Split-Tensile Strength of Lime-Stabilized Soils

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The tensile strength characteristics of lime-soil mixtures are of considerable importance in any type of rational pavement design procedure. There is little information at this time concerning the tensile strength properties of lime-soil mixtures since the majority of the reported work has dealt primarily with compressive strength.

This paper presents the principles of the split test (a diametral compression test for determining tensile strength), an evaluation of the test, and the application of the test procedure to lime-soil mixtures. Split-tensile and compressive strength test results are given for 11 soils.

The results show that lime-soil mixtures develop substantial tensile strength and that the split-tensile strength is closely correlated with unconfined compressive strength. Factors such as soil type, lime treatment, and length of curing period influence the magnitude of the split-tensile and compressive strengths, but do not affect the ratio between the strengths. The investigation clearly indicates that the split test has considerable merit as a test procedure for evaluating the tensile strength properties of lime-soil mixtures.

•AN INCREASING emphasis has been placed on the use of stabilized highway materials in recent years. Through the use of stabilizing agents, low-quality materials can be economically upgraded to the extent that they may be effectively utilized in the pavement structure. Stabilizing agents also improve natural materials of medium and high quality, and materials ranging from well-graded crushed stones to highly plastic clays have been successfully stabilized. Although these natural materials are very dissimilar in many respects, they characteristically have low tensile strengths. Stabilization with lime, lime-fly ash, portland cement, etc., imparts a tensile strength to these materials. The compressive strength of the material is also considerably increased. Generally speaking, the compressive strengths are several times greater than the tensile strengths. The stress-strain curves for a typical stabilized material are nearly linear to failure with very little inelastic yielding (so-called brittle behavior). The modulus of elasticity of the stabilized material is normally several times greater than that for the natural material. Figure 1 illustrates stress-strain curves for a typical lime-soil mixture.

Stabilized highway materials are generally incorporated into the pavement structure as base courses, subbases, or subgrades. Burmister (1) and others have shown that in a layered system of elastic materials, where the overlying layers have higher moduli of elasticity than underlying layers, tensile stresses are developed at the interfaces between the layered materials. This layered system analysis is commonly presumed to be applicable to a highway pavement where the stiffer materials are used in the upper layers. Since many stabilized materials are relatively weak in tension, any type of rational design procedure must take into account their tensile strength. Unfortunately, much of the research and development work concerning stabilized

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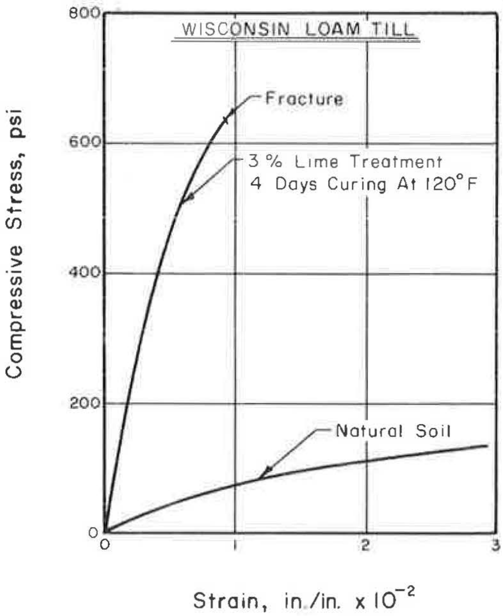

Figure 1. Typical stress-strain curves.

materials has utilized unconfined compression and triaxial testing methods and there is a lack of basic knowledge concerning their tensile strength properties.

If these stabilized materials are to be used in an efficient and economical manner, it is imperative that a satisfactory test method be developed for determining their tensile strength. The investigation described in this report is concerned with preliminary studies of the split-tensile strength of lime-stabilized soils.

SPLIT TEST

The split test was developed independently by Carneiro and Barcellos (2) in Brazil and Akazawa (3) in Japan. The test procedure has been primarily utilized to evaluate the tensile strength of concrete (ASTM Designation: C 496-64T) but it has many merits as a tensile test and could easily be adapted for stabilized highway materials, such as lime-soil mixtures, which exhibit a brittle-type behavior and have relatively low tensile strengths.

The split test is conducted by placing a cylindrical specimen horizontally between two loading surfaces and loading the specimen along two opposite generatrices as shown in Figure 2. For brittle materials weak in tension, the specimen fails in tension along the loaded diameter, A-B, of the cylinder.

Theory of Split Test

The theoretical solution of the split test is based on the theory of elasticity. Frocht's equations (4) for the stresses at a point in terms of rectangular coordinates (Fig. 3) are as follows:

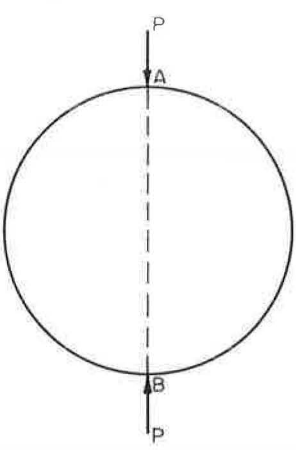

Figure 2. Split test.

