DISCUSSION

D. F. COATES[1]—Following the General Reporter's comments on his interest in having more information on the effects on specimens of the size of piston samplers, Mr. McRostie and I would like to supplement and summarize the data on our sampling investigations. The following comparisons are for peak principal stress differences where laboratory tests are involved:

1	Block specimens.........	in Qc tests	50% to 80% > 2 in. piston
		in Q tests	200% > 2 in. shelby
2	Piston samplers.........	3½ in. diameter in Qu tests	25% > 2 in. shelby
		2 in. diameter in Qu tests	0 to 25% > 2 in. shelby
		3½ in. diameter preconsolidation	15% > 2 in. shelby
3	Field vane..............	vs Q tests	40% to 70% > 2 in. shelby
		vs Qc tests	= 2 in. piston
4	Pocket penetrometer.....	vs Q, on 2 in. shelby	= 2 in. shelby
		vs Q, on block	40% < block

In spite of the important differences owing to sampling procedures, these comparisons should be viewed in the light of the probable variation of actual strength. From experience with testing other materials, it seems unlikely that any *in situ* or compacted soil would have a single value for cohesion or friction angle. They might have means for these parameters with possibly symmetrical dispersions about the means.

For example, job-mix concrete can produce test specimens with strengths that have coefficients of variation more than 20 per cent. If on one project the coefficient of variation is 20 per cent, then approximately $\frac{2}{3}$ of any series of test results would lie within a range where the maximum test strength would be 50 per cent greater than the minimum test strength. Similar results would be found in testing the yield point, which is considered by many to be a unique property of structural steel. Also the ultimate strength of a series of steel specimens would normally have higher coefficients of variation than the yield point.

Suppose then in a uniform soil we had a coefficient of variation of strength of 25 per cent. It would be found by competent testing that about $\frac{2}{3}$ of the strength values would lie within a range where the maximum was 70 per cent greater than the minimum.

From these observations we would conclude that studies should be conducted on actual dispersions of strength parameters that can occur, and on the engineering aspect of adjusting the safety factor to take into account the probable dispersion of strengths of a foundation or embankment soil.

Another aspect that we believe may warrant a comment is that test results obtained from conditions beyond a certain critical strain, particularly in sensitive soils, are not representative of the

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in situ soil. In other words, the test specimen has been significantly changed so that its strength parameters are no longer representative. This aspect has led us to look with favor on a yield point criterion such as Housel as advocated and as elaborated on in our paper.

W. G. HOLTZ[2]—*Effect of Sampling Procedures on Strengths of Natural Clays in Terms of Total and Effective Stresses:*

Remolding—The effects of remolding clayey soils during the sampling are often obvious. For instance, when specimens are taken in such a manner that the sides are significantly remolded and the specimen is merely extracted from the tube and then used for shear testing without trimming, the structure is definitely affected. The answer to this is the use of sufficiently large specimens so that the shear specimens can be trimmed to remove all obviously remolded material, or samplers with proper wall thickness and clearances may be used. Consideration should always be given to the use of hand-cut specimens or a double-barrel core sampler to eliminate driving disturbance when this is critical.

Densification—Densification is a particular problem when sampling loose unsaturated soils. The remedy for this is the use of very short drives with proper clearance within the drive sampler tube. Specimens taken with double-barreled core samplers, such as the Denison sampler, or hand-cut specimens, are often required in critical situations.

Moisture Change—Particular care is required in sampling loose, unsaturated soils and unsaturated expansive clays. When the moistures of loose soils and expansive clays are increased, the soil strengths may be drastically changed. Figure 9 shows the variations in shear

