2/60

Effective Stress Strength Parameters of Stabilized Soils

Paramètres des contraintes effectives pour les sols stabilisés

A. E. Z. WISSA, *Instructor of Civil Engineering, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, U.S.A.*

C. C. LADD, *Associate Professor of Civil Engineering, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, U.S.A.*

T. W. LAMBE, *Professor of Civil Engineering, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, U.S.A.*

SUMMARY

The effective stress strength behaviour of a number of saturated soil-cement and soil-lime systems consolidated to pressures up to 60 kg/sq.cm. was obtained from consolidated undrained triaxial tests with pore-pressure measurements. The data show that the Mohr effective stress strength envelopes for these systems are essentially straight lines with slopes which are independent of curing time. However, the magnitude of the cohesion intercept increases with increasing time of cure. The results also show that the Mohr envelopes are sensitive to the type and amount of stabilizer as well as the soil type. At large strains the cohesion intercepts are destroyed, and the stabilized soils then behave like cohesionless materials with angles of shearing resistance which are independent of curing time.

SOMMAIRE

On obtient les caractéristiques de résistance au cisaillement en fonction de la contrainte effective, pour des sols stabilisés au ciment et à la chaux. Ayant consolidé les échantillons sous des pressions atteignant 60 kg/cm.ca., on détermina ces caractéristiques par des essais triaxiaux de compression sans drainage. Les résultats démontrent que les enveloppes de Mohr exprimées en fonction des contraintes effectives sont généralement des droites dont la pente ne dépend pas de la durée de prise. Cependant, les valeurs de la cohésion augmentent en fonction du temps de prise. Les résultats montrent que les enveloppes de Mohr dépendent du type et des quantités du produit stabilisateur ainsi que des types de sol. Au niveau des déformations élevées la cohésion disparaît complètement et les sols stabilisés agissent comme des matériaux granulaires dont les angles de résistance au cisaillement sont indépendants du temps de la prise.

THE IMPROVEMENT OF SOIL PROPERTIES has been an active part of the soils research effort at M.I.T. since 1946. Several new primary stabilizers were developed and the effectiveness of the more conventional ones enhanced by means of small amounts of secondary additives. In recent years it has become increasingly apparent that a more fundamental knowledge of the behaviour of stabilized soils is needed before their potential as construction materials can be fully exploited. Such knowledge could also shed some light on the behaviour of naturally cemented soils. Based on these thoughts, an investigation of strength generation in stabilized soils was initiated in 1961. Some of the results obtained from this investigation are presented in this paper.

MATERIALS AND PROCEDURES

The two soils used in this investigation were Massachusetts clayey silt (M-21) and Vicksburg Buckshot clay (VBC); the properties of these soils are given in Table I. Hydrated lime (reagent grade calcium hydroxide) and portland cement Type I (commercial grade) were the stabilizers used. The physical properties of the soil-stabilizer systems are given in Table II.

Test specimens, 3.15 in. long and 1.405 in. in diameter, were prepared by two-end static compaction using a compactive effort of 400 psi. The as-moulded conditions are given in Table II. The stabilized samples were cured for

TABLE I. PROPERTIES OF UNTREATED SOILS

	M-21	VBC
Textural composition		
Sand, 2 mm to 0.06 mm	42	5
Silt, 0.06 mm to 0.002 mm	43	65
Clay, <0.002 mm	15	30
Physical properties		
Liquid limit, %	20.5	64.5
Plastic limit, %	14.7	28.0
Plasticity index, %	5.8	36.5
Specific gravity	2.75	2.66
Max. dry density, lb/cu. ft.[1]	123.0	106.0
Optimum water content, %	13.2	18.8
Classification		
Unified	CL-ML	CH
AASHO	A-4(0)	A-7-5(20)
Chemical properties[2]		
Organic matter, % by wt.	0.2	1.1
Cation exchange capacity, meq./100 grams	10	30
Glycol retention, mg/gram	22	65
Mineralogical composition[2]		
Clay composition, % by weight	30	50
Illite: montmorillonoid	1:0	1:1[3]
Free iron oxide, % Fe_2O_3	2.9	1.0

[1] Static compaction, 400 psi effort.

