Rept. LR 205, 1968.
20. L. R. Chadda. The Phenomenon of Aggregation in the Stabilization of Soils With Cement. Indian Concrete Journal, Vol. 44, No. 5, 1970, pp. 210-212.
21. A. Herzog. Evidence for Skeleton-Matrix Structure in Clays Stabilized With Portland Cement. Proc., 5th Australian-New Zealand Conference on Soil Foundation Engineering, Petone, 1967, pp. 55-61.
22. D. H. Everett and J. M. Haynes. Capillary Properties of Some Model Pore Systems With Reference to Frost Damage. Rilem, Bulletin 27, 1965, pp. 31-38.
23. R. G. Packard and G. A. Chapman. Developments in Durability Testing of Soil-Cement Mixtures. HRB, Bulletin 36, 1963, pp. 97-122.
24. L. T. Norling. Standard Laboratory Tests for Soil-Cement Development: Purpose and History of Use. HRB, Highway Research Record 36, 1963, pp. 1-10.
25. M. R. Thompson. Durability and Frost Resistance of Lime and Cement-Treated Soils. Proc., Symposium on Frost Action on Soils, Organization for Economic Cooperation and Development, Vol. 2, 1973, pp. 377-393.
26. D. L. Townsend and T. W. Klym. Durability of Lime-Stabilized Soils. HRB, Highway Research Record 139, 1966, pp. 25-39.
27. B. J. Dempsey and M. R. Thompson. Effects of Freeze-Thaw Parameters on the Durability of Stabilized Materials. HRB, Highway Research Record 379, 1972, pp. 10-18.

Publication of this paper sponsored by Committee on Soil-Portland Cement Stabilization and Committee on Frost Action.

Tensile-Strength Determinations of Cement-Treated Materials

L. Raad, Department of Civil Engineering, University of Illinois at Urbana-Champaign
C. L. Monismith and J. K. Mitchell, Department of Civil Engineering and Institute of Transportation Studies, University of California, Berkeley

Differences in values of the tensile strength of cement-treated materials measured by using flexure, direct tension, and split tension tests are explained analytically by using Griffith failure theory; and predicted values are shown to agree well with the strength data available in the literature. The results indicate that the direct tension test provides the most reliable values of tensile strength of cement-treated materials, even when the failure surface is close to the interface between the cap and the specimen. Flexural strengths deduced from beam tests can be as much as twice the actual tensile strengths, depending on beam geometry, moduli in tension and compression, and degree of fixity at support and load-application points. The split tension test appears most suitable for practical use in evaluation of the tensile strength of cement-treated materials because it is simple to perform and yields measured values that do not deviate by more than 13 percent from the actual tensile strength.

Cement-treated bases used in pavement structures are subjected to tensile stresses caused by applied wheel loads, shrinkage, and internal temperature gradients. When these stresses exceed the tensile strength of the material, cracking takes place (1, 2, 3), which results in increased deflections and stress transmissions to the subgrade (4). In addition, the presence of open cracks permits the entry of water into the subgrade, which leads to decreased subgrade support and thereby hastens pavement deterioration. Determination of the tensile strength of cement-treated materials is therefore a desirable part of the design process for pavement structures containing these materials.

Tensile strength has been measured by using flexure tests, direct tension tests, and split tension tests (e.g., 2, 5, 9). The estimation of tensile strength from flexural-beam tests and split tension tests is usually made by using simplifying assumptions about the stress-deformation characteristics of the material, i.e., linear-elastic behavior with the modulus in tension (E_t) equal to the modulus in compression (E_c). The values of tensile strength obtained on this basis vary and depend on the kind of test used. The split tension test gives lower values of tensile strength than do the flexure and direct tension tests (5, 6), and the flexure test gives higher values than does the direct tension test (2, 5).

Experimental work on the stress and strain behavior of cement-treated soils has shown that the modulus in compression is greater than the modulus in tension (2, 3, 5). Bofinger (5) has suggested that the flexural-beam and the split tension tests do not measure the actual tensile strength of the cement-treated soil because E_c and E_t are not equal and concluded that the direct tension test is the only method that can directly measure the tensile strength of the material. On the other hand, Pretorius (2) has shown that there are stress concentrations at the ends of the specimen in a direct tension test, so that the measured tensile strength measured by this test could also be unreliable.