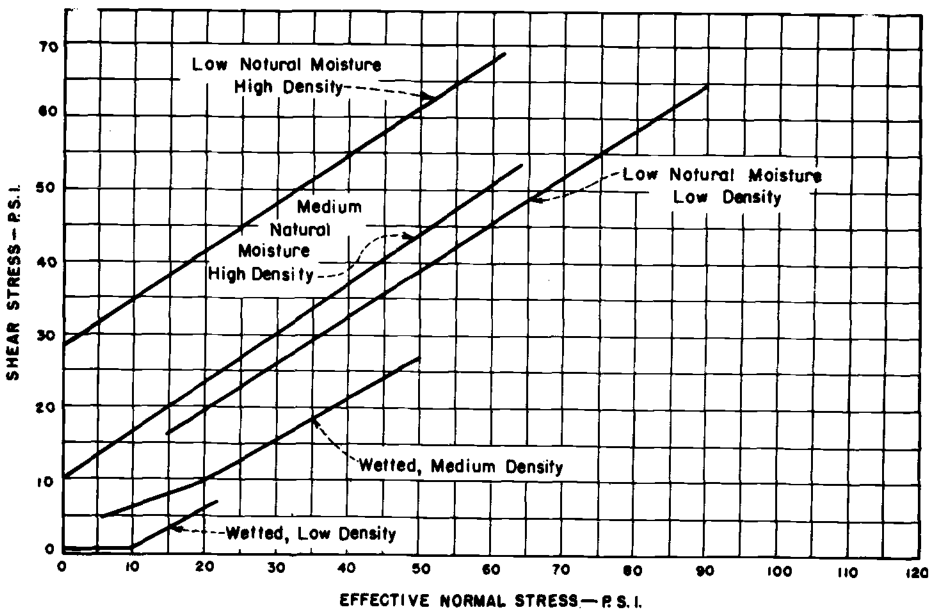

FIG. 9—Variations in Shear Strengths of Lean Clay Soils of Loessial Origin.

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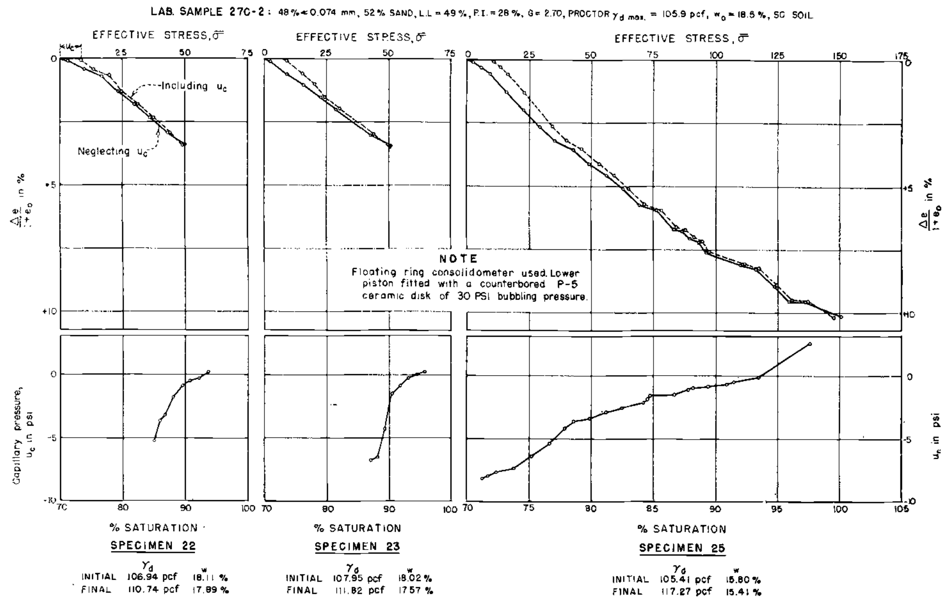


FIG. 10—Results of Confined Compression Tests With Pore Pressure Measurements.

SPECIMEN NO.	DRY DENSITY (PCF)			MOISTURE CONTENT (%)		DEGREE OF SATURATION (%)		TEST VALUES AT FAILURE					SHEAR VALUES CORRECTED FOR PORE PRESSURE	
	PLACEMENT	WETTED	CONSOLIDATED	PLACEMENT	WETTED	PLACEMENT	WETTED	PORE PRESSURE (PSI)	EFFECTIVE LATERAL PRESSURE (PSI)	VOLUME CHANGE (% OF INITIAL)	AXIAL STRAIN (%)	DEVIATOR STRESS (PSI)	TAN Ø	COHESION (PSI)
4	116.8	—	118.2	13.6	—	81.5	—	(12.0 / 6.5)	(12.5 / 18.0)	−3.00	17.73	61.4		
5	116.0	—	118.3	13.5	—	79.4	—	(24.0 / 17.9)	(24.5 / 30.6)	−4.81	19.72	76.9		
6	115.2	—	118.7	13.1	—	75.4	—	(61.9 / 57.6)	(50.9 / 55.2)	−7.14	23.09	115.1	(0.47 / 0.58)	(16.0 / 10.0)