[2] For −0.074-mm fractions obtained from a different batch of soil.

[3] Most montmorillonoid mineral is montmorillonite.

TABLE II. PHYSICAL PROPERTIES OF SOIL-STABILIZER SYSTEMS

Soil	Stabilizer	w_L* (%)	w_P* (%)	I_P (%)	Optimum γ_d (lb/cu. ft.)	Optimum w (%)	Moulding γ_d (lb/cu. ft.)	Moulding w (%)	$\bar{\phi}$ (degrees)
M-21	None	20.5	14.7	5.8	123.0	13.2	122.5±0.1	13.8±0.1	30.5
M-21	3% cement	21.3	17.2	3.6	117.5	13.7	118.1±0.1	14.9±0.2	32
M-21	5% cement	21.2	17.6	3.6	118.4	14.5	117.6±1.4	15.3±1.1	37
M-21	5% lime	22.5	19.4	3.1	113.8	16.0	112.2±0.7	15.2±0.8	31.5
VBC	None	64.5	28.0	36.5	106.0	18.8	91.0±0.3	31.4±0.5	20
VBC	5% lime	61.4	47.3	15.1	100.0	20.0	89.1±0.1	30.2	32

*Determined immediately after mixing in of the water.

periods ranging from 4 to 138 days before testing. The curing time includes a period of storage at 100 per cent relative humidity followed by one day of complete immersion in water plus the time required for consolidation and saturation, which varied from 3 to 12 days. The unstabilized soil samples were consolidated and saturated immediately after compaction.

All samples were saturated with a back pressure (i.e., an elevated pore water pressure) of 5 to 10 kg/sq.cm. To overcome the prestress effects due to compaction, and to keep to the same strength/consolidation pressure ratios as used with untreated soils, consolidation pressures ranging from zero to about 50 kg/sq.cm. were used. The samples were tested in undrained shear with pore-pressure measurements. The tests were strain controlled and run at a rate sufficiently slow to permit pore-pressure equalization during shear.

At low consolidation pressures standard triaxial equipment was used (Bishop and Henkel, 1962; Andresen and Simons, 1960); for cell pressures above 10 kg/sq.cm., high pressure triaxial equipment was employed. At high pressures 0.01-in. thick rubber membranes were used to jacket the samples and the pore pressures were determined with differential pressure transducers. Details of the high pressure triaxial equipment and of the testing procedures are presented elsewhere (M.I.T., 1963).

EFFECTIVE MOHR ENVELOPES

Typical effective stress paths (in terms of half the stress difference *versus* the average effective stress) from consolidated undrained triaxial compression tests on untreated and stabilized samples are shown in Fig. 1. The results presented are for untreated M-21 and M-21 stabilized with 5 per cent lime cured for times ranging from 20 to 138 days. For the untreated samples and for the stabilized samples with the same curing time, an essentially straight line can be drawn tangent to the effective stress paths for the entire range of pressures employed; namely, zero to 50

