Dynamic Response of Raw and Stabilized Oklahoma Shales

SUBODH KUMAR AND JOAKIM G. LAGUROS

The results of a laboratory study of the combined effects of simulated climate- and traffic-induced stresses on raw and lime-stabilized Oklahoma shales are reported. Samples were compacted to near maximum density at optimum moisture content. They were first exposed to wet-dry cycles and were then subjected to cyclic loading, which was increased in discrete magnitudes. Lime additions to the shale varied between 1 and 6 percent, and the number of wet-dry cycles varied from 5 to 50. The behavior of the samples was observed in a "humid" as well as a "dry" state. Response to loading was determined by means of the split-tensile-strength technique, in which diametral strain was measured for the first 100 cycles for various loads and load frequencies. Graphs for diametral strain provided more meaningful information than those for stresses along the horizontal or vertical diameter and are thus considered reliable predictors of dynamic shale behavior. Generally, for a given load and frequency, the strains increased monotonically with number of load applications. The effect of increasing the frequency without varying the load was found to be insignificant except when crack formation had already begun, in which case diametral strain increased with increasing frequency under the same load. In view of the various combinations of test parameters considered, the response of the shales can be explained in terms of the lime-clay reaction. Shales that contained large amounts of clay showed reduced diametral strain and brittle characteristics when only 1 percent lime was added. In shales with a low clay content, however, no beneficial effect on deformation was evident until the lime admixture reached 6 percent. The effect of wet-dry cycles on samples in the humid state was insignificant, but there was a noticeable effect on samples in the dry state.

In Oklahoma, shales are extensively used in highway construction, primarily as subgrade and embankment material. Frequently, they have to be upgraded and modified by lime stabilization to increase their stability and durability. Thus, it becomes imperative to assess their suitability for the intended purpose.

Studies have demonstrated that the structure of a soil mass is affected by the degree of compaction and moisture content (1) and that it is further affected by traffic patterns and local environmental features (2).

At small frequency and magnitude of repeated stress, the soil shows high plasticity; when frequency and level of stress increase, the behavior becomes progressively more elastic (3,4). At higher stresses, however, the effect of load repetition is completely lost. Pretorius and Monismith (5) measured tensile and compressive strains from flexural tests made on soil-cement beams and observed that the strain remained constant over a large number of stress applications, after which the rate of change in strain began to increase. Once the strain rate started to increase, it did so at an increasing rate until rupture occurred. Pretorius and Monismith further observed that the specimen did not fail even after 1 000 000 applications of load and exhibited a constant strain output during the entire test. When the specimen failed under a higher stress, the stress history remained essentially unchanged and the effect of previously induced damage was comparatively insignificant. In the experiments conducted on clays by Larew and Leonards (6), for the frequency range of 1-20 applications/min (APM), the specimen deformation depended on the number of stress applications but was independent of the frequency of application, provided the saturation was not too high. Most of the repeated-load tests, however, were conducted under essentially a constant load cycle. This is greatly at variance with actual field conditions.

In a layer of a subgrade material subjected to a load traveling on the pavement, the area immediately below the load is compressed while the areas in front of and behind the load undergo tension. On the underside of the layer, the strain pattern is just the opposite. Rather than a small compressive strain, a relatively large tensile strain develops at the underside of the layer, and it is from this side, most probably, that crack formation initiates (7). Thus, in the design of subgrades, if one takes into account the action of the traffic loading, the tension parameters appear to be critical. These parameters can be measured in the laboratory by using either the flexure test or the split tensile test. Carniero and Barcellos (8) found a significant correlation between the tensile and compressive strengths of concrete. Metcalf and Frydman (9) have shown a similar correlation for stabilized soils.

The specimens in the split tensile test display a fairly well-defined surface failure that is located in the neighborhood of the vertical diametral plane. Expressions for determining the stress distribution along the horizontal and vertical diameters have been given by Timoshenko and Goodier (10), Frocht (11), and Peltier (12). To account for the deformation during the split tensile test, Kumar and others (13) have presented expressions that relate various stress distributions to diametral strain. Inasmuch as the repeated loadings used in the split tensile test appear to simulate field conditions more accurately, this method was adopted in a study undertaken to evaluate the effects of weathering and repeated loading on raw and lime-stabilized Oklahoma shales.