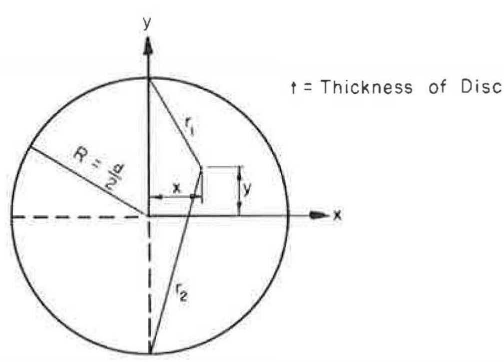

Figure 3. Coordinate system.

$$\sigma_x = \frac{-2P}{\pi t}\left[\frac{(R-Y)\,X^2}{r_1^4} + \frac{(R+Y)\,X^2}{r_2^4} - \frac{1}{d}\right] \tag{1}$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{(R-Y)^3}{r_1^4} + \frac{(R+Y)^3}{r_2^4} - \frac{1}{d}\right] \tag{2}$$

$$\tau_{xy} = \frac{2P}{\pi t}\left[\frac{(R-Y)^2\,X}{r_1^4} - \frac{(R+Y)^2\,X}{r_2^4}\right] \tag{3}$$

where

σ_x, σ_y, τ_{xy} = stress components with respect to rectangular coordinates;
x, y = rectangular coordinates;
P = load applied to specimen;
t = thickness of cylindrical specimen;
d = diameter of cylindrical specimen;
R = radius of cylindrical specimen; and
r_1, r_2 = location coordinates.

For the horizontal diameter of the cylinder, the X-axis, Y = 0, $r_1 = r_2 = X^2 + R^2$, and the stress equations simplify to:

$$\sigma_x = \frac{2P}{\pi td}\left[\frac{d^2 - 4X^2}{d^2 + 4X^2}\right]^2 \tag{4}$$

$$\sigma_y = \frac{-2P}{\pi td}\left[\frac{4d^4}{(d^2 + 4X^2)^2} - 1\right] \tag{5}$$

$$\tau_{xy} = 0 \tag{6}$$

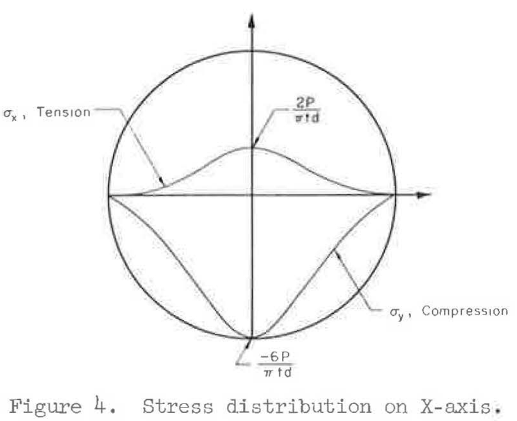

Figure 4. Stress distribution on X-axis.

The vertical stress, σ_y, along the X-axis is always a compressive stress and varies from a maximum at the center to zero at the circumference. At the center, the magnitude of σ_y is $\frac{-6P}{\pi td}$ and the accompanying horizontal stress, σ_x, is a tensile stress equal to $\frac{2P}{\pi td}$. This indicates that the material being tested must have a compressive strength at least three times its tensile strength if it is to fail in tension. The stress distribution along the X-axis is shown in Figure 4.

For a vertical plane through the center of the cylinder along the Y-axis, Frocht's equations for the stresses reduce to:

$$\sigma_x = \frac{2P}{\pi td} \tag{7}$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{2}{d - 2y} + \frac{2}{d + 2y} - \frac{1}{d}\right] \tag{8}$$

$$\tau_{xy} = 0 \tag{9}$$

The horizontal tensile stresses, σ_x, along the vertical plane have a constant value of $\frac{2P}{\pi td}$ and the vertical compressive stresses vary from $\frac{-6P}{\pi td}$ at the center of the disc to ∞ at the end of the loaded diameter. These high compressive stresses at the loading points will cause failure, thus preventing failure in the central portion of the vertical diameter of the specimen due to tensile stresses (5). Photoelastic studies have shown that the point of maximum stress concentration can be moved away from the load point by applying a distributed load through a loading strip. In addition, the distributed load changes the σ_x stresses in the vicinity of the loading strip to compressive stresses, placing the material in the immediate area beneath the loading strips under the influence of compressive stresses (5). Most brittle materials are fairly strong under such a state of stress and, therefore, the specimen fails in tension in the central part of the loaded diameter.