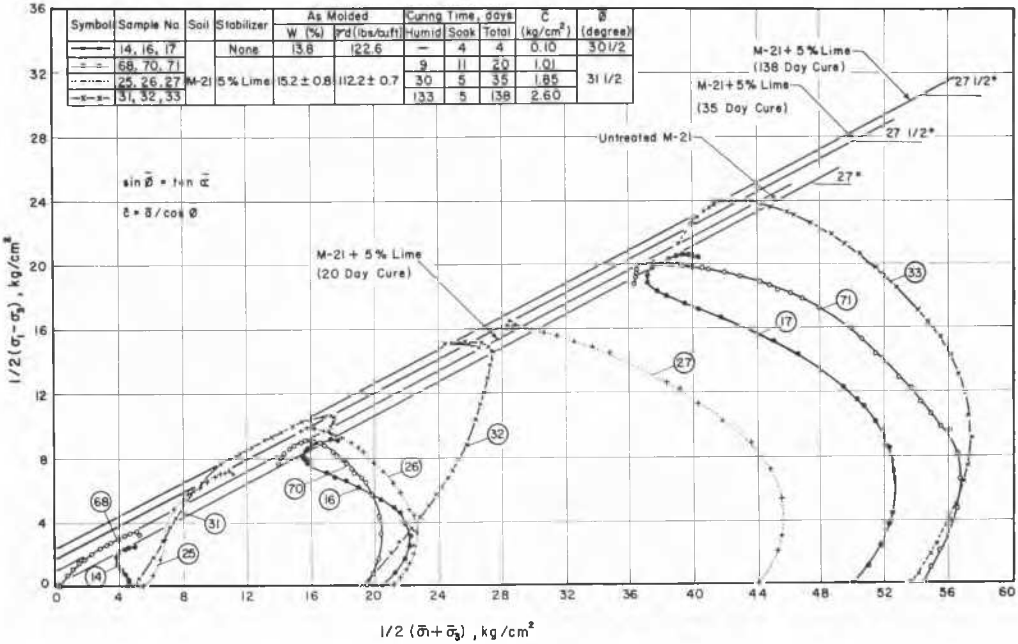

FIG. 1. Typical effective stress paths and Mohr-Coulomb effective stress envelopes for untreated and lime stabilized M-21 (shown on a ½ $(\sigma_1 - \sigma_3$ *versus* ½$(\bar{\sigma}_1 + \bar{\sigma}_3)$ plot).

kg/sq.cm. (the intercept $\bar{a}$ and slope $\bar{\alpha}$ of these envelopes are uniquely related to the Mohr-Coulomb envelopes by $\sin \bar{\phi} = \tan \bar{\alpha}$ and $\bar{c} = \bar{a}/\cos \bar{\phi}$). Similar straight envelopes occurred with the other soil-stabilizer systems investigated. Most previous investigations over such a large range of pressures were drained tests which resulted in curved envelopes. Hirschfeld and Poulos (1963) have shown that applying the volumetric energy correction suggested by Rowe (1962) to the results of drained tests reduced the curvature of these envelopes. Since the tests in this investigation were undrained, and hence no volume changes occurred during shear, it is reasonable that the results yielded essentially straight-line envelopes.

The influence of curing time, t, on the effective stress envelopes of M-21 stabilized respectively with lime and cement was determined for total curing periods ranging from four days to a little over four months. Typical envelopes for the soil-lime system at various curing times are shown in Fig. 1. Similar results were obtained for the soil stabilized with cement. In both cases the Mohr-Coulomb effective angle of shearing resistance, $\bar{\phi}$, was independent of curing time, whereas the effective cohesion intercept, $\bar{c}$, was a function of t. The relations between $\bar{c}$ and t are shown in Fig. 2.

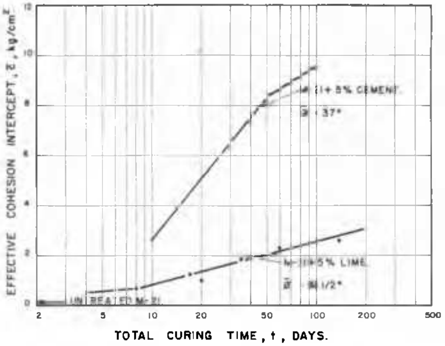

FIG. 2. Effective cohesion intercept as a function of curing time for lime stabilized and cement stabilized M-21.

The influence on $\bar{\phi}$ of the soil type, as well as type and amount of stabilizer, is shown in the last column of Table II. The results indicate that with a given type and amount of stabilizer the magnitude of the increase in $\bar{\phi}$ will depend to a large degree on the type of soil. For example, the addition of 5 per cent lime caused $\bar{\phi}$ to increase by 12 degrees for the clay (VBC) and only by one degree for the silt (M-21). The amount of stabilizer also had an influence on $\bar{\phi}$. In the case of M-21, increasing the cement content from 3 to 5 per cent caused $\bar{\phi}$ to increase from 32 to 37 degrees.

Data at low consolidation pressures have indicated that $\bar{\phi}$ for a given soil-stabilizer system is essentially independent of moulding conditions although the magnitude of $\bar{c}$ varies. It is believed that weathering of stabilized samples, such as cycles of wetting and drying or freezing and thawing, might also influence $\bar{c}$ without having an appreciable effect on $\bar{\phi}$.