In this paper, an attempt has been made to explain the differences observed in the three types of tests by using analytic simulations and actual experiments.

METHOD OF ANALYSIS

The cement-treated material was assumed to be linearly elastic with a modulus in compression that might differ from the modulus in tension. The finite-element method of analysis was used to determine the stresses and deformations for a specific test configuration (Figures 1 and 2) resulting from particular load. A planar stress condition was assumed, and a successive-approximation technique was used to include the bimodular material properties (i.e., $E_c \neq E_t$). In this procedure, the modulus in compression was used for all elements on the first iteration. On successive iterations, the modulus in tension was substituted in the directions of principal tension. Three or four iterations were usually sufficient to attain reasonable convergence.

For the specific test under consideration (i.e., flexural beam, split tension, or direct tension), the failure load is calculated as follows:

1. The applied load is incremented until the most critically stressed element fails. Failure, as defined by the Griffith criterion (7), which is applicable for tension and small compressive-stress fields, takes place when

$$(\sigma_1 - \sigma_3)^2/8(\sigma_1 + \sigma_3) = T_a \quad (\sigma_1 + 3\sigma_3 \geq 0)$$
$$\sigma_3 = -T_a \quad (\sigma_1 + \sigma_3 \leq 0) \tag{1}$$

where

σ_1 = major principal stress,
σ_3 = minor principal stress, and
T_a = actual tensile strength of the material and corresponds to the uniaxial uniformly applied tensile stress required to cause fracture in the material.

(Tensile stresses are negative; compressive stresses are positive.)

2. The failed element is removed from the system, and a new state of stress for each element is determined.

3. This process is continued until complete fracture of the specimen occurs. A failure load (P_f) in terms of the actual tensile strength of the material can then be determined.

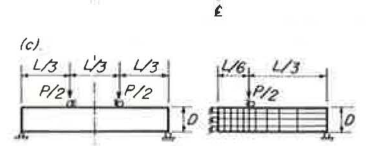


Figure 1. Finite-element representation of flexural-beam test.

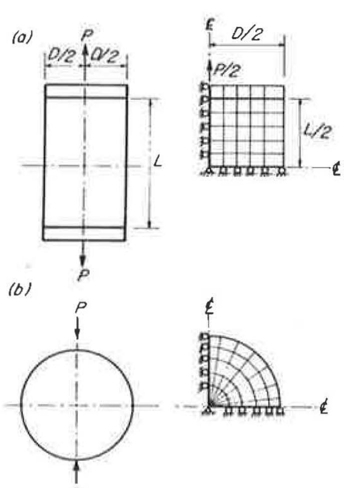


Figure 2. Finite-element representation of direct and split tension tests.

By using this value of P_f, a tensile-strength value (T_c) of the material is then calculated on the assumptions that

1. The material is linearly elastic ($E_c = E_t$),
2. Simple beam theory holds true in the flexure test,
3. Lateral restraints that may exist at the points of application of the load in the flexure test [Figure 1(c)] have negligible effects on the stress distribution in the beam, and
4. In the direct tension test, the stress concentration at the ends of the specimen where the loads are applied is negligible.

For the flexural-beam test,

$$T_c = M_f c/I \tag{2a}$$

where

M_f = moment that corresponds to P_f and acts at a section through the center of the beam,
c = half the depth of the beam, and
I = moment of inertia of the beam section.

For the split tension test,

$$T_c = P_f/\pi R \tag{2b}$$

where R = radius of the sample. For the direct tension test,

$$T_c = P_f/A \tag{2c}$$

where A = cross-sectional area of the sample.

The value of P_f in all of these tests corresponds to the load per unit thickness that is required to cause complete fracture. P_f is expressed in terms of the actual tensile strength of the material. T_c can be expressed in terms of T_a, and the ratio T_c/T_a for a given E_c/E_t can be calculated.

