because of inherent structural discontinuities, if not for reasons of unpredictable crack formation in the stabilized soil, the static modulus of elasticity in flexure may lead to unsafe high values of design, whereas the same modulus in compression is more conservative and does not appear to lead to uneconomical design since both moduli are of the same order (5).

DISCUSSION OF RESULTS

It is an established fact that the addition of hydrated lime to clay soils produces a notable improvement in their workability as a result of the predominant interaction between the clay particles and the calcium ions (9). In view of this aggregating influence of lime, a comparison of the consistency, strength, compressibility, durability, and permeability properties of the natural soil with those of the lime-stabilized soil leads to the conclusion that, in the stabilized soil, these properties are greatly improved. Thus, the lime-stabilized soil gains strength, attains resistance to weathering, and becomes more permanently compressible in a much shorter time. A reduction in the measured permeability of the lime-stabilized soil is also an indication of the improvement effected by lime.

Further modifications in consistency and unconfined compressive strength resulting from the addition of sodium hydroxide, sodium orthosilicate, and calcium chloride to the soil-lime mixture are obtained. Although the data indicate small changes (about 7 percent), they prove that the montmorillonitic clayey soil responded favorably to the additions of sodium and calcium ions—at least in the form in which they were used in this study. That sodium hydroxide was more effective than sodium orthosilicate may be explained in terms of the additional effect of hydroxide which interacts with the silica surfaces of the quartz in the soil, producing more numerous bonds. Consistency test results do not show any difference between the effectiveness of sodium hydroxide and calcium chloride, and the difference in the unconfined compressive strength is too small to provide a definite conclusion. Although this is true at 6 percent lime, a study (10) with 3 percent lime has indicated that unconfined compressive strength with sodium hydroxide is 50 percent more (280 psi) than with calcium chloride (180 psi). This difference in response may be explained by the lime fixation or retention point theory (8) whereby at low Ca^{++}-ion concentration, resulting from 3 percent lime and 0.50 percent $CaCl_2$, very little, if any, lime is available for pozzolanic reactions; at 6 percent lime, a sufficient amount of lime is available for such reactions and, therefore, relatively high strengths are obtained. On the other hand, sodium hydroxide, both at low and high lime concentrations, seems to promote the reaction of calcium and silica either by making more silica available or by providing suitable alkaline conditions for the reaction to take place.

The durability tests produced similar results as the unconfined compressive strength test and, therefore, similar conditions of stabilization mechanism may be assumed as controlling. Qualitatively, however, a beneficial divergence is noted in the behavior of the lime-stabilized soil when chemical additives are used. On the basis of the R_f values of the Iowa freeze-thaw test, the weather resistance of the soil is substantially increased from zero to about 80 percent; the standard freeze-thaw test indicates an

increase from 3 to 5 cycles on the addition of the chemicals. These increases can only be attributed to the effect of the added chemicals, which lower the freezing point of water and also increase the cohesion of the mix.

When lime was added to the soil, its cohesive strength was improved simultaneously with an increase in the angle of internal friction as evidenced from the results of the triaxial compression test. When the chemicals were added to the soil-lime mix, a gain in the overall shearing resistance was recorded, but the increase in the cohesive strength was somewhat masked by a slight decrease in the angle of internal friction. Similar trends have been reported for soil-cement (11), and it is reasonable to expect that the same explanation for this phenomenon holds true. The increase in cohesion has already been accounted for. The decrease in the internal friction could not be traced to a possible reduced agglomeration of particles, at least not as far as the plastic properties of the mix were concerned. The soil-lime mixes did not seem to have plasticity indices very different from those of soil-lime modified with chemicals. The addition of the chemical, then, gives greater cohesion resulting from greater interparticle attraction, but the ions possibly increase the interparticle distance slightly as they take their places between the already aggregated particles, with the result that interparticle contact is reduced and, therefore, internal friction is slightly reduced.

The compressibility tendency of a soil, which has been compacted to a maximum standard Proctor density, presents a problem only when traffic compaction in excess of that anticipated takes place. This, coupled with the saturation of the soil mass, is a condition that cannot be excluded as a possibility during the life of a highway pavement. Therefore, the consolidation properties and, associated with this, the coefficient of permeability, k, were separately determined for the soil and all the mixtures.

Solely in terms of the order of k, it may be said that the fat clayey soil attains the permeability properties of a loam when it is stabilized with lime and a further reduction in the k value occurs on the addition of the chemicals. This modification is reflected in the coefficient of consolidation, c_v, and it is attributable to the already discussed flocculating character of lime and the chemicals. Also, the comparatively flat e-log p curves for the stabilized soil suggest that large applied pressures will not cause high settlements as appears to be the case for the natural soil.