A schematic representation of the test specimen and the loading strips is shown in Figure 5. The utilization of the loading strips somewhat alters the stress distribution in the specimen. According to Wright (6) the horizontal stress distribution σ_x along the vertical diameter is closely approximated by:

$$\sigma_x = \frac{-2P}{\pi td}\left[1 - \frac{d}{2a}(\alpha - \sin\alpha)\right] \tag{10}$$

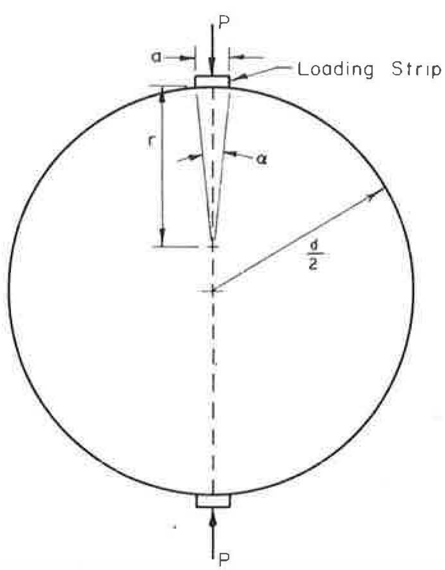

Figure 5. Loading strips.

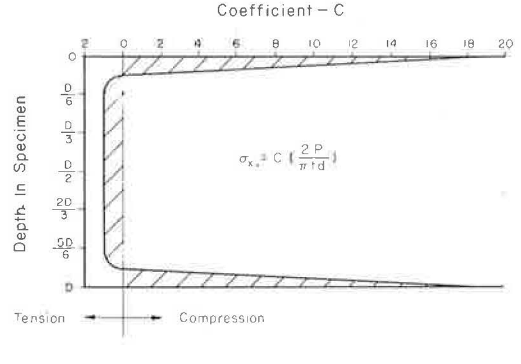

Figure 6. Horizontal stress distribution on Y-axis for loading strip width equal to d/12.

provided the width of the loading strip, a, is less than d/10. The resulting horizontal stress distribution on the vertical diameter is shown in Figure 6. Peltier's (7) work on loading strips indicated that the tensile stresses remain uniform over a reasonable proportion of the diameter if the loading strip width is less than d/5. It thus appears that when loading strips of the widths suggested above are utilized, the tensile stresses over a substantial portion of the loaded diameter have a value of approximately $\frac{2P}{\pi td}$.

Loading strip characteristics and cylinder size are two factors which have received considerable attention. According to Mitchell (5), loading strip characteristics such as width, thickness, and type of material, affect the type of rupture but not the tensile stress at failure. Rudnich et al. (8), in their evaluation of the split test for use with ceramic materials, concluded that of the various types of failure that may occur in the split test, only a shear-type failure is unsatisfactory for determining the split-tensile strength. It is recommended that the loading strip be of a rather pliable material which can conform to any surface irregularities in the specimen.

The effect of cylinder size on the split-tensile strength of concrete was investigated by Carneiro and Barcellos (2), Akazawa (3), and Wright (6). All investigators agreed that the size of cylinder had very little effect on the test results but that larger cylinders gave results with a smaller coefficient of variation.

Evaluation of Split Test

Since the theoretical analysis of the split test is based on the theory of elasticity (4), it is logical that the test would provide a good indication of the tensile strength of materials which behave elastically to failure. Rüsch and Vigerust (9) used the split-tensile test to evaluate the tensile strength of concrete and concluded that the tensile splitting strength is near the true strength, especially for high-strength concrete which is more nearly an elastic material.

Although the tensile stress distribution along the vertical diameter of a test specimen is approximately constant, the complete stress distribution on the diameter is quite complicated. Bawa (10) pointed out that the vertical stress, σ_y, has a large variation along the vertical diameter and, therefore, the stress difference, $\sigma_y - \sigma_x$, is highly variable.

With brittle materials, strain as well as stress may be important in determining the tensile strength of the material. Since there is not an uniaxial state of stress in the split test, Poisson's ratio may have an effect on the indicated tensile strength. Bawa (10) and Ramesh and Chopra (11) emphasized that Poisson's ratio cannot be ignored in a biaxial state of stress such as exists in the split test. If strain is an important factor in determining the tensile strength, higher strengths will be obtained if Poisson's ratio is low and lower tensile strengths will be obtained if Poisson's ratio is high. Bawa (10) observed this strain influence for tests on cement mortar.

A major disadvantage of the split test is that it does not resemble the actual field service conditions of many materials which exhibit a "slab type" of behavior. Many engineers prefer a flexural test for evaluating such materials.

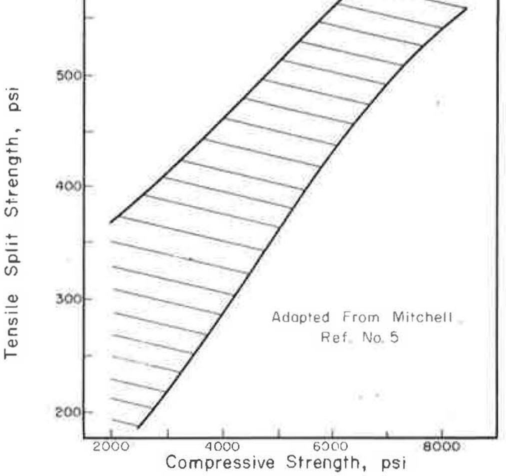

Figure 7. Tensile strength vs compressive strength for concrete cylinders.