EXPERIMENTAL PROCEDURE

Shale Characteristics

Four shales that vary mineralogically were selected from four different regions in Oklahoma and were stabilized (treated) with lime [U.S. Pharmacopeia $Ca(OH)_2$] in amounts from 0 to 6 percent based on previous experience (14-16). The shale properties are given in Table 1 (17). The shales were pulverized to pass the no. 10 sieve. The shales and their stabilized counterparts were then compacted statically, at a 1000-lb load, to maximum dry density at optimum moisture content as determined in accordance with AASHTO T-99-70. The specimens were 1.35 in in diameter and 2.95 in in height, which gives an aspect ratio greater than 2.

Wet-Dry Cycles

The compacted samples were left for equilibration in a humidifier for seven days and were then subjected to the required number of wet-dry cycles. Based on the information provided by Laguros (18) for wet-dry cycles in Oklahoma, the numbers of cycles chosen for this study were 0, 5, 15, 30, and 50. The imbibing of water by the samples resulted in their destruction; therefore, wetting included placing the samples in a 100 percent relative humidity atmosphere at 72°F for 24 h. Drying was achieved by placing the samples in an oven at 140°F for 12 h. Half of

Table 1. Properties of four shales studied.

Property	Shale 13	Shale 15	Shale 21	Shale 24
County of origin	McCurtain	Leflore	Stephens	McIntosh
Physiographic region	Red River	Quachita Mountain	Red Bed Plains	Prairie Plains
Geologic unit	Washita	Stanley	Claypool	Senora
Grain size (%)				
Silt	35	9	49	44
Clay				
5μm	59	18	39	23
<2μm[a]	48	14	25	14
Volume change[b] (%)	10.3	0.2	6.8	4.4
AASHTO classification	A-7-6	A-1-b	A-6	A-4
Clay minerals (%)				
Chlorite	-	22	6	-
Illite	-	38	15	68
Kaolinite	11	40	-	28
Montmorillonite	89	-	22	4
Mixed-layer montmorillonite-illite	-	-	57	-
Other minerals	Quartz	Quartz, feldspar	Quartz, feldspar	Quartz, feldspar

[a]Included in <5μm fraction.
[b]Determined by using Soiltest C-290 volume-change apparatus and associated method.

Figure 1. Equipment for cyclic split-tensile-strength test.

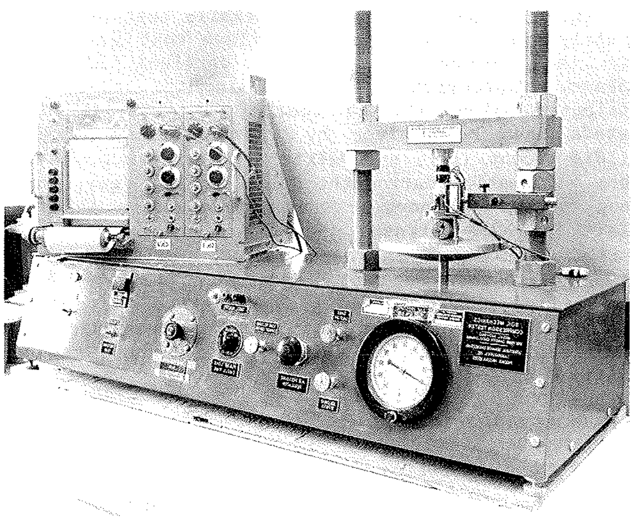


Figure 2. Arrangement of load cell and displacement strain gage during testing.