A comment deserves to be made regarding the apparent ineffectiveness of chemicals in the lime-stabilized clayey soil when measured by the unconfined compressive strength and the consistency limit tests. In both tests, the values obtained are indications of the shear strength of the mixture, but the role of hygroscopic or gravitational water is not distinguished from that necessary to give cohesion to the mix. It has been argued (14) that the water acts as a filler separating particles and resisting close approach and that shearing possibly occurs along the water space. Had the effective values been measured in the triaxial test, it would not have been surprising to record lower values of cohesion and slightly greater values of internal friction than the ones observed. For the durability test results, it may be said that the continuous moist environment was the factor which enhanced the effectiveness of chemicals. The permeability and consolidation test results, where neutral stresses are of no bearing, prove the beneficial effect of the chemicals on the lime-stabilized soil.

The E_{sc} values tend to reflect the same beneficial trend due to the chemicals both for the lime-stabilized and the cement-stabilized soil (11). Although the data suggest a relationship between unconfined compressive strength and E_{sc} values, unreported work on different soils by the author indicates that this may not be true. Furthermore, this limited study seems to indicate that the E_{sc} values may heavily depend on either the clay content or the type of clay mineral, since they manifest a propensity for a kaolin-illite-montmorillonite order.

FINITE BEAM CONCEPT APPLIED TO SOIL-LIME AND SOIL-CEMENT

The stresses induced in the constituent layers of a pavement by surface loads have been the subject of experimental and theoretical research for many years. The Boussinesq approach and the layered system solution (15) have been used in various forms which necessarily include simplifying assumptions when applied to compacted or modified soil masses.

Finite Beams

Lime- or cement-stabilized soil manifests such properties of strength that it is reasonable to classify it as a stiff, but not altogether rigid, construction material possessing an adequate modulus of elasticity, especially in its free-of-cracks condition. Therefore, it seems equally reasonable to analyze the stresses in such a pavement component by a deflection beam method, the simplest of which is the one given by the traditional Winkler model (7). In drawing such a similarity, it is assumed that:

1. The soil-lime or soil-cement base, which has a thin asphaltic overlay, rests on a subgrade which constitutes the elastic foundation;
2. Since the Winkler model idealizes the foundation with an infinite number of springs, the stabilized soil "beam" is made up of independent beams of length equal to the width of the pavement, of width of unit length, and of height equal to the base course thickness;
3. Since the Winkler approach provides a good engineering approximation for all but rigid beams (4), it can be used in our case; and
4. By the method of superposition (end conditioning principle) an infinite beam is transformed to a finite beam, such as the stabilized soil beam, for the particular load(s) under consideration.

The Winkler model requires the solution of the equation:

$$EI \frac{d^4 y}{dx^4} + k y = w(x) \tag{2}$$

where

EI = flexural rigidity of the beam;

k = a constant;

$w(x)$ = applied loading;

$y = e^{\lambda x} (C_1 \cos \lambda x + C_2 \sin \lambda x) + e^{-\lambda x} (C_3 \cos \lambda x + C_4 \sin \lambda x)$;

C_1, C_2, C_3, C_4 = constants of integration; and

x = distance from load to any section.

The expression y gives the deflection and, from its derivatives, the slope, moment, and shear of the beam may be obtained.

Whichever solution is used (3), a common phase is that of selecting certain constants, including the modulus of elasticity of the beam.

Evaluation of λ

In the equation for the deflection, y, the important term λ appears. This is the "characteristic" of the system, and it is evaluated from the relationship;

$$\lambda = \sqrt[4]{\frac{kb}{4EI}} \tag{3}$$

where

E = modulus of elasticity of beam;
I = moment of inertia of beam;
k = subgrade reaction coefficient; and
b = width of beam.

For the stabilized soil base courses, it is proposed to use the E_{sc} values for E, to calculate I assuming a 1-ft wide solid beam and to evaluate the usable value of k of the subgrade by the plate loading test modified accordingly (16).

Loads

Loads on pavements, the form of area through which they are applied and, consequently, their intensity have raised formidable engineering problems. Simplifications and approximations have been used successfully.