In summary, as a good first approximation, the strength behaviour of soils which have been stabilized with small amounts of inorganic additives such as lime or cement can be expressed by the Mohr-Coulomb equation in terms of effective stresses:

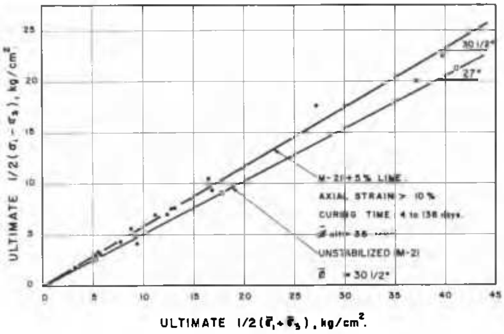

FIG. 3. Relations between ultimate shear stress and ultimate average effective stress for untreated M-21 and for M-21 plus 5 per cent lime with curing times ranging from 4 to 138 days.

$$\tau = \bar{c} + \bar{\sigma} \tan \bar{\phi}$$

where $\bar{\phi} = f$ (soil, type of stabilizer, amount of stabilizer); $\bar{c} = f$ (soil, type of stabilizer, amount of stabilizer, curing history, moulding conditions, weathering).

ULTIMATE STRENGTH

When the stabilized samples were sheared beyond the maximum stress difference and point of tangency with the effective stress envelope, they ultimately reached a condition at which the pore pressure and shear stress remained essentially constant with further straining.* The area used in computing the ultimate stresses was the maximum measured cross-sectional area of the samples at the end of the tests, since at these large strains it was not possible to compute with any accuracy the areas from the axial strains. At this ultimate condition the values of shear stress and effective stress lie on a single envelope having zero cohesion which is independent of curing time, as shown in Fig. 3 for M-21 plus 5 per cent lime. Hence large strains completely destroy the mechanical cementation within the failure zone. It is interesting to note that for the stabilized soils the ultimate angle of shearing resistance, $\bar{\phi}_{ult}$, was greater than the Mohr-Coulomb $\bar{\phi}$ whereas for the untreated soils they were essentially the same. For example, in the case of M-21 (Fig. 3) $\bar{\phi}_{ult}$ was five degrees larger for the lime-stabilized soil than for the untreated soil even though both systems had about the same $\bar{\phi}$ (Fig. 1). This effect may possibly be due to the lime cementing the smaller soil particles into larger ones, as was suggested by Lambe (1960).

INITIAL TANGENT MODULUS AND PORE-PRESSURE RESPONSE

The initial tangent moduli, obtained from the stress difference *versus* axial strain plots, increased with increasing consolidation pressure and increasing curing time. Accompanying the increased modulus was a decrease in the pore-pressure response, i.e., pore-pressure parameter B (Skemp-

---

*The ultimate condition could not always be achieved with the stronger systems, especially at low consolidation pressures, because brittle failure prevented straining beyond the maximum stress difference.

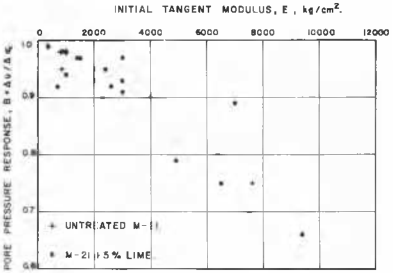

FIG. 4. Effect of initial tangent modulus on the pore-pressure response prior to shear for untreated and lime stabilized M-21.

ton, 1954) measured at the end of consolidation. The scatter in the results, shown in Fig. 4 for M-21 and M-21 plus lime, was presumably due to specimen seating imperfections causing the apparent moduli to be lower than the actual moduli. The lowering of B as E increased reflects the lower compressibility of the mineral skeleton as the consolidation pressure and curing time are increased.

PORE PRESSURES DURING SHEAR (FIG. 1)

At low consolidation pressures the excess pore pressure during shear increased slightly and then dropped and became negative at larger strains. This is similar to the behaviour of heavily over-consolidated clays. At the higher consolidation pressures the pore-pressure behaviour of the stabilized soils became more characteristic of normally consolidated clays, that is, the pore pressure increased at a decreasing rate with strain. The excess pore pressure at maximum stress difference increased linearly with increasing consolidation pressure and was essentially independent of curing time (Fig. 5). The large scatter in the results at the lower consolidation pressures was due to small variations in the moulding conditions of the test specimens.