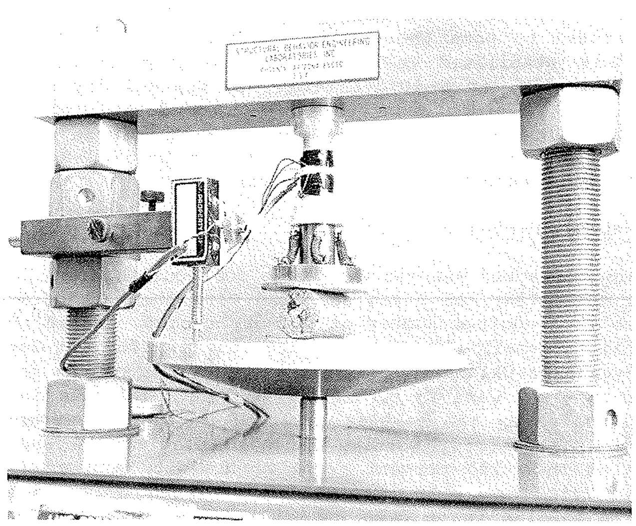
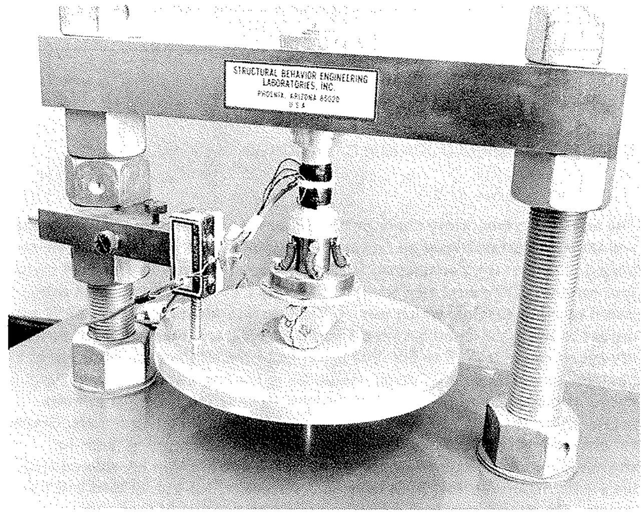


Figure 3. Detailed view of sample under test.



the specimens were in a "humid" state at the time of load testing, and the other half were "dry". The distribution of water within the sample cross section was not determined.

Repetitive Loadings

The equipment used for the application of repetitive loads is shown in Figure 1. It is capable of applying a maximum load of 1250 lb at discrete frequencies ranging from 2.4 to 120 cycles/min. The dwell time could be varied continuously from 0.05 to 10 s. In view of the total setup and its limitations, a dwell time of 1 s was chosen. The frequencies of load application chosen were 6, 12, 24, and 40 applications/min, and the loads were 40, 80, 120, and 160 lb.

The tests on the samples were essentially multistage tests that followed the work of Silver and Park (19); the lowest load was applied first, starting with the lowest frequency. After 100 applications of load, the frequency was increased and the load was kept constant. After all four frequencies had been used, the load was increased to the next higher value and the procedure was repeated until failure occurred. During load and frequency adjustments, the machine was stopped. The upper limit of load application was 160 lb at 24 applications/min.

For a given load and load frequency, the only parameter measured was diametral strain (see Figures 2 and 3). As we point out elsewhere (13), the stresses at any point within the sample body are dependent on diametral deformation and load, and diametral strain was established as a dependable failure criterion.

FAILURE CRITERIA

In the graphs for diametral strain versus frequency/load shown in Figure 4, three zones can be identified:

1. The initial zone is limited to the first 50 cycles of the lowest (40-lb) load applied at 6 applications/min. It was during this stage that the seating of the sample occurred, and that, in certain cases, seems to have caused appreciable sample disturbance.
2. The intermediate zone lies between the initial and the failure zones.

3. The failure zone is the zone in which extensive cracking developed and failure of the sample occurred.

A number of specimens showed less than the minimum measurable strength in that they broke at the first impact. For others, the initial and failure zones were the same (see curve A in Figure 5). It was hard to determine the value of the failure diametral strain for such specimens. In some instances, the initial zone was immediately followed by the failure zone (curve B). Some samples, especially those in a dry state, did not fail within the range of testing (curve C). For such samples there is no failure zone. In general, however, for most of the samples the three zones could be identified.