RESULTS

The variation of T_c/T_a as a function of E_c/E_t has been determined for all three tests. The results are summarized in the following sections.

Flexure Test

Three loading conditions were investigated: (a) central loading [Figure 1(a)], (b) third-point loading with no lateral restraints [Figure 1(b)], and (c) third-point loading with lateral restraints [Figure 1(c)]. The results are shown in Figures 3 and 4 for different length-to-depth beam ratios (L/D); the variation of T_c/T_a as a function of E_c/E_t is also plotted in Figure 3 by using simple beam theory. The derivation of the following expression for T_c/T_a in terms of E_c/E_t by using simple beam-theory assumptions has been given by Raad (3).

$$T_c/T_a = 2\left\{1 - [(R^{1/2} - 1)/(R - 1)]\right\} \tag{3}$$

where $R = E_c/E_t$.

Analysis of the results shows that

1. T_c/T_a increases as E_c/E_t increases for all of the cases studied;

Figure 3. Analytically predicted values of measured tensile strength in terms of actual tensile strength.

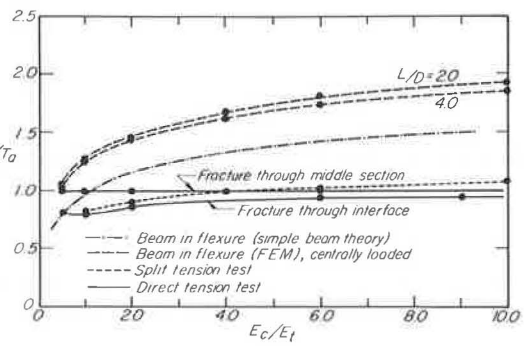


Figure 4. Effect of lateral restraints of applied vertical load on measured values of tensile strength in flexural-beam test.

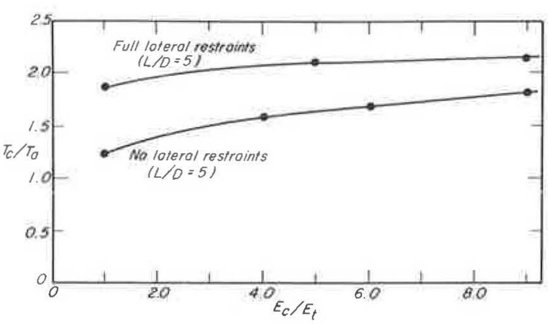
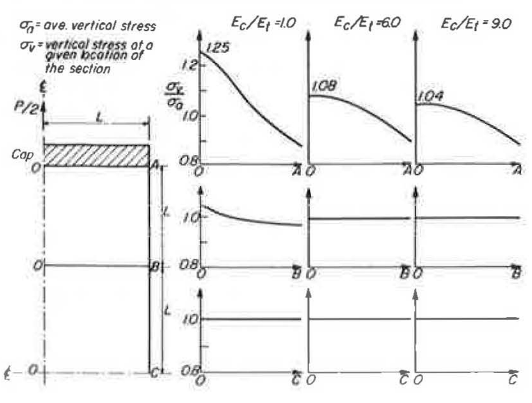


Figure 5. Stress concentrations in direct tension test.



2. The value of T_c/T_a corresponding to a given E_c/E_t will be greater if the applied load is laterally restrained from movement than if there are no restraints;

3. Both simple beam theory and the finite-element method of analysis predict higher values of strength than the actual measured tensile strength of the material for $E_c/E_t > 1$ (i.e., $T_c/T_a > 1$); and

4. The values of measured tensile strength, T_c, obtained in the flexural-beam test are higher than those obtained in the direct or the split tension tests.

