THE EFFECTIVE STRESS SHEAR PARAMETERS OF A CLAY STABILIZED WITH LIME

by

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Department of Chemical Engineering
Massachusetts Institute of Technology
Cambridge 39, Massachusetts
January 16, 1961

Professor Philip Franklin
Secretary of the Faculty
Massachusetts Institute of Technology
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Dear Sir:

The thesis entitled, "The Effective Stress Shear Parameters of a Clay Stabilized with Lime", is hereby submitted in partial fulfillment of the requirements for the degree of Bachelor of Science in Chemical Engineering.

Respectfully submitted,

Leslie G. Bromwell

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I. SUMMARY

The importance of an understanding of the mechanism of strength generation in cohesive soils has been recognized for some time both by investigators in soil stabilization and in shear strength. The lack of good data on well controlled systems has hampered the verification of theoretical investigations.

In fine grained soils, strength generation is a result of submicroscopic interactions. One approach to the study and interpretation of these interactions is through their effect on the macroscopic behavior of the soil. Measuring the effective stresses and shear characteristics resulting from an alteration of inter-particle forces can provide fundamental insight into soil behavior.

The purpose of this investigation was to determine the effect of lime stabilization on the shear parameters of Vicksburg Buckshot Clay (VBC). These parameters were determined by running eight consolidated undrained triaxial tests with pore pressure measurements on compacted samples of VBC + 5% $Ca(OH)_2$. The samples were molded at a water content of 21% and dry density of 96 lbs/ft^3. They were cured for one week at 100% R.H. and immersed one day before testing. All samples were saturated prior to shear. Comparison tests were run on identical samples in unconfined compression. These samples were not saturated, however.

The major conclusions drawn from this investigation are the following:

1. Stabilization results in increased strength. At the same void ratio and water content, lime stabilized VBC is twice as strong as the unstabilized

soil.

2. Lime stabilization of VBC results in a high cohesion intercept (1.6 Kg/cm^2). and an increase in friction angle from 22^o to 32.5^o. These effects are thought to be caused by increased bonding between particles. Part of the increase in these parameters may also be due to pre-stressing during compaction and curing.
3. The mobilization of shear strength does not require the development of large negative pore pressures in stabilized soils.
4. Stabilization increases the rigidity and reduces the compressibility of a plastic soil.

It is recommended that the generality of the results and conclusions derived from this investigation be checked by similar tests on different soil-stabilizer systems.

Tests should be run using higher pressure to provide closer correlation with triaxial tests on natural clays and to permit testing of more rigid systems. The use of high pressures will require new equipment.

Significant additions and correlations with this report could also be obtained by running similar tests on partially saturated samples, measuring both water and air pore pressures. This would provide further insight into the relative importance of various components of soil behavior. Such an approach is also more realistic, since it studies stabilized soils as they are generally employed - in a partially saturated state.

II. INTRODUCTION

A. Background of Lime Stabilization

The improvement of the engineering properties of soils by the addition of chemical stabilizers has long been practiced. The early Romans employed lime stabilization in the construction of their roads. However, it was not until the development of modern soil mechanics in the late 1920's and early 1930's that reliable methods of design and construction, based on laboratory tests, were employed.

Concurrent with the advance of means of determining the effects of stabilization came a desire to understand the stabilization process, from a fundamental mechanistic point of view. The commonly observed effects upon adding small amounts of Calcium Hydroxide $C_a(OH)_2$ to soil are a reduction in plasticity and an increase in shear strength. The plasticity of fat clays is reduced to such an extent that they behave like coarse-grained, friable soils. The chemical reactions leading to these effects are not completely known. The probable mechanism is the following:

1) Rapid flocculation of colloidal soil particles caused by:
   a) Increased cation concentration
   b) Exchange of C_a^{++} ions for singly-charged cations such as N_a^{+}.
2) Relatively slow cementation or "pozzolanic action." This probably involves limited particle-to-particle cementation by reaction between C_a^{++} and reactive Alumina and Silica in the soil.

B. Mechanisms of Shear Strength in Compacted Clays

Previous investigations, both in soil stabilization and in the shear strength of cohesive soils, have indicated the necessity for an understanding of the mechanism of strength generation.

For the past several years much soil engineering research at M.I.T. has been directed towards the development of a mechanistic picture of shear strength, both in natural and in stabilized soils (1, 2).

An outgrowth of this research has been a growing conviction of the importance of the effective stress principle as a means of determining the influence of submicroscopic interactions on macroscopic soil properties.

Lambe (2) has shown how soil stabilization may be expected to give insight into soil behavior. The influence of stabilization on strength parameters such as cohesion, friction angle, and pore pressures may hopefully help elucidate the mechanism by which these components of strength are generated.

The purpose of this investigation was to determine the effect of lime stabilization on the shear parameters of a very plastic clay. These parameters were determined by running a series of consolidated undrained triaxial tests with pore pressure measurements. Extensive work has been done in the field of lime stabilization. Effects of time, concentration, additives, molding conditions, and curing conditions have been determined on a wide variety of soils. However, strength characteristics are usually determined by rapid, relatively simple tests such as unconfined compression or cone penetration. Data relating these various factors to shear strength and shear parameters in terms of effective stress is lacking.

III. PROCEDURE

A. Testing Program

The soil used in this investigation has been designated by M.I.T. as Vicksburg Buckshot Clay (VBC). Its properties are described in Appendix A. The stabilizer was reagent grade Calcium Hydroxide.

Eight consolidated undrained triaxial tests with pore pressure measurements were run on this soil + stabilizer system. An equal number of unconfined compression tests were run on identical samples.

Preliminary tests were run on VBC + 5% Cement. The effect of consolidation pressure on strength was quite small for this system at the pressures attainable using conventional equipment. This more rigid system was therefore considered less satisfactory than the soil-lime system for this investigation.

B. Sample Preparation

VBC at its natural water content of 8% was mixed with 5% $Ca(OH)_2$ (based on dry weight of soil). Distilled water was added to bring the water content to approximately 21%. This gave an initial degree of saturation of about 75%. Optimum water content for the density used in these tests is about 26%. The clay, lime, and water were then mixed in a finger blade mixer for five minutes. A weighed amount was placed in a mold and statically compacted from both ends in a hydraulic press. The compaction pressure used was that necessary to give a dry density of about 96 $lbs./ft.^3$. This was usually about 810 psi or 57 $kg./cm^2$.

The sample was then extruded. Weight, length, and diameter were measured before placing in a dessicator maintained at 100% relative humidity and room temperature. After curing for one week, samples were immersed in distilled water for twenty-four hours.

C. Testing Procedure

After soaking, weight, length, and diameter measurements were again taken. The degree of saturation was found to have increased to about 91%. The sample was then placed in a triaxial chamber and allowed to consolidate for one day under the desired consolidation pressure. Filter strips were used to facilitate drainage on six of the samples. They were not used in two of the tests. Volume changes and change in length of sample during consolidation were noted. After consolidation, the samples were back pressured in order to dissolve the remaining air. A pore pressure response of about 80% or greater was used as the criterion for high degrees of saturation. This usually required four kilograms per square centimeter water pressure. Tests were then run with constant pore pressure, varying the chamber pressure. A strain rate of approximately 1% per hour was used.