Whenever there was an identifiable intermediate zone, failure was taken at the last "bending point" (see Figure 6) before rupture of the sample took place. In the case of the failed specimens, it was observed that the diametral strain had increased more than 0.25 percent over a 50-cycle period. Thus, this value was also accepted as a criterion for failure.

OBSERVATIONS

Even after one wet-dry cycle, the moisture contents of the samples fell below their optimum moisture contents. The "end-of-cycle" moisture contents were not much different for a given shale and its lime admixtures (see Table 2). On drying, some samples showed cracks. The cracking was less apparent for the samples stabilized with lime. In fact, for those that contained 4 percent or more lime, it was not detectable.

Figure 4. Zones in diametral strain.

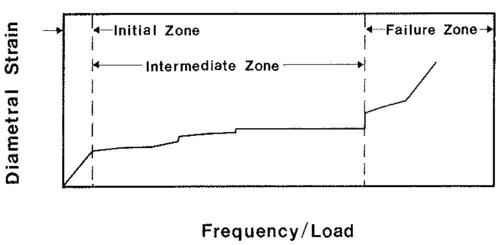


Figure 5. Typical diametral-strain curves.

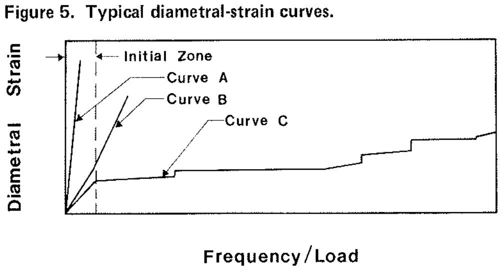
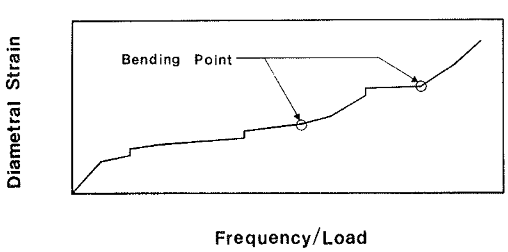


Figure 6. Identification of bending points in diametral strain.



Effect of Load and Load Application

The data collected for samples in the dry condition that withstood all four loads and all four frequencies of testing are given in Table 3. Even with the

Table 2. Optimum average molding.

Shale Number	Amount of Lime (%)	Optimum Moisture Content (%)	Average Molding Moisture Content (%)	End-of-Cycle Moisture Content (%) Humid[a,b]	Dry[a]
13	0	18.3	18.7	9.3	1.8
	1	21.6	22.0	9.3	1.4
	2	21.6	21.9	9.2	1.7
	4	21.6	21.6	9.2	1.9
	6	22.8	22.5	10.7	2.5
15	0	13.2	14.1	5.4	1.1
	1	15.2	16.0	4.5	1.0
	2	16.1	16.9	5.5	1.6
	4	16.3	17.2	6.2	1.5
	6	16.6	17.4	6.8	1.3
21	0	21.5	21.8	11.4	2.3
	1	24.0	24.4	11.0	2.1
	2	25.5	25.9	10.4	2.2
	4	25.8	25.9	10.7	2.0
	6	25.3	25.2	11.0	2.0
24	0	18.0	18.0	6.7	0.8
	1	18.2	18.4	7.1	1.0
	2	20.0	19.2	6.7	0.8
	4	20.2	20.5	7.7	1.1
	6	21.5	21.0	7.9	0.8

[a]State of specimen at time of testing.
[b]Does not include samples in humid state not subjected to at least one drying.

Table 3. Effect of load on diametral strain of some shale and shale-lime samples in the dry condition.