Direct Tension Test

In this test, the load is applied through rigid caps (i.e., made of a material having a high modulus of elasticity compared to that of the cement-treated material) bonded to the specimen (i.e., with no relative movement of the interface between the caps and the specimen allowed). The effect of this kind of restraint on the stress distribution and on the measured tensile strength of the material was investigated. The predicted values of the tensile strength are shown in Figure 3, and the stress concentrations are shown in Figure 5. The results can be summarized as follows:

1. If the failure surface is located at the center of the specimen, then the measured tensile strength (T_c) will be the same as the actual tensile strength (T_a) for all values of E_c/E_t;

2. If the failure surface occurs at the interface between the cap and the material, then the measured tensile strength will vary from 0.8 to 0.94 T_a, depending on the value of E_c/E_t (for $E_c/E_t > 4.0$, T_c/T_a will be equal to 0.94); and

3. Stress concentrations occur near the interface of the cap and the specimen and increase with the decrease in the radial distance from the center of the specimen (the normal tensile stress at the interface near the center of the cap could have values of 1.25 and 1.04 times the average applied normal stress for $E_c/E_t = 1.0$ and 9.0 respectively.

Split Tension Test

The results of analysis of this test configuration are given in Figure 3 and can be summarized as follows: (a) T_c/T_a increases as E_c/E_t increases (the measured tensile strength varies from 0.8 T_a at $E_c/E_t = 1.0$ to 1.04 T_a at $E_c/E_t = 9.0$) and (b) the values of tensile strength measured by using this test are smaller than those given by the flexural test, but can be either smaller or larger than those measured by the direct tension test, depending on the value of E_c/E_t and on the position of the failure surface.

Table 1. Predicted values of tensile-strength ratio of cement-treated materials using different laboratory tests.

	Failure Near Interface[a]					Failure Through Midsection[a]				
		T_f/T_d					T_f/T_d			
		L/D[b] = 5					L/D[b] = 5			
E_c/E_t	T_a/T_d	FR[d]	NR[e]	L/D[b,c] = 4	L/D[b,c] = 2	T_a/T_d	FR[d]	NR[e]	L/D[b,c] = 4	L/D[b,c] = 2
1	1	2.32	1.52	1.52	1.57	0.80	1.85	1.25	1.25	1.26
10	1.11	2.28	1.94	1.96	2.04	1.04	2.15	1.85	1.84	1.92

[a] Refers to the position of the failure surface in the direct tension test.
[b] L/D = length of beam/depth of beam in flexure test.
[c] A central or a three-point loading system with no lateral restraint is used in the flexure test.
[d] Full lateral restraint of the applied vertical load in the flexure test.
[e] No lateral restraint of the applied vertical load in the flexure test.

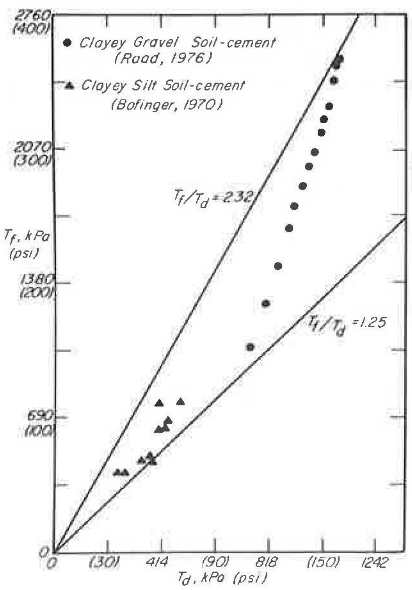


Figure 6. Measured and predicted values of direct tensile and flexural strengths.

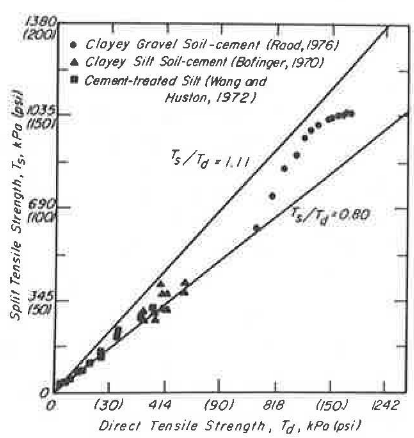


Figure 7. Measured and predicted values of direct and split tensile strengths.