It should be noted that these tests were run on essentially saturated samples. Normally, strength tests for stabilized soils are run on samples which have been cured and soaked and are therefore only partially saturated. For example, the unconfined tests run in this investigation were at an average S of 91%.

Details of the test procedure and test equipment are described in Bishop and Henkel (3).

After testing, the samples were unloaded at constant water content. The final water content was determined by oven drying the samples for one day at 105°C.

IV. RESULTS

Test results are graphically and tabularly presented in Figures I through VIII and in Table I.

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Figure II	Mohr*s Circles in terms of Effective Stresses
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Figure IV	A Factor, Strength vs. Consolidation Pressure
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Table I	Summary of Test Data

VBC + 5% $Ca(OH)_2$

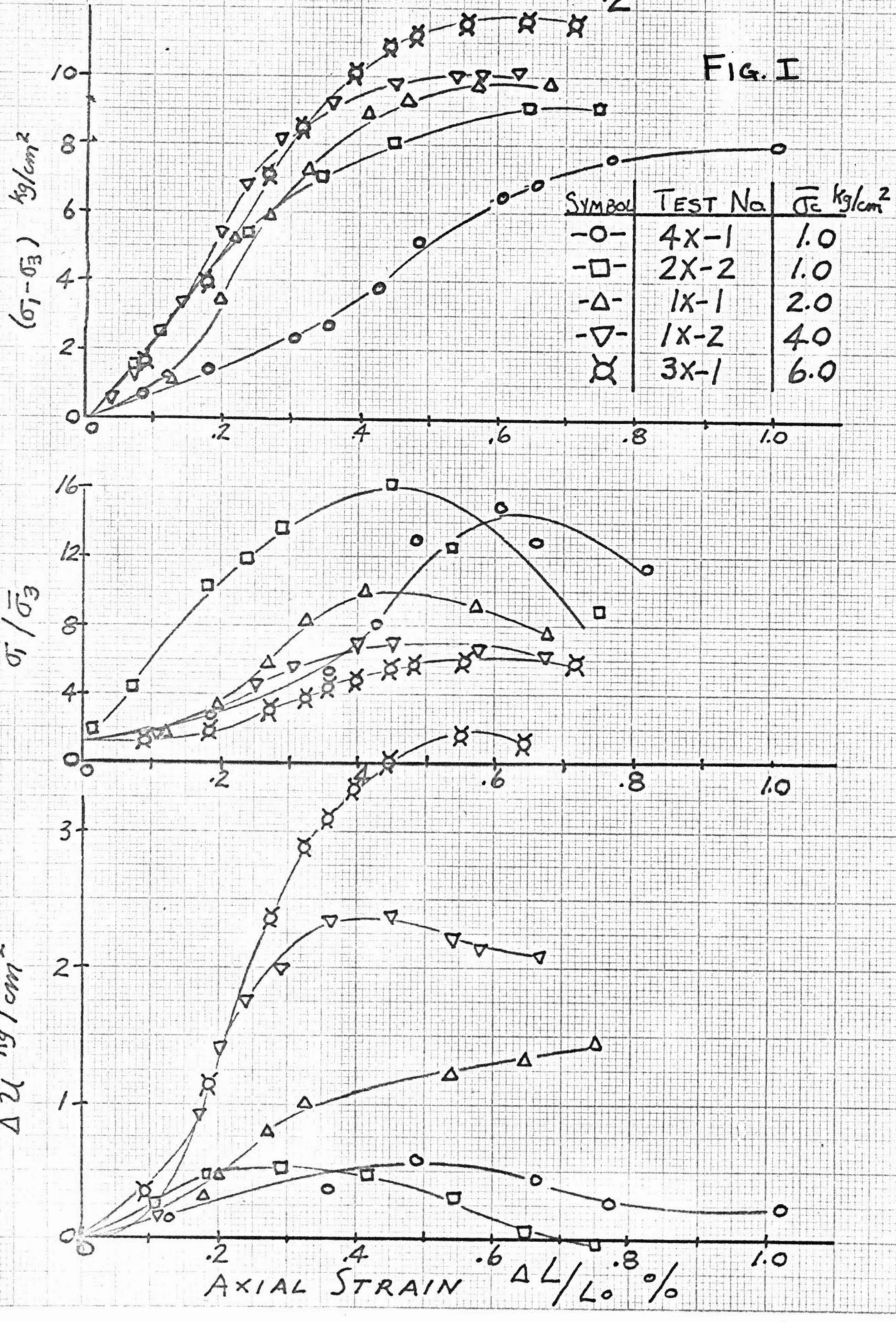
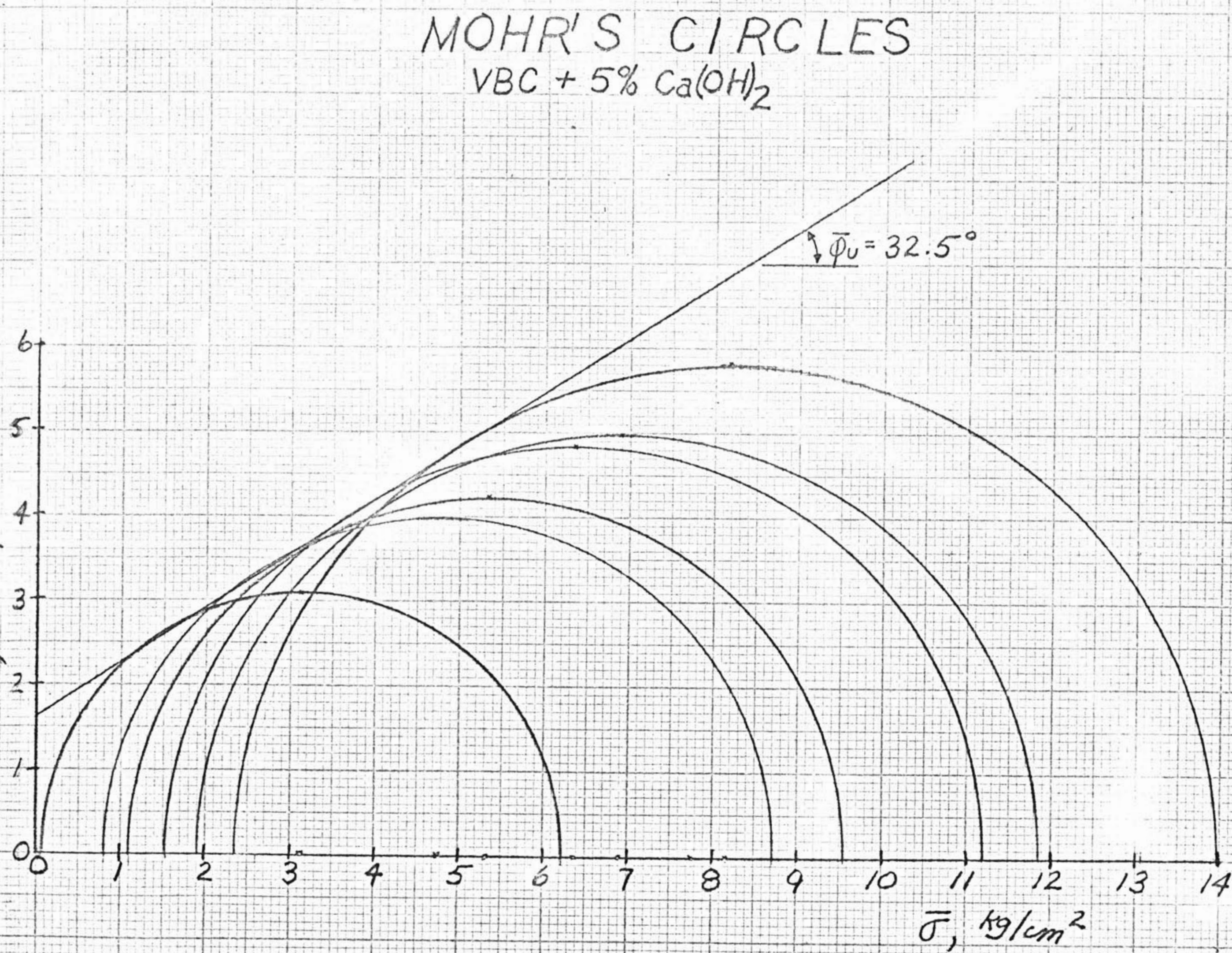
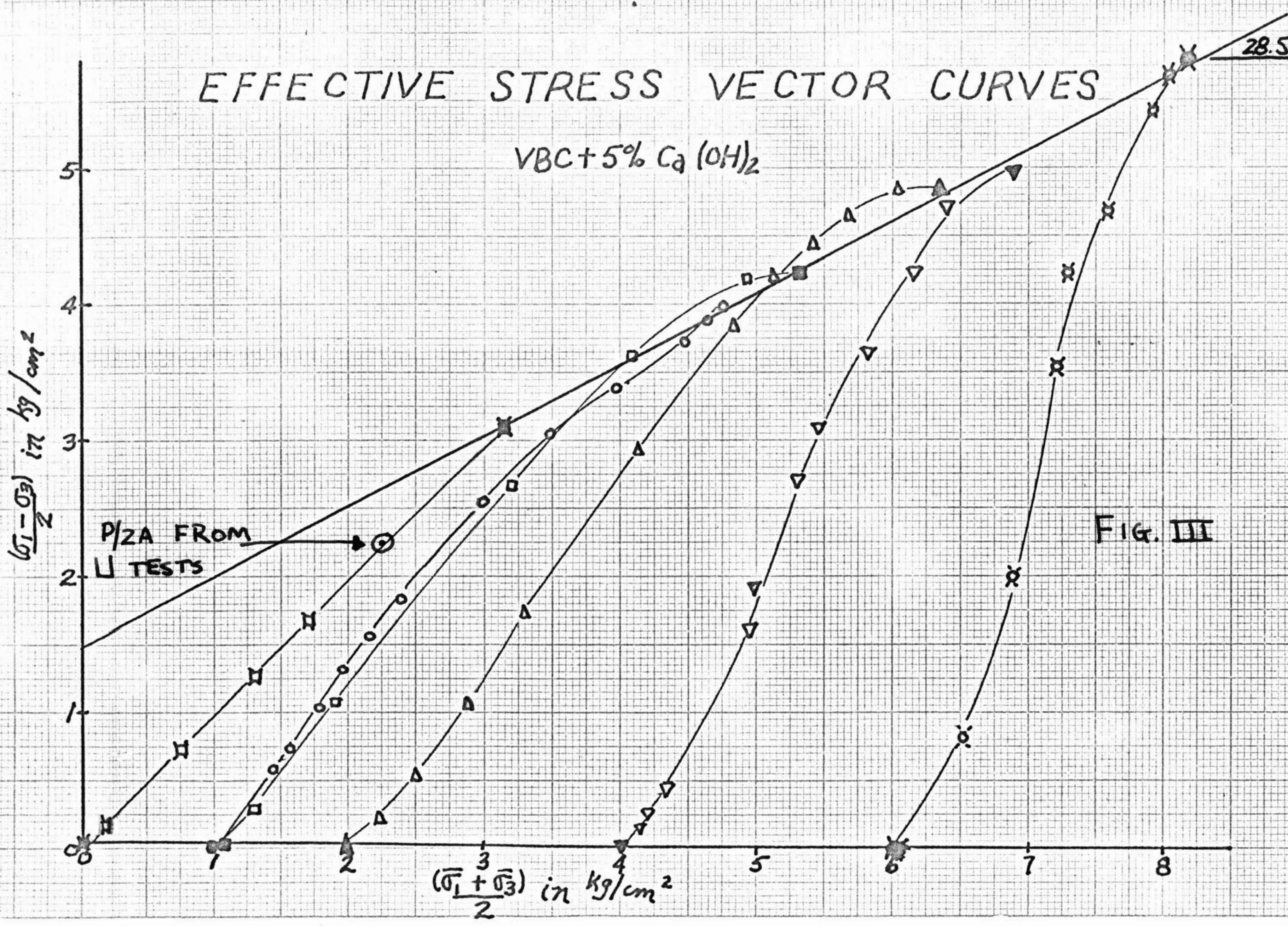