Correlation With Other Test Methods

Mitchell (5) concluded, as a result of his literature review, that the split test gives tensile strengths less than the flexure test and greater than the briquet test with the split test having better reproducibility than the other forms of tension tests. The literature indicates that the split-tensile strength of concrete is approximately 50 to 60 percent of the flexural strength. Many investigators attempted to establish a general relation between split-tensile and unconfined compressive strengths of concrete. Figure 7 shows the general trends of several of these investigations. The hatched area encompasses the range of test results reported.

Summary

The split test appears to be a test method that can be easily adapted for evaluating the tensile strength of stabilized highway materials. Theoretical and experimental data indicate that a relatively uniform tensile stress distribution exists on the major portion of the loaded diameter of the test cylinder. This uniform stress distribution cannot be obtained with any other type of tensile test procedure currently being used to evaluate stabilized highway materials.

INVESTIGATION

The objectives of this investigation were to evaluate the split-tensile strength characteristics of cured lime-soil mixtures and the relation of split-tensile strength to unconfined compressive strength. The factors of lime type and percentage, curing period, and soil type were varied to determine their influence on split-tensile strength characteristics.

Materials

Eleven soils of diverse physical and mineralogical properties and three types of lime were used in the study.

Soils. —Loess-derived, Wisconsinan till-derived, and Illinoian till-derived soils were used in the investigation. The soils were sampled in the field, air dried in the laboratory, pulverized, screened to remove the plus No. 4 material, and stored for subsequent use. Selected physical, engineering and mineralogical properties were determined for the soils. A summary of selected soil properties is presented in Table 1.

Limes. —High-calcium hydrated lime, monohydrated dolomitic lime, and a by-product high-calcium lime were used as stabilizing agents. The limes were produced by conventional lime manufacturing procedures, with the exception of the by-product lime which is obtained from the manufacture of acetylene gas. Properties of the limes are presented in Table 2.

Mixing

Proper quantities of lime and air-dry soil were thoroughly blended in a Lancaster mixer. The amount of water required to bring the lime-soil mixture to optimum

TABLE 1

SELECTED SOIL PROPERTIES

Designation	Type	Source	AASHO Classification	$< 2\mu$ Clay (%)	L. L. (%)	P. I.	Predominant Clay Mineral
1	Accretion gley	Effingham Co.	A-6 (7)	18	33.7	18.4	Montmorillonite
2	Accretion gley-1	Sangamon Co.	A-6 (10)	25	32.5	14.2	Mixed layer
3	Accretion gley-2	Sangamon Co.	A-6 (12)	26	35.9	21.9	Mixed layer
4	Bryce B	Iroquois Co.	A-7-6 (18)	52	53.1	28.8	Illite
5	Cowden B	Montgomery Co.	A-7-6 (19)	34	53.9	31.4	Montmorillonite
6	Cowden C	Montgomery Co.	A-6 (9)	20	32.4	12.6	Montmorillonite
7	Cowden B	Randolph Co.	A-7-6 (19)	38	54.2	32.5	Montmorillonite
8	Illinoian B	Sangamon Co.	A-6 (11)	29	37.2	19.2	Mixed layer
9	Illinoian till	Sangamon Co.	A-6 (6)	14	25.5	11.0	Illite
10	Illinoian till	Effingham Co.	A-6 (6)	17	24.6	11.7	Illite
11	Ottawa A-6	LaSalle Co.	A-6 (8)	25	25.2	10.8	Illite

TABLE 2

PROPERTIES OF LIMES

Lime Designation	Type	$\%Ca(OH)_2$	$\%MgO$	$\%Mg(OH)_2$	% Passing No. 325 Sieve
A	High-calcium hydrated	96	--	--	95
B	Monohydrated dolomitic	58.8	33.3	1.7	85
C	By-product high-calcium hydrated	96	--	--	76

moisture content was then added and mixing was continued for approximately 2 min. Following mixing, the lime-soil mixture was covered and allowed to stand for 1 hr before specimens were compacted.

Sample Preparation

A series of sixteen 2-in. diameter by 4-in. specimens were prepared for each test condition, i.e., lime type, soil type and curing period. The specimens were molded in three equal layers with each layer receiving a compactive effort of 20 blows of a 4-lb hammer dropping 12 in. Each layer was scarified to provide bond between the adjacent layers. After proper trimming, the specimens were extruded from their molds and cured.

All specimens were compacted at approximately optimum moisture content as determined by a moisture-density test. The moisture-density test was conducted in a manner similar to AASHO Designation: T 99-57, except that 2-in. diameter by 4-in. molds were used and the compactive effort was applied through 20 blows of a 4-lb hammer having a 12-in. drop. This compactive effort produces maximum dry densities and optimum moisture contents similar to those obtained from testing by Method A of AASHO Designation: T 99-57 for moisture-density relations of soils. Optimum moisture contents and maximum dry densities for the various lime-soil mixtures are presented in Table 3.