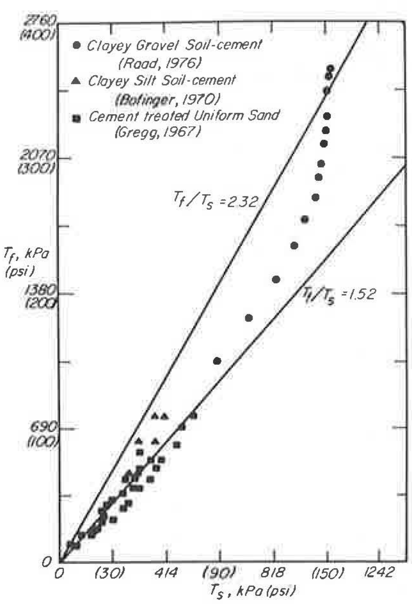


Figure 8. Measured and predicted values of flexural and indirect tensile strengths.

EXPERIMENTAL VERIFICATION

Previously published test data for flexural strength, direct tensile strength, and split tensile strength can be compared with the values predicted by using the suggested analytical procedure. A clayey, gravel soil-cement and a cement-treated, clayey silt were tested in tension and in compression using metal caps bonded to the specimens with an epoxy resin to apply the tensile loads. The values for initial tangent moduli and secant moduli from these tests are given by Raad (3) and Bofinger (5); E_c/E_t varied from 5 to 11. Analytical predictions for the ratios of the split tensile strength (T_s) to the direct tensile strength (T_d) and of the flexural strength (T_f) to T_d for $E_c/E_t = 1$ and 10 are summarized in Table 1.

Measured values of tensile strength for cement-treated soils using the direct tension test, the split tension test, and the flexural test have been given by a number of investigators (4, 5, 6, 8). Comparisons between the measured and the predicted values are shown in Figures 6, 7, and 8. Figure 6, for example, shows that the measured values of the flexural strength (T_f) and of the direct tensile strength (T_d) should vary, according to the results of analysis (Table 1), such that T_f/T_d will be in the range of 1.25 to 2.32. This figure also shows that the correspondence between the actual and the predicted results is actually obtained. Figures 7 and 8 indicate similar results.

SUMMARY AND CONCLUSIONS

In this paper, an attempt has been made to explain analytically the differences in measured tensile strength for cement-treated soils. The analyses showed that the direct tension test gives reliable values of the actual tensile strength of such materials. Even if the failure surface is near the interface between the cap and the tested specimen, the measured tensile strength will be about 0.94 times the actual tensile strength. The flexural strength deduced from beam tests by using simple beam theory and assuming that $E_c = E_t$ can be as much as twice the true tensile strength, depending on the beam geometry, the value of E_c/E_t, and the degree of fixity at the support and load-application points. The split tension test would seem best for the practical evaluation of tensile strength because of its simplicity. The true tensile strength can be estimated as 1.11 times the split tensile strength. On this basis, the error in the estimated value would not exceed 13 percent.

REFERENCES

1. K. P. George. Shrinkage Characteristics of Soil-Cement Mixtures. HRB, Highway Research Record 255, 1968, pp. 42-58.
2. P. C. Pretorius. Design Considerations for Pavements Containing Soil-Cement Bases. Univ. of California, Berkeley, PhD dissertation, 1970.
3. L. Raad. Design Criterion for Soil-Cement Bases. Univ. of California, Berkeley, PhD dissertation, 1976.
4. P. E. Fossberg, J. K. Mitchell, and C. L. Monismith. Cracking and Edge-Loading Effects on Stresses and Deflections in a Soil-Cement Pavement. HRB, Highway Research Record 379, 1972, pp. 25-38.
5. H. E. Bofinger. The Measurement of the Tensile Properties of Soil-Cement. British Road Research Laboratory, Crowthorne, England, Rept. LR 365, 1970.
6. J. S. Gregg. The Significance of Compressive, Tensile, and Flexural Strength Tests in the Design of Cement-Stabilized Pavement Foundations. Proc., 4th Regional Conference for Africa on Soil Mechanics

and Foundation Engineering, Capetown, South Africa, Dec. 1967.
7. A. A. Griffith. Theory of Rupture. Proc., 1st International Congress for Applied Mechanics, Delft, Netherlands, 1924, pp. 55-63.
8. M. C. Wang and M. T. Huston. Direct Tensile Stress and Strain of a Cement-Stabilized Soil. HRB, Highway Research Record 379, 1972, pp. 19-24.
9. J. N. Anagnos, T. W. Kennedy, and W. R. Hudson. Evaluation and Prediction of the Tensile Properties of Cement-Treated Materials. Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 98-8, Oct. 1970.