An examination of the form and size of area of tire imprints (15, 17) reveals that the actual elliptical form, which has been approximated to a circular area with equivalent area, may as well be simplified to an equivalent rectangular area having a width of 1 ft, so that it extends over the entire width of the proposed beam, and a length to be calculated from the equivalent area. In fact, in recent studies (15) tire imprints have been accepted as having a form similar to a rectangle with semicircles attached at the two opposite ends. Thus, a 9 x 20 truck tire transmitting a load of 9,000 lb at a pressure of 86 psi will be equivalent to a uniformly distributed load of about 1,000 psi over a 10-in. section of the beam. Since also more than one wheel will be applied along a transverse line on a pavement, the problem assumes the form of a beam with free ends carrying discontinuous uniform loads. The total effect of the various uniform loads is the summation of the effects of each load. The numerical problem resulting from this analysis is expedited by the use of coefficients or influence lines or some other method (3, 4).

The preceding treatment of stresses and deflections at a stabilized soil base-subsoil interface is suggested as a possible method of analysis. Although oversimplified, at least it may prompt questions and answers which will lead to more refined and sophisticated approaches.

CONCLUSIONS

1. Lime stabilization of the Texas montmorillonitic clayey soil offers some promise which becomes considerable with the use of chemical additives.
2. The strength beneficiation of the soil-lime and the soil-lime chemical mixtures seems to be small, judging from the unconfined compression test results; triaxial test results indicate greater improvement.
3. The addition of the chemicals does not improve the consistency properties of the soil-lime mixtures once lime is used as the main stabilizing agent.
4. The resistance of the soil-lime mixtures to weathering, as measured by the Iowa freeze-thaw and the standard test, is greatly increased with the use of the chemicals.
5. Similarly, the chemicals are effective in promoting the compressibility properties of the soil-lime by increasing its permeability.
6. The static modulus of elasticity in compression, E_{sc}, reflects the beneficial effect of sodium hydroxide, but it is not necessarily related to the unconfined compressive strength.
7. A lime- or cement-stabilized soil base course may be treated structurally as a finite beam resting on an elastic foundation, where stresses and deflections are calculated by employing the Winkler model and the Hetenyi approach, the suggested method being regarded as tentative until actual measurements can be made.

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REFERENCES

1. ASTM Standards, 1958, Part 4. ASTM, 1958.
2. Balmer, G. C. Shear Strength and Elastic Properties of Soil-Cement Mixtures Under Triaxial Loading. Proc. ASTM, Vol. 58, 1958.

3. Barden, L. Finite Beams Resting on an Elastic Soil. Civil Eng. and Public Works Rev., Vol. 57, Nov. 1962.
4. Barden, L. Influence Coefficients for Beams Resting on the Soil. Civil Eng. and Public Works Rev., Vol. 58, May 1963.
5. Felt, E. J., and Abrams, J. S. Strength and Elastic Properties of Compacted Soil-Cement Mixtures. ASTM STP No. 206, 1957.
6. George, K. P., and Davidson, D. T. Development of a Freeze-Thaw Test for the Design of Soil-Cement. Highway Research Record No. 36, pp. 77-96, 1963.
7. Hetenyi, M. Beams on Elastic Foundation. Univ. of Michigan, 1946.
8. Ho, C., and Handy, R. L. Characteristics of Lime Retention by Montmorillonitic Clays. Highway Research Record No. 29, pp. 55-69, 1963.
9. Ladd, C. C., Moh, Z. C., and Lambe, W. T. Recent Soil-Lime Research at the Massachusetts Institute of Technology. Highway Research Board Bull. 262, pp. 64-85, 1960.
10. Laguros, J. G., and Davidson, D. T. Effect of Chemicals on Soil-Cement Stabilization. Iowa Eng. Exp. Sta., Final Rept., April 1962.
11. Laguros, J. G., and Davidson, D. T. Effect of Chemicals on Soil-Cement Stabilization. Highway Research Record No. 36, pp. 172-203, 1963.
12. Soil-Cement Laboratory Handbook. Portland Cement Assoc., 1959.
13. Road Research Laboratory. Soil Mechanics for Road Engineers. London, Her Majesty's Stationery Off., 1954.
14. Seed, H. B., et al. The Strength of Compacted Cohesive Soils. In Research Conference on Shear Strength of Cohesive Soils. New York, ASCE, 1960.
15. Sowers, G. F., and Vesic, A. B. Vertical Stresses in Subgrades Beneath Statically Loaded Flexible Pavement. Highway Research Board Bull. 342, pp. 90-123, 1962.
16. Terzaghi, K. Evaluation of Coefficients of Subgrade Reaction. Geotechnique, Vol. 5, 1955.
17. Yoder, E. J. Principles of Pavement Design. New York, John Wiley and Sons, 1959.