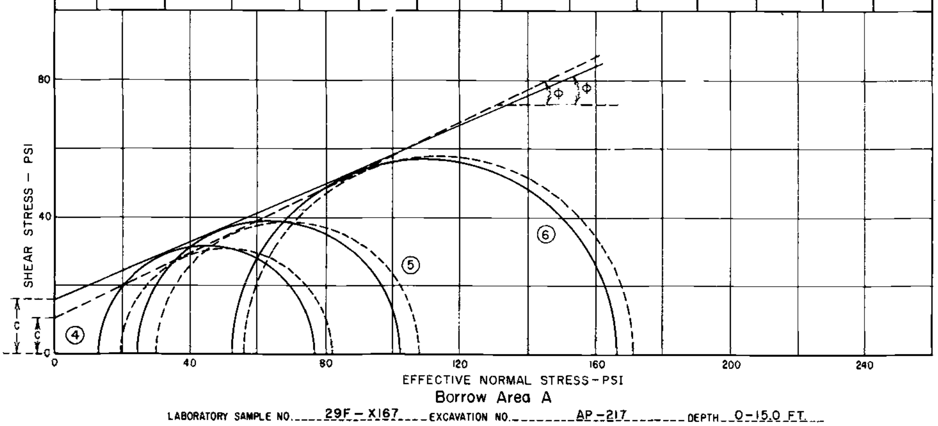


FIG. 11—Shear Strength of Compacted Cohesive Soil.

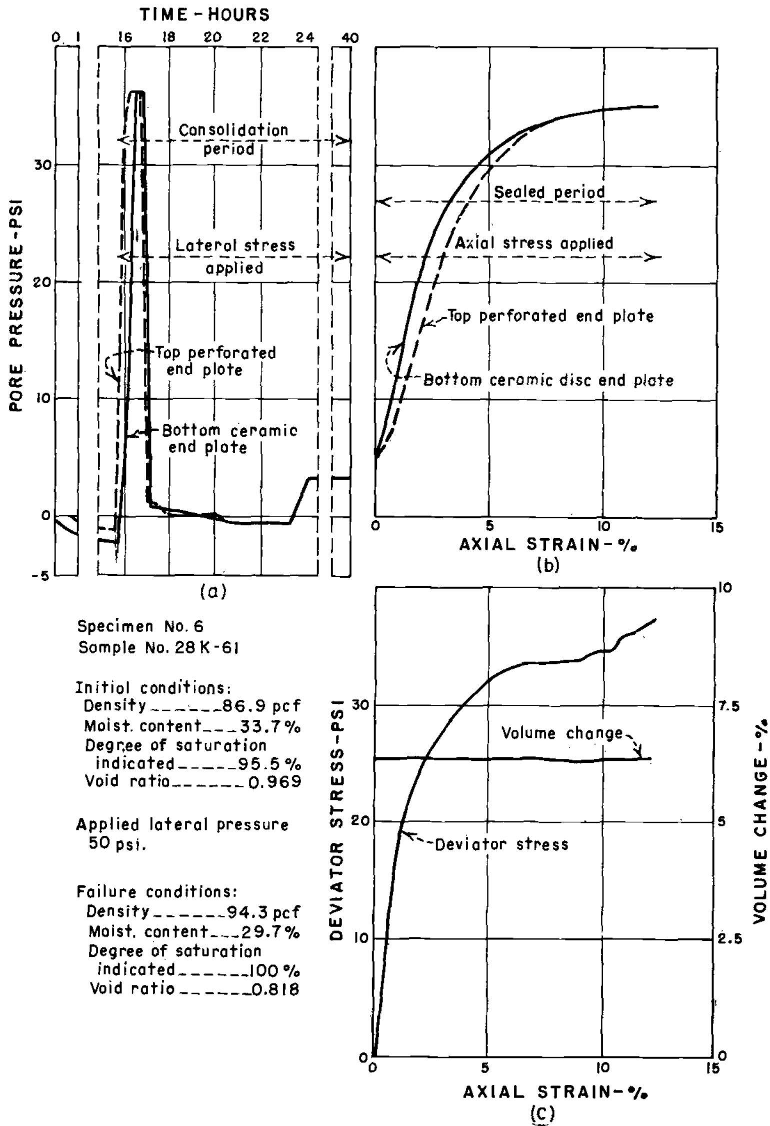
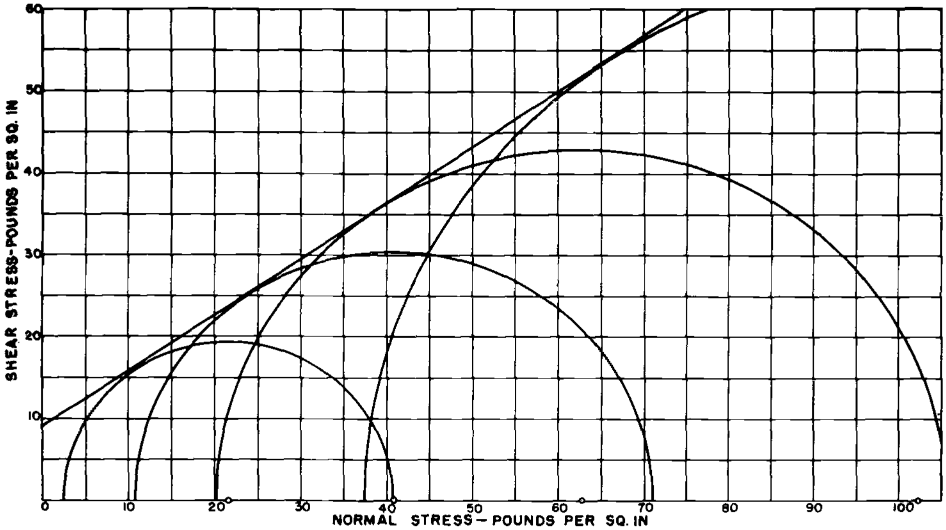