FIG. I



MOHR'S CIRCLES
VBC + 5% Ca(OH)2
$\bar{\varphi}_u = 32.5°$
τ, kg/cm²
σ̄, kg/cm²
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14

FIG. II

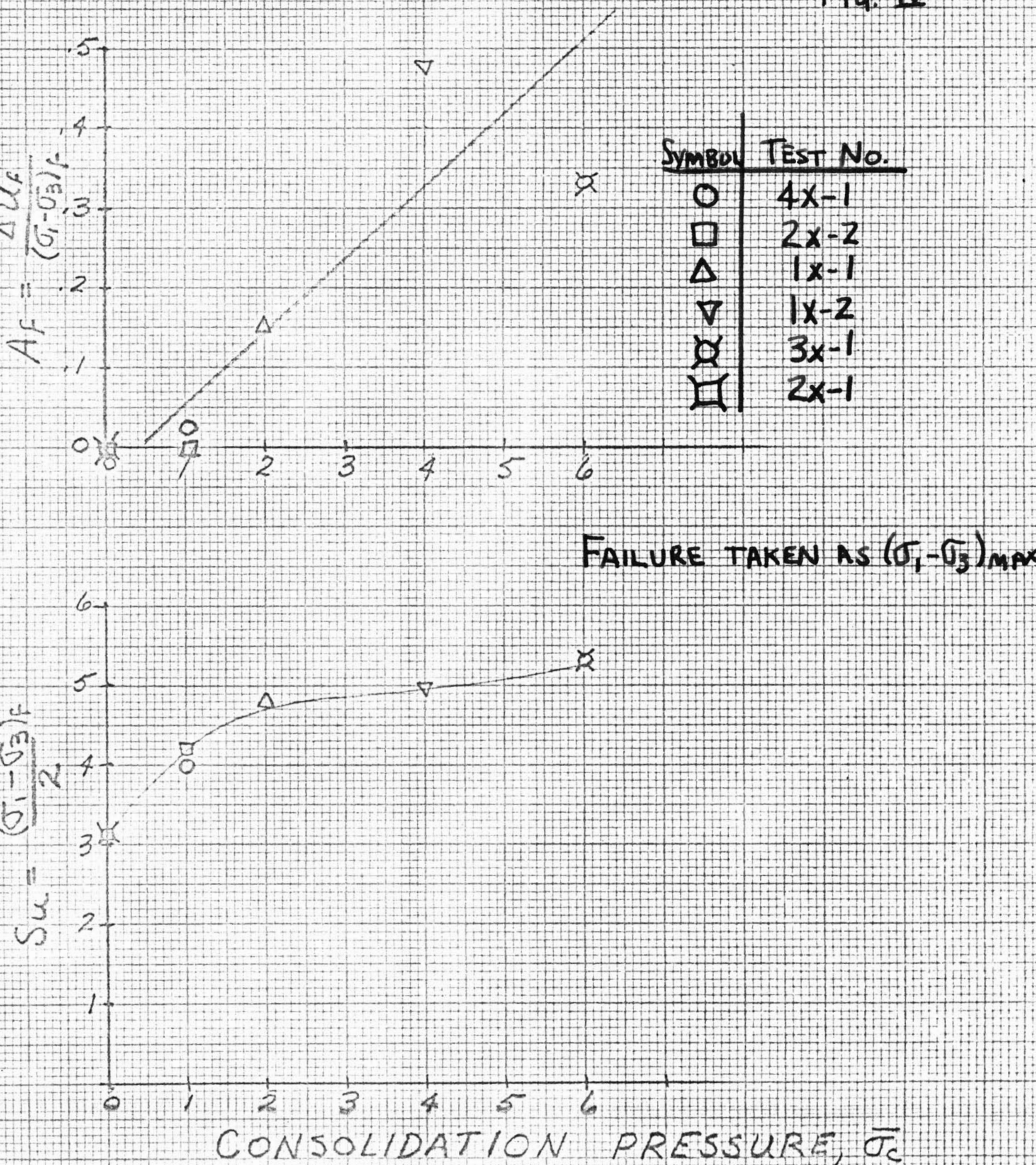


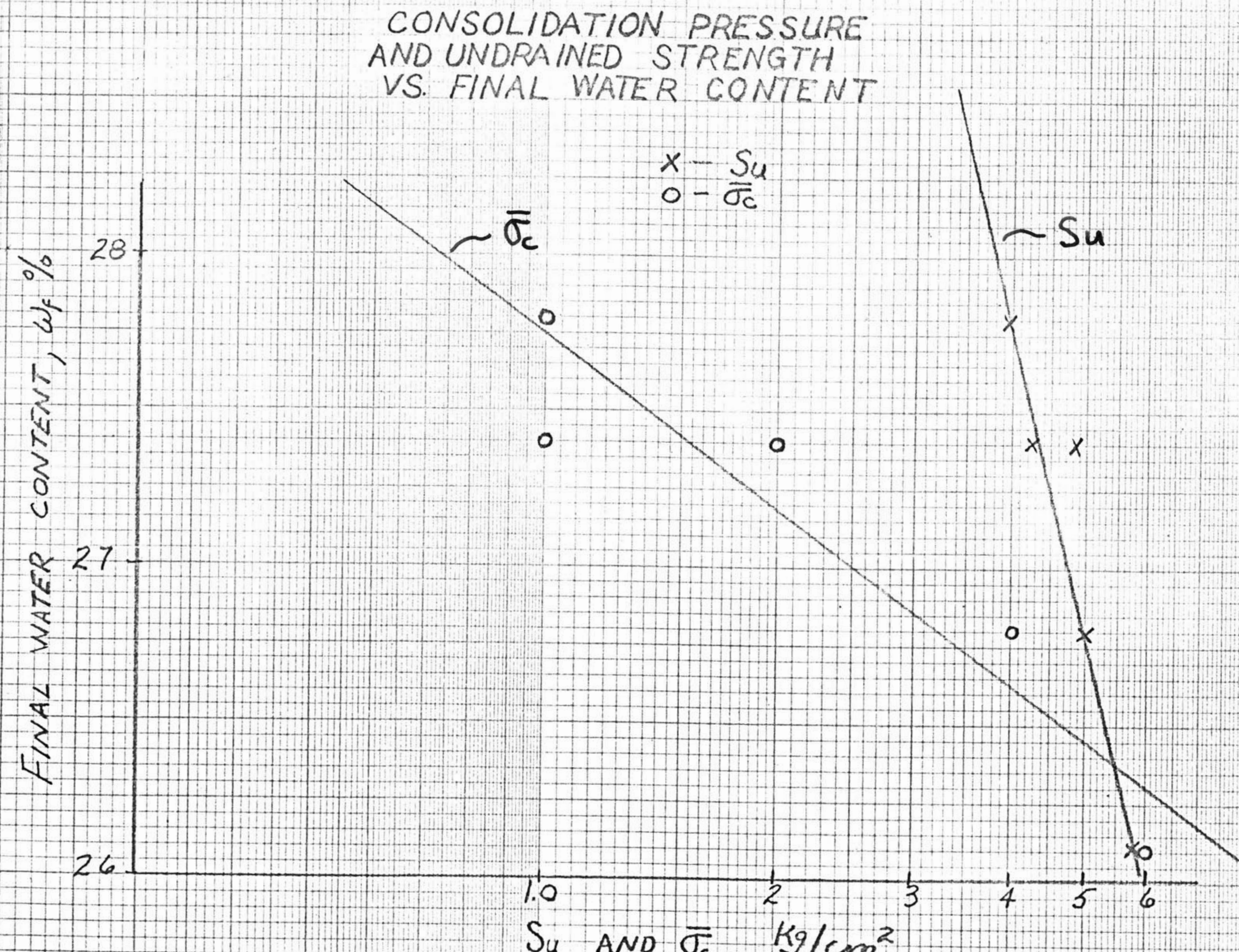
EFFECTIVE STRESS VECTOR CURVES
VBC + 5% $Ca(OH)_2$
28.5°
P/2A FROM
U TESTS
$\frac{(\sigma_1 - \sigma_3)}{2}$ in kg/cm^2
$\frac{(\sigma_1 + \sigma_3)}{2}$ in kg/cm^2
0
1
2
3
4
5
6
7
8