Shale Number	Amount of Lime (%)	No. of Wet-Dry Cycles	Diametral Strain ($\times 10^{-4}$) 40-lb Load	80-lb Load	120-lb Load	160-lb Load
13	0	0	25	0	13	25
		30	12	13	25	12
			0	13	13	38
		50	0	12	12	24
			0	11	12	24
	1	0	12	13	0	0
			0	0	0	13
		15	12	37	37	0
		30	0	0	0	13
			0	0	12	13
		50	12	12	13	12
	2	0	0	12	37	12
			12	12	12	12
		50	12	12	0	25
	4	0	0	12	0	12
	6	0	0	12	35	0
			24	24	25	12
21	0	0	25	12	12	25
			0	13	28	38
		15	12	13	25	12
			0	25	37	12
		30	12	24	24	12
			12	12	0	36
		50	37	12	0	37
			13	38	13	13
24	0	0	0	0	12	36
			13	24	49	49
		15	0	36	12	0
			0	12	0	24
		30	0	0	0	33
			0	12	12	25
		50	0	12	12	25
			0	24	12	12

Figure 7. General relation between diametral strain and load.

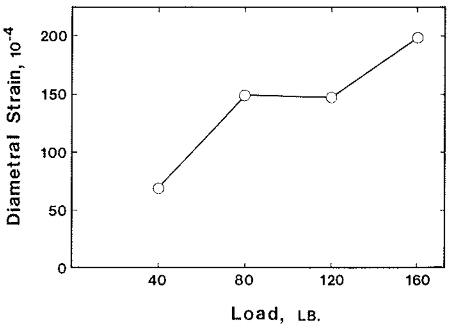
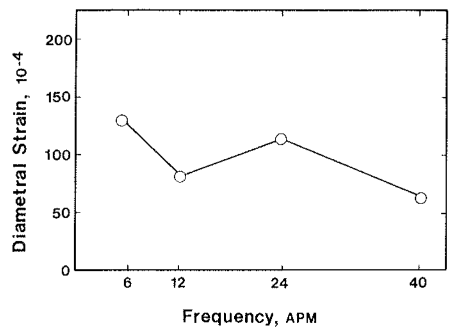


Figure 8. General relation between diametral strain and increasing frequency of load application.



wide variation in strain values, it could be stated that, generally, the greater the load, the greater was the diametral strain (see Figure 7). The difference between the 80-lb and the 120-lb values is very small and appears to be attributable to the fact that most of the bending points are located within this range.

The strains increased progressively with increase in the number of load applications. Within the intermediate zone, for a given load, initially the strain increased with a decreasing rate. This trend continued unless failure ensued. Then the strain per cycle started to increase with increasing number of load applications.

The effect of increasing frequency at the same load appears to be significant (see Table 3 and Figure 8). The lower frequency caused greater change in diametral strain and, as the frequency of load application increased, the change in strain decreased unless sample failure ensued. In the latter case, the change in strain increased with increased frequency of load application.

Shale 13

Shale 13, classified A-7-6, contained 48 percent clay-sized particles; montmorillonite was the predominant mineral. Addition of even 1 percent lime reduced the diametral strain significantly. Beyond the 1 percent value, there were no significant changes either in the strain magnitude or in the capacity of the shale to take greater stresses. The effect of wet-dry cycles was significant on the raw shale but not on the treated one. Very little difference was noted in the deformation characteristics of the sample after 15 wet-dry cycles. Increasing the amount of lime and the number of weathering cycles only made the sample more brittle. There was not much difference between the dry and the humid samples from the point of view of diametral strain; however, the dry samples took greater loads and a greater number of load applications. During the early stages of weathering, values of diametral strain showed an increase as the lime treatment level approached 6 percent.

Shale 15

Shale 15, classified A-1-b in its natural state, is associated with high bearing capacity and high permeability. When it is ground up to pass the no. 10 sieve, its granular structure is broken down and the shale is converted to a material of low strength. Only when the lime addition reached the 6 percent level did the shale show some measurable strength, and even then the samples failed in the 40-lb load range. At 6 percent lime level, the samples became more elastic with increasing number of wet-dry cycles in that they exhibited smaller diametral strains for the same load.

Shale 21

Shale 21, classified A-6, contains 26 percent clay, predominantly of montmorillonite-illite mixed-layer minerals. The samples that had not been subjected to any wet-dry cycles exhibited measurable strength only when the lime addition exceeded 1 percent. Then the samples that contained a greater amount of lime took greater diametral strains. Since there had hardly been enough time for shale-lime reaction products to form, the strength gain could only be attributed to the flocculation of clay. The subsequent application of wet-dry cycles seemed to have destroyed whatever bond formation took place and, even after 50 wet-dry cycles, the samples did not show any measurable strength.