Curing

After compaction, trimming, and extrusion, the specimens were placed in 1-gal metal cans and the can lids were sealed with Perma-Tex. The sealed cans were then placed in a 120 F curing cabinet for periods ranging from 1 to 75 days.

TABLE 3

COMPACTION PROPERTIES OF LIME-SOIL MIXTURES

Soil	Lime		Max. Dry Dens. (pcf)	Opt. Moisture (%)
	Type	%		
1	B	7	114.3	15.3
2	A	7	106.0	17.0
3	A	5	112.0	15.8
4	A	5	97.3	25.8
5	B	7	95.5	24.5
6	C	3	112.6	15.0
7	B	5	98.2	23.0
8	C	5	108.5	17.8
9	A	3	121.0	13.0
10	C	3	124.3	11.5
11	A	3	119.6	14.3
	A	5	116.4	15.0
	B	3	118.5	13.4
	B	5	117.4	14.6
	C	3	119.5	14.2
	C	5	116.0	15.1

Testing Procedure

At the termination of the curing period, eight alternate specimens (1, 3, 5, etc., or 2, 4, 6, etc.) were tested in unconfined compression and the remaining eight were tested in split tension. A Riehle hydraulic machine with a strain rate of 0.05 in./min was used for all testing. The unconfined specimens were tested in the usual manner. Loading strips 0.25 in. wide and approximately 0.07 in. thick were used with the split-tension specimens. The 0.25-in. wide loading strips gave a width to diameter (a/d) ratio of $\frac{1}{8}$. Specimen moisture content at the time of testing was determined and found to be approximately optimum.

Testing Program

The testing program was divided into two phases. Only one soil type was used in Phase I, but 11 soils were used in Phase II.

Phase I. —This part of the program involved only the A-6 subgrade soil, a calcareous Wisconsinan till, from the site of the AASHO Road Test near Ottawa, Ill. Lime type, lime percentage, and curing periods were varied over wide limits to evaluate the influence of such variations on split-tensile strength and the ratio of split-tensile strength to unconfined compressive strength. Lime treatments, curing periods, and test results are presented in Table 4.

Phase II. —Eleven different soils were used in Phase II. The prime objective was to determine the effect of soil type on the ratio of split-tensile to unconfined compressive strengths. The soils were stabilized with the amount of lime required to produce maximum 28-day compressive strengths (73 F curing temperature and sealed container curing). These optimum lime requirements were established by the author in a previous investigation. The type of lime used (A, B, C) and the curing period (7, 15, 20, 30, 50 days at 120 F) were assigned to the soils in a random fashion. Lime treatments, curing periods, and test results are shown in Table 5.

TABLE 4

PHASE I TEST RESULTS
(Ottawa A-6 Soil)

Lime Type	Lime %	Curing (days)	q_u (psi)[a]	Split[a] Strength (psi)	Split/q_u
A	3	1	173	20.8	0.12
		5	270	40.3	0.15
		10	434	62.3	0.14
		20	559	72.7	0.13
		30	410	58.0	0.14
		40	557	67.3	0.12
		50	703	90.1	0.13
		75	1033	157.0	0.15
A	5	1	201	28.2	0.14
		5	231	26.0	0.11
		10	527	75.4	0.14
		20	620	69.3	0.11
		40	790	85.4	0.11
		50	1298	184.0	0.14
		75	959	143.0	0.15
B	3	1	199	25.1	0.13
		5	349	47.2	0.14
		11	494	60.1	0.12
		20	615	65.4	0.11
		30	763	96.3	0.13
		40	778	87.3	0.11
		50	764	80.4	0.11
		75	897	107.0	0.12
B	5	1	196	19.4	0.10
		5	362	42.2	0.12
		10	371	46.5	0.13
		15	529	70.7	0.13
		20	823	124.0	0.15
		31	1136	160.0	0.14
		40	1236	183.0	0.15
		75	1474	178.0	0.12
C	3	3	323	35.4	0.11
		7	449	51.6	0.11
		20	757	108.0	0.14
		31	815	115.0	0.14
		53	875	129.0	0.15
		75	1217	157.0	0.13
C	5	3	310	40.9	0.13
		7	497	60.7	0.12
		20	646	96.9	0.15
		30	712	99.6	0.14
		40	857	99.5	0.12
		50	862	107.0	0.12
		75	1582	200.0	0.13

[a]Average of eight specimens.

PRESENTATION AND ANALYSIS OF TEST RESULTS

Phase I

Examination of the test results from Phase I (Table 4) shows that lime-soil mixtures possess substantial tensile strength. The split-tensile strengths range from approximately 20 to 200 psi. Figure 8 illustrates the typical variation in compressive strength, split-tensile strength, and the ratio of split-tensile strength to compressive strength (S_T/q_u) for different curing periods. Longer curing periods generally increased compressive strength and split-tensile strength, regardless of lime type and percentage, but the S_T/q_u ratio varied only from 0.10 to 0.15. It is apparent that split-tensile

TABLE 5

PHASE II TEST RESULTS

Soil	Lime Type	Lime %	Curing (days)	q_u (psi)[a]	Split Strength (psi)[a]	Split/q_u
1	B	7	15	1004	133	0.13
2	A	7	66	739	92	0.13
3	A	5	50	1618	207	0.13
4	A	5	20	640	97	0.15
5	B	7	15	608	85	0.14
6	C	3	30	798	119	0.15
7	B	5	31	408	59	0.15
8	C	5	7	806	103	0.13
9	A	3	20	863	104	0.12
10	C	3	50	1051	154	0.15
11	B	3	40	778	87	0.11

[a]Average of eight specimens.