FIG. III

A-FACTOR AND UNDRAINED STRENGTH
VS. CONSOLIDATION PRESSURE
VBC + 5% $Ca(OH)_2$

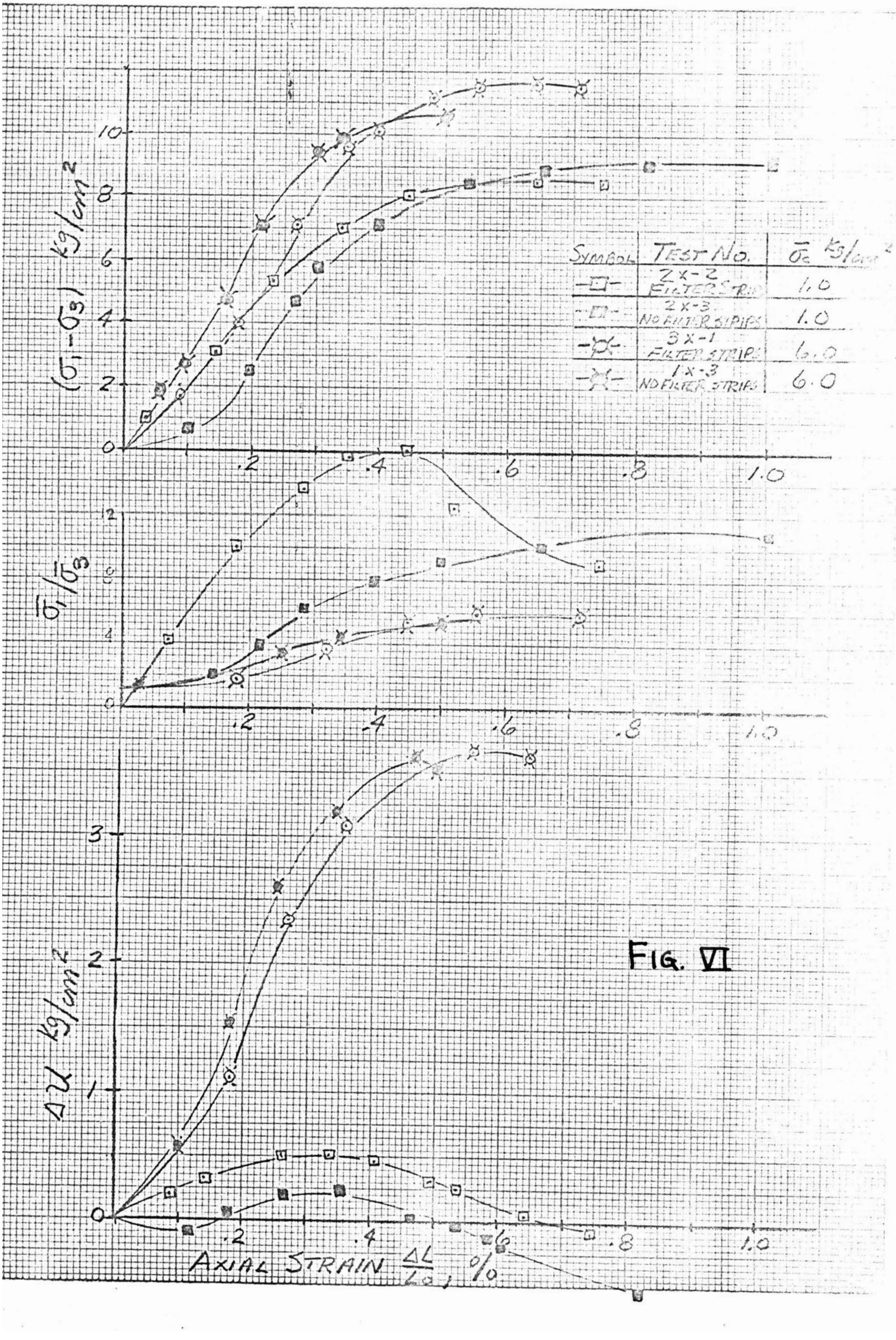
FIG. IV





CONSOLIDATION PRESSURE
AND UNDRAINED STRENGTH
VS. FINAL WATER CONTENT
X – Su
O – $\bar{\sigma}_c$
$\bar{\sigma}_c$
Su
FINAL WATER CONTENT, w_f %
28
27
26
1.0
2
3
4
5
6
Su AND $\bar{\sigma}_c$, Kg/cm²

FIG. V



10
8
6
4
2
0
(σ1 - σ3) kg/cm²
.2
.4
.6
.8
1.0
SYMBOL
TEST NO.
σc kg/cm²
2X-2 FILTER STRIPS
1.0
2X-3 NO FILTER STRIPS
1.0
3X-1 FILTER STRIPS
6.0
1X-3 NO FILTER STRIPS
6.0
12
8
4
0
σ1/σ3
3
2
1
0
Δu kg/cm²
AXIAL STRAIN ΔL/Lo, %