Increasing amounts of lime made the shale brittle, and the samples failed not only at smaller diametral strains but also at smaller loads. The effect of wet-dry cycles on the raw shale was significant: The shale became more brittle. However, as the amount of lime increased, the effect of wet-dry cycles became less significant. At 6 percent lime, the samples exhibited increasingly elastic behavior with increasing number of wet-dry cycles in that they showed gradually smaller diametral strains for the same load.

Shale 24

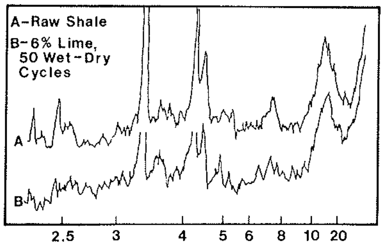
Shale 24, classified A-4, contains 14 percent clay and consists essentially of illite and kaolinite particles. It is basically not a lime-reactive shale. The flocculation attributable to lime seemed to provide some plasticity in the initial stages, but the effect was completely lost when 30 wet-dry cycles had been applied and the sample was in the humid state. Only the untreated shale showed any measurable strength for 30 and 50 wet-dry cycles.

In the initial stages, the addition of lime made the dry samples more plastic. Later, however, the strength characteristics improved and the shale gradually exhibited more elastic behavior.

X-Ray Diffraction Analysis

The diffraction patterns for the shales studied did not indicate any significant changes with the wet-dry cycles for the raw material (see Figure 9). Patterns for the shale combinations that contained 2 percent lime showed the presence of little or no unreacted lime. Calcite peaks could be observed for all wet-dry cycles in most cases. Peaks for both the unreacted lime and calcite generally became more pronounced for the mixes that contained 6 percent lime. The dry samples indicated the presence of more unreacted lime than did the humid samples; they also showed stronger calcite peaks, which suggests the presence of a greater amount of calcium carbon-

Figure 9. Selected X-ray diffraction patterns for shale 13 and its lime combinations.



ate. In general, the diffraction patterns suggested a reaction between lime and kaolinite, some reaction between lime and illite, but a very limited reaction between lime and montmorillonite.

DISCUSSION OF RESULTS

The addition of lime to shale causes flocculation of clay particles. The strength of the soil-lime matrix depends on the "house-of-cards" structure of clays and friction between various particle edges and surfaces. Castro and Christian (20) found that the structure of clay is not significantly affected by the action of applied cyclic loading. In the presence of moisture, the shale-lime reactions would proceed with time and various reaction products would be formed. The strength of the matrix would now be affected by these reaction products as well. The repeated introduction and removal of moisture cause destruction of the bonds between the particles. Humidification of the sample introduces moisture in the shale-lime system, and now the strength of the sample depends not only on the bonds between the particles and the strength of these bonds but also on the porosity of the matrix and the moisture-imbibing characteristics of the shale-lime mass.

Shales 15 and 24 each contain only 14 percent clay-sized particles; their initially porous structure seems to become more porous with the application of wet-dry cycles and with the addition of lime. Even a small amount of moisture resulting from humidification seems to provide a layer of water thick enough to bring about the failure of the sample at a small load. The sample that contained 6 percent lime, however, was able to carry a greater load while showing nonplastic characteristics. The X-ray diffraction patterns for this sample indicated the presence of many shale-lime reaction products, especially the tobermorite gel; thus, the strength of the sample is attributable to bond formation by the gel. This strength is still too little and, when the sample was subjected to humidification, it showed some, but not much, resistance to stress.

The strength of the raw shale samples is obviously attributable to their less porous structure. The same reasoning seems to hold for the strength of the samples of shales 13 and 21 that contained 1 or 2 percent lime. The differences between shales 13 and 21 are caused by the difference in their clay contents: Shale 13 has 48 percent clay whereas shale 21 has only 25 percent.