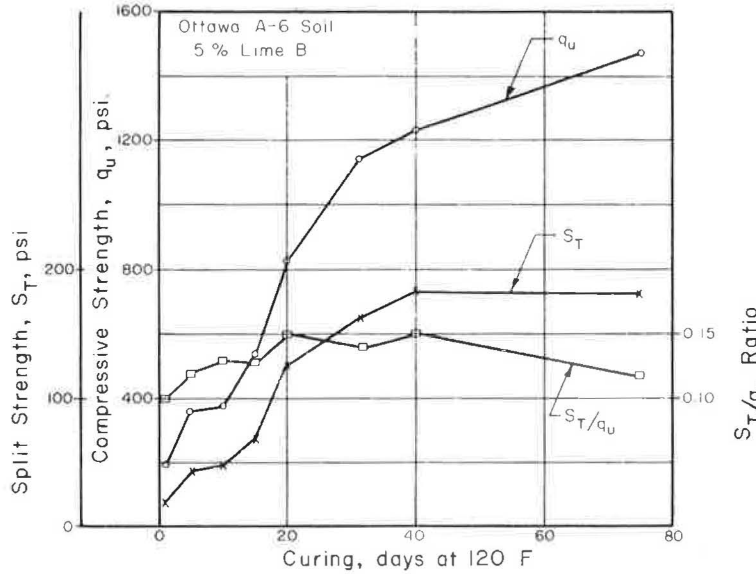

Figure 8. Curing effects.

and compressive strength vary in a similar manner and that the ratio between them exhibits little variation.

In Figure 9, the S_T/q_u ratio for selected lime treatments is plotted as a function of q_u to determine if this ratio is constant over a wide range of compressive strengths. Linear regression analyses (Table 6) show that for every lime treatment the S_T/q_u ratio is not a function of compressive strength (slope of regression line is not significantly different from 0, $\alpha = 0.05$). Since the ratio is not related to compressive strength, the best estimate of the S_T/q_u ratio for a given lime type and percentage is the average for all curing periods.

Analysis of variance test results (Table 7) also indicate that the S_T/q_u ratios for various lime types and percentages are not significantly different ($\alpha = 0.05$).

In summary, the S_T/q_u ratio for this soil (Ottawa A-6) is not influenced by compressive strength (curing period), lime type, or lime percentage. Factors which increase or decrease the compressive strength of lime-soil mixtures influence split-tensile strength in a similar manner.

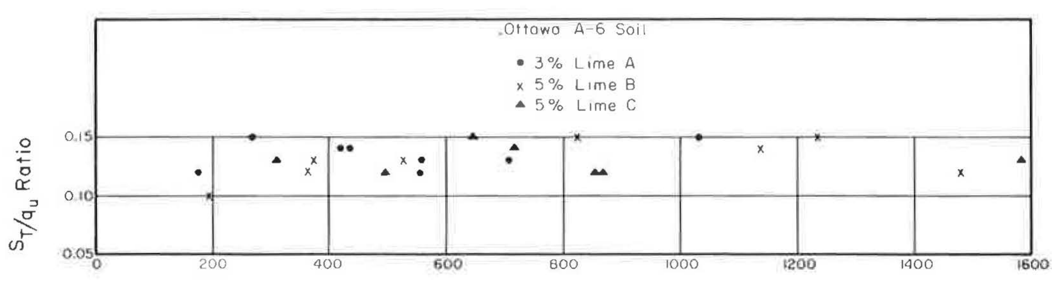

Figure 9. S_T/q_u ratio vs compressive strength.

TABLE 6

LINEAR REGRESSION ANALYSIS
(Ottawa A-6 Soil)

Description: This test was conducted to determine if the S_T/q_u ratio is linearly related to the magnitude of the compressive strength, q_u. Varying compressive strengths were obtained by altering the length of the curing period.

Hypothesis: $\beta = 0$, or the slope of the regression line between S_T/q_u and q_u is not significantly different from 0.

Lime Type	Lime %	F Calculated
A	3	0.003
A	5	0.56
B	3	2.99
B	5	2.06
C	3	3.2
C	5	0.03

NOTE: None of the calculated F values were significant ($\alpha = 0.05$); therefore, the hypothesis is accepted in every case.

TABLE 7

ANALYSIS OF VARIANCE TEST RESULTS
(Ottawa A-6 Soil)

Description: This test was conducted to determine if there is a significant difference in the S_T/q_u ratios for different lime types and percentages. The S_T/q_u ratio shown below is the average of all test results for a given lime type and percentage, regardless of curing period.