FIG. VI

STRENGTH of NATURAL & STABILIZED VICKSBURG BUCKSHOT CLAY

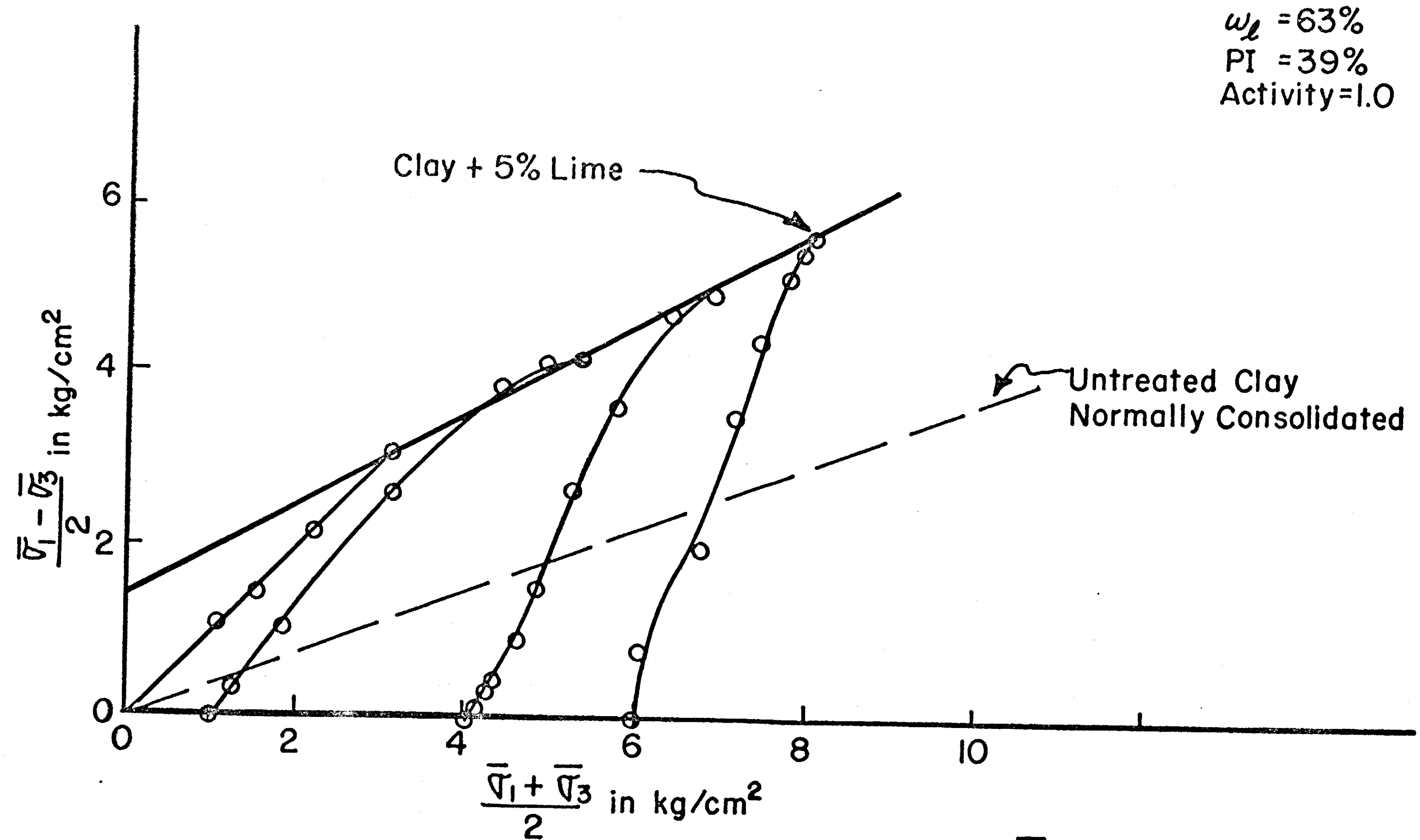


FIG. VII [redacted]

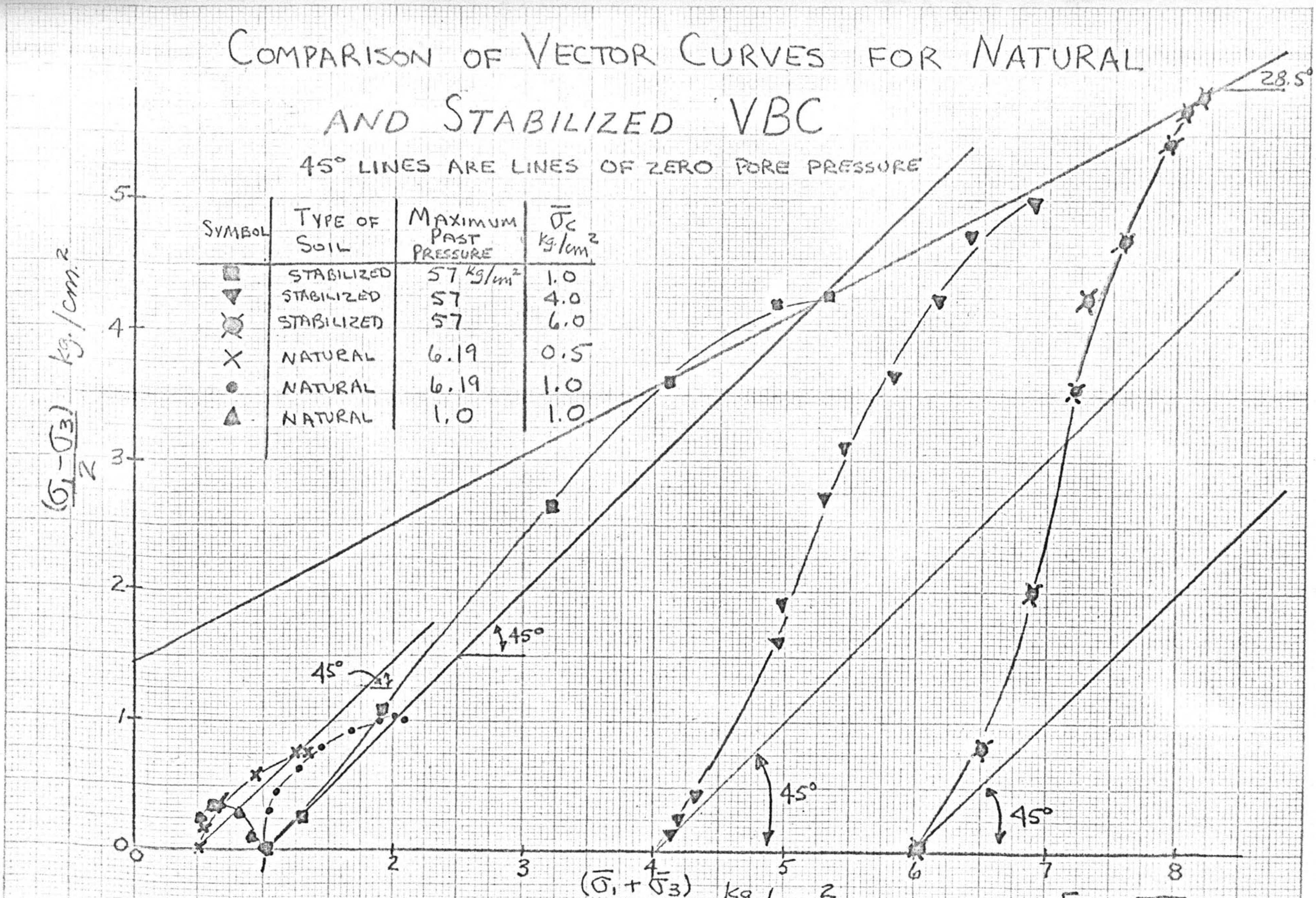

COMPARISON OF VECTOR CURVES FOR NATURAL
AND STABILIZED VBC
45° LINES ARE LINES OF ZERO PORE PRESSURE
SYMBOL
TYPE OF SOIL
MAXIMUM PAST PRESSURE
$\bar{\sigma}_c$ kg./cm.²
STABILIZED
57 kg/cm²
1.0
STABILIZED
57
4.0
STABILIZED
57
6.0
NATURAL
6.19
0.5
NATURAL
6.19
1.0
NATURAL
1.0
1.0
28.5°
45°
$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{2}$ kg./cm.²
$\frac{(\bar{\sigma}_1 + \bar{\sigma}_3)}{2}$ kg./cm.²
0
1
2
3
4
5
6
7
8

FIG. VIII

SUMMARY OF TEST DATA

TEST NO.	AS MOLDED w	AS MOLDED S	AS MOLDED γ_d	FINAL w	FINAL S	FINAL e	$\bar{\sigma}_c$	PORE PRESSURE RESPONSE	AT $(\sigma_1-\sigma_3)_{max}$: ε_f	$(\sigma_1-\sigma_3)$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\Delta u/(\sigma_1-\sigma_3)$	$\frac{\bar{\sigma}_1-\bar{\sigma}_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$
2X-1	21.2	76.5	96.1	27.9	101	0.74	0	94%	0.32	6.20	156	-.006	3.10	3.14
2X-2	21.0	74.6	954	27.45	97.7	0.76	1.00	>80%	0.71	8.49	8.9	-.005	4.24	5.32
4X-1	20.6	74.7	96.0	27.8	100	0.74	1.00	88%	1.07	796	11.2	.028	3.98	4.76
1X-1	20.9	75.5	96.0	27.4	98.7	0.74	2.00	80%	0.68	9.71	7.47	.155	4.85	6.36
1X-2	20.75	76.2	96.5	26.8	98.4	0.73	4.00	80%	0.63	9.96	6.15	.476	4.98	6.90
3X-1	19.1	71.4	97.5	26.1	97.5	0.72	6.00	77%	0.64	11.62	5.93	.314	5.81	8.17