MAXIMUM SHEAR STRENGTH

The axial strain required to reach the maximum stress difference was independent of both consolidation pressure and curing time, provided brittle failure did not occur. Brittle failure was most apt to occur at the very low consolidation pressures and with the more strongly cemented systems. At the higher consolidation pressures most of the samples failed by bulging.

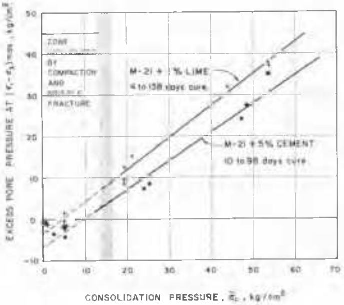

FIG. 5. Excess pore pressure at maximum stress difference versus consolidation pressure for M-21 stabilized with lime and cement respectively.

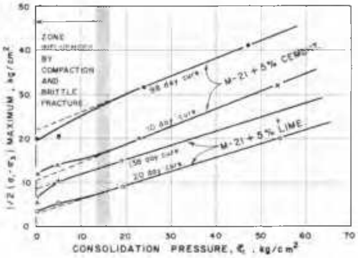

FIG. 6. Influence of curing time on the undrained shear strength of stabilized M-21.

For any given set of curing conditions, provided brittle fracture did not occur, the undrained shear strength increased linearly with consolidation pressure once the effects due to moulding and the tendency for brittle fracture had been overcome (Fig. 6). The rate of increase of strength with consolidation pressure was independent of curing time (Fig. 6) because both the pore pressure at failure and effective angle of shearing resistance are independent of curing time. However, the shear strength at zero consolidation pressure increased with increasing curing time because the effective cohesion intercept is a function of curing time. A reasonably good correlation existed between undrained shear strength and initial tangent modulus. This correlation was independent of consolidation pressure and curing time, as illustrated in Fig. 7 for M-21 plus 5 per cent lime.

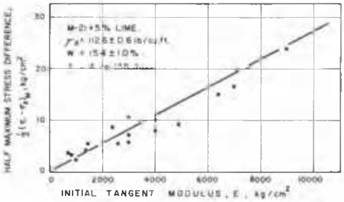

FIG. 7. Relation between initial tangent modulus and shear strength for lime stabilized M-21 with Curing times ranging from 4 to 138 days.

ACKNOWLEDGMENTS

The research on which this paper is based was sponsored by the Waterways Experiment Station, Corps of Engineers, U.S. Army. Mr. Jin H. Kim, a research assistant in Soils at M.I.T., performed most of the laboratory tests.

REFERENCES

Andresen, A., and N. E. Simons (1960). Norwegian triaxial equipment and technique. *ASCE Research Conference on Shear Strength of Cohesive Soils* (Colorado), pp. 695–709.

Bishop, A. W., and D. J. Henkel (1962). *The measurement of soil properties in the triaxial test.* London, Edward Arnold Ltd.

HIRSCHFELD, R. S., and S. J. POULOS (1963). High pressure triaxial tests on a compacted sand and an undisturbed silt. *ASTM Symposium on Shear Testing of Soils* (Canada). (Preprint.)

LAMBE, T. W. (1960). A mechanistic picture of shear strength of clays. *ASCE Research Conference on Shear Strength of Cohesive Soils* (Colorado), pp. 576–7.

M.I.T. (1963). Triaxial equipment and computer program for measuring the strength behavior of stabilized soils. Soil Stabilization Phase Report No. 2, Department of Civil Engineering.

ROWE, P. W. (1962). The stress-dilatancy relation for static equilibrium of an assembly of particles in contact. *Proc. Royal Society*, Series A, Vol. 269, No. 1339, pp. 500–27.

SKEMPTON, A. W. (1954). The pore-pressure coefficients A and B. *Géotechnique*, Vol. 4, pp. 143–4.