Hypothesis: S_T/q_u ratios are equal for the various lime types and percentages.

Lime Type	Lime %	Avg. S_T/q_u
A	3	0.135
A	5	0.129
B	3	0.121
B	5	0.130
C	3	0.130
C	5	0.130

Calculated F value = 0.05.
F value not significant at $\alpha = 0.05$; therefore, hypothesis is accepted.

Phase II

Since the results of Phase I indicate that lime type and curing period do not influence the S_T/q_u ratio, the random assignment of lime type and curing period for the 11 soils included in Phase II is justified. The results from Phase II of the test program (Table 5) show a wide variation in compressive and split-tensile strengths, depending on soil type, lime percentage and curing period. It is evident, though, that the S_T/q_u ratio remains essentially constant. Analysis of variance test results (Table 8) indicate a significant difference ($\alpha = 0.05$) between the S_T/q_u ratio for the individual soil types. Although significant differences exist between ratios for some of the soils, the range is only from 0.113 to 0.155. For practical purposes, an overall average of approximately 0.13 for the S_T/q_u ratio may be appropriate.

GENERAL DISCUSSION

It is evident that cured lime-soil mixtures possess substantial tensile strength, but, like other typically brittle materials, it is small in comparison to its compressive strength. The average S_T/q_u ratio of 0.13 compares favorably with results reported for concrete (5, 12) but appears to be slightly higher. Though the split-tensile strength is low, it is emphasized that the modulus of rupture (flexural strength) for other materials reported in the literature is from approximately 1.5 to 3.0 times larger than its split-tensile strength (8). With a correlation factor of this magnitude, the tensile strength properties of lime-soil mixtures are comparable with the flexural strengths of other stabilized highway materials with approximately the same compressive strength.

TABLE 8

ANALYSIS OF VARIANCE TEST RESULTS, INFLUENCE OF SOIL TYPE

Description: This test was conducted to determine if there is a significant difference in the S_T/q_u ratios for different soil types. The S_T/q_u ratios shown are the averages of the ratios of the eight pairs of specimens (one pair is a split specimen and an unconfined specimen) for each lime-soil mixture.

Hypothesis: The S_T/q_u ratios are equal for the different soil types.

Soil	Lime		Curing at 120 F (days)	Avg. S_T/q_u
	Type	%		
Bryce B, Iroquois Co.	A	5	20	0.155
Cowden B, Randolph Co.	B	5	31	0.149
Illinoian till, Effingham Co.	C	3	50	0.147
Cowden B, Montgomery Co.	B	7	15	0.143
Accretion gley-2, Sangamon Co.	A	5	50	0.129
Illinoian B, Sangamon Co.	C	5	7	0.128
Illinoian till, Sangamon Co.	A	3	20	0.126
Accretion gley-1, Sangamon Co.	A	7	6	0.126
Cowden C, Montgomery Co.	C	3	30	0.116
Illinoian B, Effingham Co.	B	7	15	0.113
Ottawa A-6, LaSalle Co.	B	3	40	0.113

Calculated F value = 6.8.
F value is significant at $\alpha = 0.05$; therefore, hypothesis is rejected.

Results of Duncan's Multiple Range Test are shown above.

NOTE: Any two means not indicated by the same line are significantly different. Any two means indicated by the same line are not significantly different.

The tensile strength of a lime-soil mixture should greatly contribute to its performance as a highway construction material. Since the lime-stabilized material can withstand tensile stresses, a lime-soil pavement layer should behave differently than a flexible-type pavement. The work of Nichols (13, 14) appears to substantiate the foregoing statement. In his investigations, he attributed the reduced deflections of pavement sections with stabilized subgrades (lime or cement) to the "slab strength afforded by a semi-rigid subgrade." The improved performance noted for many pavement sections with lime-treated subgrades may also be partially attributable to the slab strength which can be developed if the material possesses substantial tensile strength.

CONCLUSIONS

Based on the results of this investigation, the following conclusions were drawn:

1. The split test has considerable merit as a test procedure for evaluating the tensile strength properties of lime-soil mixtures;
2. Lime-soil mixtures possess substantial tensile strength which is influenced by such factors as soil type, lime treatment and curing period;
3. The ratio of split-tensile strength to compressive strength (S_T/q_u) shows no statistically significant variation for a particular soil type;
4. Small but significant differences were observed in the S_T/q_u ratios for the different soil types; and
5. The best available estimate of the S_T/q_u ratio for lime-soil mixtures is an overall average of 0.13.

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Discussion

DAVID L. TOWNSEND, Queen's University, Kingston, Ontario, Canada.—The author's application of the split-tensile strength test to the study of lime-stabilized soils represents an interesting and potentially very useful step towards a clearer understanding of the mechanical properties of these soils. There is some justification in considering that the tensile strength may be a better criterion to apply for failure conditions than the usual unconfined compression test, and it is possible that it may have some wider applications with regard to durability of the stabilized soils.