V. DISCUSSION OF RESULTS

The following points should be considered regarding the data presented in Section IV:

1. Undrained Strength.

A comparison of the strength of VBC + 5% lime with that of the natural soil show the stabilized soil to be appreciably stronger. Hoyt (4) ran a series of consolidated undrained (CU) tests on sedimented VBC. The following is a comparison of his results with the lime-stabilized soil:

Natural Soil				Stabilized Soil			
$\bar{\sigma}_c$ kg./cm.^{2}	w_f	e_f	S_u kg./cm.^{2}	$\bar{\sigma}_c$ kg./cm.^{2}	w_f	e_f	S_u kg./cm.^{2}
1.50	39.2	1.07	0.5	1.00	27.8	0.74	4.0
6.19	29.3	0.78	1.6	6.00	26.1	0.72	5.8
---	*27.8	0.75	2.0	1.00	27.8	0.74	4.0

At the same void ratio and water content, the lime-stabilized soil is twice as strong as the natural VBC.

2. Shear Parameters $\bar{\Phi}_u$ and c.

Lime-stabilized VBC has a $\bar{\Phi}_u$ of 32.5° and a cohesion intercept (c) of 1.6 kg./cm.^{2} (Fig. II). da Cruz (5) found that for natural VBC $\bar{\Phi}_u$= 22° and c= 0.

Similar results are reported by Lambe (2) on a clayey silt stabilized with lime. Stabilization resulted in an increase of strength angle from 37° to 45°. The increase in cohesion intercept, however, was very slight for this soil.

---

*Extrapolated from test results.

Both the increase in $\bar{\phi}_u$ and the larger intercept are indications of increased cohesion. When small particles are bonded together to form larger ones, the material behaves like a coarse-grained soil with a high friction angle rather than as a fat clay. In addition the soil may show a component of strength independent of external stresses. While the reason for this stress independent cohesion is not clear from the available data, it is probably related to the extent of soil-lime interaction and the magnitude of the bonding forces resulting from this interaction. Fine-grained, plastic soils might, therefore be expected to show this effect to a larger extent than silts.

3. Effective Stress Path during Shear.

Stress vectors are plotted in Figure III interms of average normal effective stress and average shear stress. In Figure VIII stress vectors for stabilized and unstabilized VBC are compared. The data for the unstabilized tests were obtained by Hoyt and da Cruz.

The effective stress path for stabilized VBC closely resembles the path for overconsolidated natural VBC. At $\bar{\sigma}_c = 1.0$ the stabilized VBC has an apparent overconsolidation ratio between 6 and 12 kg./cm^2. The decrease in curvature of the stabilized sample curves at higher consolidation pressures indicates decreasing overconsolidation ratios. This is the effect one would expect if all of the samples were subjected to a uniform prestressing before shear.

During compaction the stabilized samples were subjected to a vertical stress of about 57 kg./cm^2. The average effective stress probably reached the necessary level to produce the effects of high overconsolidation.

It should also be noted that overconsolidated samples of natural VBC have a friction angle of 25^o, three degrees higher than normally consolidated samples. A part of the increase in friction angle for stabilized VBC can therefore be attributed to overconsolidation or prestressing.

4. Increase in Stress-Strain Modulus.

The slope of the stress-strain curve in Figure I indicates an average modulus for lime-stabilized VBC of about 25 kg./cm^2. This is about 8 times higher than unstabilized VBC in the same consolidation pressure range (2 - 6 kg./cm^2.). In addition, all failures were brittle and along a well-defined failure plane.

Lime stabilization probably contributes to the more rigid structure of this system. However, compaction on the dry side of optimum normally results in an increased stress-strain modulus, so the rigidity may be largely due to compaction rather than chemical stabilization.

5. Compressibility.

Lime stabilization greatly reduces the compressibility of the soil skeleton. This is shown by the void ratios in Table I and the flat slope of the water content vs. consolidation pressure curve in Figure V.

From the void ratio values in Table I, a compressibility (C_s) of the soil skeleton may be calculated of 1/5000 in.^{2}/lb. The value for natural VBC is approximately 1/350 in.^{2}/lb.(6). Thus stabilization in this case reduced the compressibility by a factor of 14.

6. B Factor.

The pore pressure response was measured as $\Delta u/\Delta\sigma_3$ for an increment of chamber pressure (σ_3). The values listed in Table I are those resulting from the final increment of back-pressuring, usually from 3 kg./cm^2. to 4 kg./cm^2.

Skempton's pore pressure parameter B is defined by the following equation:

or numerically

B = pore pressure response

B as measured in these tests was always less than 1. There are two possible explanations for this:

A. B 1 for saturated lime-stabilized VBC. This is not what one would expect from theoretical considerations. B may be expressed in the following way:

$$B = \frac{1}{1 + \frac{nC_p}{C_s}}$$

where

C_p = Compressibility of pore fluid

C_s = Compressibility of soil skeleton

n = Porosity

Taking data from table I, for the lime-VBC system:

$$C_s = -\frac{\Delta V}{V_1}\frac{1}{\Delta p} = -\frac{\Delta e}{1+e_1}\frac{1}{\Delta p} = \frac{.74-.72}{1+.74}\frac{1}{(6-2)\,14.2}$$

$$= \frac{1}{4950}\ \text{in.}^2/\text{lb.}$$

$$C_p = \frac{1}{300{,}000}\ \text{in.}^2/\text{lb.} \quad \text{for pure water } (s = 100\%)$$

$$n = \frac{e}{1+e} = \frac{.74}{1.74} = 0.425$$

$$B = \frac{1}{1 + \frac{.425\ {}^1/_{300{,}000}}{{}^1/_{4950}}} = \frac{1}{1.007}$$

$$\boxed{B = 0.994}$$

B. B was not measured for a saturated system. This is probably the best reason. It should be remembered that the B factor was always measured on the last increment of back pressuring. It is probable that the degree of saturation was always less than 100% on this increment, even when S was measured to be 100% at the end of the test. Further support for the view that a B factor of 1 could have been measured was provided by some preliminary tests on VBC + 5% cement. In two of these tests, pore pressure responses of 96% and 100% were measured when the samples were back pressured to 7 kg./cm^2. It was not possible to employ this high a back pressure for samples consolidated to pressures greater than 2 kg./cm^2., however.

7. A Factor.

The A factor measured in these tests was typical of lightly overconsolidated clay or a compacted clay-gravel(Fig. IV). The possibility of induced prestressing causing such behavior has already been discussed in part 3 of this section.

8. Influence of Pore Pressures on Strength

Triaxial test 2x - 1 was run with zero consolidation pressure. The sample was back pressured, however, and thus had a confining pressure of 4 kg./cm^2. The maximum pore pressure which developed during this test was -0.04 kg./cm^2. The effective stress path is plotted in Figure III. It is seen to agree with the other tests, reaching the same failure line.

This test indicates that negative pore pressures are not important as a mechanism for strength generation in lime-stabilized VBC. Further support for this is provided by the results of unconfined tests, descrived in part 9 of this section.