Publication of this paper sponsored by Committee on Soil-Portland Cement Stabilization.

Performance Study of Asphalt Road Pavement With Bituminous-Stabilized-Sand Bases

Claude P. Marais and Charles R. Freeme, National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa

The possibility of using the windblown sands that occur in the northern areas of South West Africa for the construction of all-weather roads to carry heavy truck traffic has been investigated. Laboratory investigations and field trials in Pretoria, South Africa, showed that bituminous stabilization of these sands was promising, and a full-scale road experiment to test a limited number of bases of bituminous-stabilized sand was constructed in the homeland of Owambo, South West Africa. This paper describes the laying of the experiment and the construction techniques and control measures used. A new technique that establishes the optimum time for the compaction of a cutback bituminous-stabilized sand mixture after aeration by using a vane shear apparatus is described. The vane shear apparatus was also used to measure the in situ shear strengths of the various experimental bituminous-stabilized sand bases after compaction and during service; the results of these measurements, together with performance data after 8 years service with respect to deformation and cracking, are discussed. Laboratory and field studies are described and predictions about the performance of a bituminous-stabilized sand base under varying traffic conditions are made by using the best known techniques available at this time.

Vast areas of the southern subcontinent of Africa are covered with a deep blanket of aeolian sand. Probably these sands were originally derived from preexisting sedimentary rocks in the general area and first emplaced by wind during the lower most Pleistocene epoch (approximately 2 000 000 years ago). They were subsequently redistributed by wind and water during the Pleistocene; the latest major redistribution was brought about by wind action, probably some 10 000 to 15 000 years ago, although some minor redistribution is still occurring (1).

Because of their widely spread occurrence, these sands, apart from various types of calcrete (caliche), are sometimes the only natural building material available to the civil engineer. From an economic point of view, they are therefore extremely important and have been studied for use in concrete structures, building construction, and, more recently, pavement construction by the National Institute for Transport and Road Research (NITRR) of the Council for Scientific and Industrial Research in Pretoria, South Africa (2, 3, 4).

This paper describes the use of these aeolian sands as the base layer of a road pavement in the recently proclaimed homeland of Owambo, in the northern part of South West Africa (SWA). It discusses the performance results of the experiment and relates these to the probable performance that might be expected under much heavier traffic on a normal freeway.

The accelerated development of the infrastructure of Owambo during the past decade necessitated upgrading the existing gravel road linking Owambo to the more developed, southern part of SWA to an all-weather, 8-m-wide, black-topped facility. The construction of the R60 000 000 ($84 000 000) hydroelectric facility at Ruacana Falls and other major building schemes in Owambo have resulted in a significant increase in heavy freight vehicles using this, the only surface transportation route to the south.

Initial laboratory work by the NITRR in the early 1960s showed that the most suitable method of improving the engineering properties of the in-place sand was to blend it with 15 percent calcareous filler (mechanical stabilization) and then to bind the blend with a bituminous binder. Both the hot-mix and cold wet-mix processes were studied; the latter was adopted as the more practical because of the length of road required and the problems associated with the establishment of a hot-mix facility in this remote area.

After extensive preliminary research into the wet-sand process of bituminous stabilization of fine-grained wind-blown sands, a full-scale road experiment was carried out in May 1965 in Owambo to test the techniques developed during the preliminary study (3, 4).

DETAILS OF EXPERIMENT

The experiment was designed and constructed with the following objectives:

1. To demonstrate in the field the feasibility of in situ bituminous stabilization of sand by using cutback binders and a cationic bitumen emulsion;
2. To investigate the stability and durability, under the traffic conditions and climatic environment of the site, of various bituminous-sand mixtures containing cutback bitumens, a cutback tar, and a cationic bitumen emulsion at binder contents considered suitable from