Shrinkage, or reduction in volume, was significant in shales 13 and 21, which have higher percentages of clay. In spite of the reduced load-carrying area caused by cracking, these shales were capable of withstanding the total range of loads and load applications. In the lime-treated shales, however, on drying the existing house-of-cards structure prevents the particles from coming close together. Thus, the shale-lime matrix remains porous and exhibits lower strength. In addition, the presence of small amounts of unreacted lime and calcite particles provides additional areas of discontinuity from which cracks may initiate or propagate without requiring much energy. This may explain why some samples that contain high percentages of lime exhibited lower strengths for the same wet-dry-cycle treatment. The porous structure of lime-stabilized shales, however, inhibits the effects of wet-dry cycles and thus provides a more desirable material.

Beyond the initial stage, the chemical reactions between shale and lime are dependent on time and temperature parameters. Because of the greater surface area available for shale-lime reaction, montmorillonite in clay should provide early strength development, and this indeed is the case. Samples of lime-treated shale 13 are generally capable of withstanding greater loads and number of load applications than the samples of lime-treated shale 21. In this case, shale 21 is, in turn, better than lime-treated shale 24. Thus, from the present study, it can be concluded that lime stabilization seems to be more effective with the shale that contains a higher percentage of clay and the shale that has a greater montmorillonitic clay content. It must be pointed out that this observation is based on the study of only four shales that varied widely in character.

Addition of lime was largely ineffective in stabilizing shale 15. For shales 13, 21, and 24, addition of even 1 percent lime was sufficient to alter their plastic nature into brittle behavior. Addition of 2 percent lime made these shales nonplastic, but until this percentage was reached a great portion of the lime seems to have reacted with the shale, since X-ray patterns indicated the presence of little or no unreacted lime. At the 6 percent lime treatment level, the strength and plasticity characteristics are not significantly different from the same characteristics at the 4 percent lime treatment level for these three shales. However, the X-ray diffraction patterns for these shales and mixes with 6 percent lime subjected to 50 wet-dry cycles showed strong calcite and lime peaks. This indicated not only that much more time was required for the lime to react with the shale but also that some of the lime must be wasted as calcium carbonate, which imparts but little strength to the shale-lime mix. Thus, for highway construction purposes (the focus of this study), the optimum amount of lime seems to be between 2 and 6 percent and the exact amount to depend on the content and the type of clay in the shale selected for stabilization.

CONCLUSIONS

1. In split tension tests, where loads are increased in discrete magnitudes and not in a continuous manner, diametral strain is a very good criterion by which to determine failure zones. Once the failure zones are identified, limiting stresses can be computed and used to evaluate the strength characteristics of different shales.

2. The presence of unreacted lime in the shale-lime mix tends to cause brittleness. Samples exhibit increasingly elastic behavior as the amount of unreacted lime decreases.

3. After 50 wet-dry cycles, in all shales much of the lime is used for the shale-lime reaction at the 2 percent lime level. However, at the 6 percent lime level, the lime is only partially used and X-ray diffraction patterns reveal the presence of unreacted lime and calcite.

4. Humidification of samples causes loss of strength. Thus, the dry samples withstand greater loads and a greater number of load applications than the moist samples. The effect of humidification on loss of strength was greatest for shale 15 and its lime mixes. This effect decreased with shale 24, shale 21, and shale 13, in that order, which shows the effect of increasing clay contents in soils.

5. The destructive effect of wet-dry cycles is greater on untreated shales than on lime-treated shales. The greater the percentage of lime in the shale-lime mix, the less is the effect of the wet-dry cycles on the mix.

6. Weathering of untreated shale and the addition of lime to it both reduce the plastic characteristics of shale and introduce brittleness to it.

7. The addition of lime significantly alters the character of shale. However, the extent to which the shale-lime reaction proceeds under the simulated conditions of wetting and drying depends on the number of wet-dry cycles.

ACKNOWLEDGMENT

This investigation was conducted as part of a study by the Oklahoma Department of Transportation. The contents and conclusions do not necessarily reflect the official views or policies of the Federal Highway Administration or the Oklahoma Department of Transportation.