The lack of large negative pore pressures would tend to support the concept of a true cohesion or strength at zero effective stress for this soil system.

9. Unconfined Compression Tests

A series of unconfined compression (U) tests were run on samples prepared identically to those used in the triaxial tests. However, the unconfined samples were not back pressured prior to shear, and thus were tested at a lower degree of saturation. Also, the strain rate in these tests was about 20 times faster than in the triaxial tests.

The average strength of these samples was P/2A = 2.25 kg./cm^2. This is about 0.75 kg./cm^2. lower than the strength of the unconsolidated sample tested in the triaxial apparatus. Both the lower degree of saturation and the higher strain rate would be expected to increase the strength rather than decrease it. The higher strength of the triaxial sample may, however, be due to the confining pressure to which it was subjected for back pressuring.

The failure point for the unconfined test samples in terms of total stress is indicated in Figure III. If one assumes that the point would fall on the failure envelope if it were plotted in terms of effective stresses, the sample must have had a positive pore pressure at failure of about 0.75 kg./cm^2. (horzontal distance between point and envelope). This is rather unorthodox behavior for a soil. However, the high cohesion intercept lends support to the possibility of its occurrence. Still, the more likely explanation would appear to be the effect of confining pressure.

10. Effect of Filter Strips on Strength and Pore Pressures.

Two triaxial tests were run without filter strip drains. The results of these tests are compared with similar tests using filter strips in Figure VI. There does not appear to be any consistent trend to the differences between the samples. The fact that one test without filter strips showed higher pore pressures while the other gave lower casts suspicion the reproducibility of pore pressures from sample to sample. The pore pressure agreement between Test 4X - 1 and Test 2X - 2 is rather poor also. This might be expected since pore pressure development and dissipation occurs at many points during a sample's stress history.

11. Discussion of Errors

A. Influence of rubber membranes and filter strips on measured strength.

The contribution of the rubber membranes and filter strips to the shear strength is felt to be negligible. A correction of 0.1 kg./cm^2. is sometimes used in work with soft clays. This is a small part of the cohesion intercept of VBC + 5% $Ca(OH)_2$. Furthermore, it is doubtful if this strength could be completely mobilized at the low values of strain necessary to fail these samples.

B. Piston friction

The error due to piston friction has been neglected in this investigation. Since sample deformation was very small, large lateral loads were probably not transmitted to the loading piston. In addition, the piston was well lubricated at all times.

C. Leakage through rubber membranes or in pore pressure measuring device.

Membrane leakage was minimized by using two membranes with silicone grease between them. In addition, the cell water was deaired to prevent air diffusion through the membranes.

To ensure that leaks did not occur in the pore pressure measuring system, samples were back pressured for twenty-four hours. Before testing, the null indicator was carefully checked for a no-movement condition, indicating that no leaks were present.

VI. CONCLUSIONS

1. Addition of 5% $Ca(OH)_2$ to Vicksburg Buckshot Clay increases $\bar{\phi}u$ from 22° to 32.5° and increases the cohesion intercept from zero to 1.6 kg./cm.^{2}. These effects are thought to be caused by cementing small particles together to form larger ones. Part of the increase in these shear parameters may also be due to prestressing during the compaction and curing process.
2. The mobilization of shear strength does not depend on the development of large negative pore pressures in lime-stabilized VBC. This lends support to the theory of a true cohesion or strengh at zero effective stress for some stabilized soil systems.
3. Lime stabilization makes the soil skeleton more rigid..Samples fail at lower strains. The stress-strain modulus is increased by almost a factor of 10.
4. Stabilization decreases the compressibility of the soil by about a factor of 14. There is very little void ratio change with changes in effective stress. However, the soil skeleton is not so incompressible as to preclude the possibility of having B = 1.
5. Compacted lime-stabilized VBC has stress characteristics typical of overconsolidated soils. The stress vedtor curves are similar to those of overconsolidated samples of unstabilized VBC. The major part of this effect is probably due to the high pressures used to compact the samples. The curing process, involving soil-lime reaction and partial drying of the sample, may also contribute to prestressing.

VII. RECOMMENDATIONS

It is recommended that similar tests be run on different soil-stabilizer systems to check the generality of the results and conclusions derived from this investigation. Triaxial tests on cement stabilized soils would provide important additions to and correlations with this report. However, work on these less compressible systems will require new equipment, capable of accurate control at much higher pressures. This equipment would also permit testing at a $S_u/\bar{\sigma}_c$ ratio similar to that obtained in triaxial testing of unstabilized clays.

It is further recommended that tests be run on partially-saturated stabilized samples. Practical applications for soil stabilization almost always involve unsaturated soils. Furthermore, the influence of partial saturation on strength and shear parameters would provide further insight into soil behavior. However, an effective stress analysis of these systems would require control and measurement of both water and air pore pressures. This would necessitate the design and construction of new equipment, for tests of this nature have never been run on stabilized soils.

APPENDIX

A. PROPERTIES OF VICKSBURG BUCKSHOT CLAY

The soil used in this investigation was obtained from the U. S. Army Corps of Engineers, Waterways Experimental Station, Vicksburg, Mississippi. It is a silty clay found in the flood plains of the Mississippi Valley.

Average properties of the soil are as follows:

<u>MIT Grain Size Classification</u>

	%
Sand	0
Silt	65
Clay	35

<u>Physical Properties</u>

Liquid Limit, %	60
Plastic Limit, %	28
Plasticity Index, %	32
Specific Gravity [Soil + 5% $Ca(OH)_2$]	2.68

<u>Mineralogical Composition</u>

Clay Composition, %	50
Illite: Montmorillonoid: Chlorite	1:1:0
Free Iron Oxide, % Fe_2O_3	1.9

NOMENCLATURE

Skempton's A factor	A	---------
A factor at failure	A_f	---------
Skempton's B factor	B	---------
Cohesion intercept	c	kg./cm.^{2}
Void ratio	e	---------
Specific gravity	G	---------
Degree of saturation	S	per cent
Undrained shear strength	S_u	kg./cm.^{2}
Average pore water pressure	u	kg./cm.^{2}
Applied back pressure	u_{BP}	kg./cm.^{2}
Water content relative to dry weight	w	per cent
Initial or as-molded water content	w_i	per cent
Final water content	w_f	per cent
Angle of stress obliquity	α	---------
Dry weight of solids	γ_d	lbs./ft.^{3}
Axial strain	ϵ	---------
Axial strain at failure	ϵ_f	---------
Normal effective stress	$\bar{\sigma}$	kg./cm.^{2}
Consolidation pressure	$\bar{\sigma}_c$	kg./cm.^{2}
Total principal stresses	σ_1, σ_3	kg./cm.^{2}
Effective principal stresses	$\bar{\sigma}_1, \bar{\sigma}_3$	kg./cm.^{2}
Shear stress	τ	kg./cm.^{2}
Angle whose tangent gives relationship between available shear strength and total normal stress	ϕ	---------
Effective stress measured in undrained triavial tests with pore pressure measurements	$\bar{\phi}_u$	---------

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