In connection with a study of the durability of lime-stabilized soils under freezing and thawing conditions, T.W. Klym, at the discussor's suggestion, conducted a preliminary study of the suitability of the split-tensile test as a measure of resistance to heaving. The results indicated as much lack of relationship as the unconfined compression test, and further testing was discontinued in favor of developing a better understanding of the mechanism by which lime-stabilized soils resist deterioriation under freezing and thawing conditions. However, in view of the author's results, it is pertinent to augment his reported information with these preliminary results.

The testing methods were as follows. Normal dolomitic hydrated lime was manually mixed with five soils and allowed to cure for 24 hr before being dynamically compacted to 100 percent standard proctor compaction in a 2.0-in. diameter by 3.0-in. long mold and cured in sealed containers at 70 F for various periods. The sample size was dictated by the durability test methods. The split-tensile specimens were tested at a rate of 0.05 in./min between loading strips with an a/d ratio of ⅛; and the unconfined compression tests were conducted at 0.10 in./min in a standard soil testing compression unit. As this was a preliminary study, it was felt that the difference in testing rates would not be significant insofar as indicating if the tensile strength could be related to durability.

The results of the individual tests are given in Table 9. Clay mineral identification was performed by standard X-ray diffraction analysis, and other index tests follow the standard procedures. The actual unconfined compression test values are given, and these have been corrected by a factor of 0.96 to convert to a standard length-to-diameter ratio of 2:1. Although this conversion factor is based on concrete samples, it is felt to be reasonably valid for these weakly cemented soils. The amount of lime for each soil was based on the amount that would normally be used for these soils under highway construction conditions and is approximately 4 percent more than the amount required for fixation as indicated by plasticity tests. Unconfined compression and split-tensile strength tests were also performed immediately after sample preparation, but it is felt that these values are not pertinent to this discussion.

From the very limited results, it may be apparent that:

1. There is considerable variation in the observed values of split-tensile strength for any one curing period and soil-lime combination (in some cases, as high as 20 percent from a mean value); and
2. There is a variation in the ratio of S_T/q_u for the five soils tested which is similar to those reported by the author. However, in two cases, the ratio appears to be larger or smaller than the values reported.

There may be some small variation in the ratio of S_T/q_u when related to the activity of the soil, but there is far from sufficient evidence to establish any trend.

In many engineering feasibility studies, it is customary to use only two or three test samples for the measurement of physical properties. However, from these results, it may appear that there can be significant variation within the split-tensile strength test. If, in the future, designs are to be related to this strength, it would

TABLE 9

Soil No.	Classification	Clay Minerals	% Clay	I_p	Activity	% Lime	14-Days Curing				28-Days Curing				4-Months Curing			
							Act. q_u (psi)	Corr. q_u (psi)	Act. S_T (psi)	Ratio	Act. q_u (psi)	Corr. q_u (psi)	Act. S_T (psi)	Ratio	Act. q_u (psi)	Corr. q_u (psi)	Act. S_T (psi)	Ratio
S-2	CI	Mont.	22	25	1.1+	6	174	167	16.6		228	219	27.4		465	446	41.7	
							172	165	13.9		240	230	18.4		422	405	49.5	
							176	169	19.3		246	236	25.8		462	443	56.2	
								167	16.6	0.10		228	23.9	0.10		431	49.1	0.11
A-1	CH	Mont.	75	68	0.9	7	198	190	21.8		252	219	27.4		328	315	25.1	
							206	198	17.3		250	240	30.6		322	309	23.1	
							190	183	19.2		222	213	22.8		301	289	23.5	
								190	19.4	0.10		232	26.6	0.11		301	23.9	0.08
M-2	CH	Mont.	71	56	0.8	7	260	250	27.1		320	307	28.8		376	361	36.4	
							257	247	27.9		295	283	33.9		369	354	41.9	
							265	254	29.4		296	284	30.7					
								250	28.1	0.11		291	31.1	0.11		358	39.2	0.11
Q-1	CL	-	32	18	0.6	6	110	106	14.6		148	142	15.1		298	286	44.1	
							116	111	17.6		139	134	23.0		305	293	43.5	
							118	113	15.2		142	136	22.1		301	289	53.1	
								110	15.8	0.14		134	20.1	0.15		289	46.9	0.16
O-1	CI	Kaol.-ill.	44	21	0.5	6	127	122	13.6		165	158	18.7		378	363	51.5	
							113	108	13.0		150	144	23.1		370	355	38.6	
							110	106	12.7		142	136	26.1		377	362	48.2	
								112	13.1	0.12		146	22.6	0.15		360	46.1	0.13

appear that the variation within the test itself should be known, so that extrapolations to erroneous values are not made. It would be appreciated if the author could indicate the typical variations which developed within the eight samples of a series used in the various parts of his study.

MARSHALL R. THOMPSON, Closure. —Mr. Townsend's comments are appreciated. The values he reports for the ratio of split-tensile strength to compressive strength are of the same general magnitude as those presented in the paper.

In reply to Mr. Townsend's question, the coefficient of variation, $\frac{\sigma}{\bar{x}}$, obtained by using eight samples in a series was approximately 10 percent for both the split-tensile specimens and the unconfined compression specimens.