REFERENCES

1. C.L. Nalezny and M.C. Li. Effect of Soil Structure and Thixotropic Hardening on the Swelling Behavior of Compacted Clay Soils. HRB, Highway Research Record 209, 1967, pp. 1-22.
2. E.K. Sauer and C.L. Monismith. Influence of Soil Suction on Behavior of a Glacial Till Subjected to Repeated Loading. HRB, Highway Research Record 215, 1968, pp. 8-23.
3. F. Moavenzadeh and R.A. Carnaghi. Viscoelastic Response of Sand Asphalt Beams on Elastic Foundations Under Repeated Loading. Proc., Assn. of Asphalt Paving Technologists, Ann Arbor, MI, Vol. 35, 1966, pp. 514-528.
4. H.B. Seed and others. Prediction of Flexible Pavement Deflections from Laboratory Repeated Load Tests. NCHRP, Rept. 35, pp. 102-115.
5. P.C. Pretorius and C.L. Monismith. Fatigue Crack Formation and Propagation in Pavements Containing Soil-Cement Bases. HRB, Highway Research Record 407, 1972, pp. 102-115.
6. H.G. Larew and G.A. Leonards. A Strength Criterion for Repeated Loads. Proc., 41st Annual Meeting, HRB, Vol. 41, 1962, pp. 529-556.
7. H. Hong. Fatigue Characteristics of Flexible Pavements. Journal of Highway Division, American Society of Civil Engineers, New York, Vol. 93, No. HW1, 1967, pp. 43-67.
8. F.L.L.B. Carneiro and A. Barcellos. Concrete Tensile Strength. International Union of Testing and Research Laboratories for Materials and Structures, Paris, Bull. 13, 1953, pp. 97-123.
9. J.B. Metcalf and S. Frydman. A Preliminary Study of the Tensile Stresses in Stabilized Soil Pavements. Proc., Australian Road Research Board, Melbourne, Vol. 1, Part 2, 1962.
10. S. Timoshenko and J.N. Goodier. Theory of Elasticity. McGraw-Hill, New York, 1951.
11. M.M. Frocht. Photoelasticity. Wiley, New York, Vol. 2, 1961.
12. R. Peltier. Theoretical Investigation of the Brazilian Test. International Union of Testing and Research Laboratories for Materials and Structures, Paris, Bull. 19, 1954, pp. 26-69.
13. S. Kumar, M. Annamalai, and J.G. Laguros. A New Analytical Approach to Split Tensile Strength of Pavement Materials. Proc., 26th Annual Highway Geology Symposium, Idaho Department of Transportation, Boise, 1975, pp. 187-204.
14. M. Annamalai and others. A Study of Three Oklahoma Shales. Department of Civil Engineering and Environmental Science, Univ. of Oklahoma, Norman, 1970.
15. G.H. Hilt and D.T. Davidson. Lime Fixation in Clayey Soils. HRB, Bull. 262, 1960, pp. 20-32.
16. C. Ho and R.L. Handy. Effect of Lime on Electrokinetic Properties of Bentonites. Proc., 12th National Conference on Clays and Clay Minerals (England), Pergamon Press, Oxford, England, 1964, pp. 267-280.
17. L.F. Sheerar. The Clays and Shales of Oklahoma. Oklahoma A&M College, Stillwater, Vol. 3, No. 5, 1932.
18. J.G. Laguros. Predictability of Physical Changes of Clay-Forming Materials in Oklahoma. Research Institute, Univ. of Oklahoma, Norman, Project 1677, Final Rept., 1972.
19. M.L. Silver and T.K. Park. Testing Procedure Effects on Dynamic Soil Behavior. Journal of Geotechnical Engineering Division, American Society of Civil Engineers, New York, Vol. 101, No. GT10, 1975, pp. 1061-1083.
20. G. Castro and J.T. Christian. Shear Strength of Soils and Cyclic Loading. Journal of Geotechnical Engineering Division, American Society of Civil Engineers, New York, Vol. 102, No. GT9, 1976, pp. 1171-1184.

Publication of this paper sponsored by Committee on Engineering Geology.