FIG. 12—Negative Pore Pressures for a Lean Clay Soil Below Water Level.

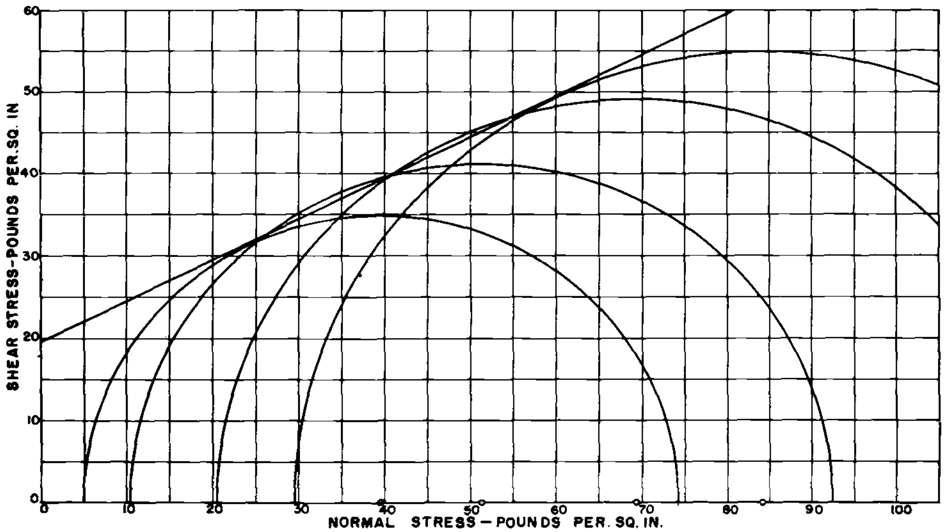
strengths of lean-clay soils of loessial origin from Nebraska which can be obtained with variations in density and moisture. Very high shear strengths are obtained for dense and dry soils, while extremely low shear strengths are obtained for loose and wet soils. When expansive clays are wetted during the sampling operation, the shear strengths, as well as the expansive characteristics of the soils, cannot be properly evaluated. In these cases, the answer, with respect to deep sampling, is the use of a very heavy drilling mud or air in a double core-barrel drilling operation with the inner barrel extended into the soil. For



Specimen No.	16	17	19	18
Placement Dry Density (p.c.f.)	111.94	112.04	112.42	112.25
Failure Dry Density (p.c.f.)	112.06	113.83	116.82	120.99
Placement Moisture Content (%)	11.43	11.38	11.35	11.39
Applied Lateral Pressure (p.s.i)	3.1	12.5	25.0	50.0
Effective Lateral Pressure (p.s.i)	2.4	10.7	20.2	37.5
Deviator Stress at Max. σ_1/σ_3 (p.s.i.)	38.5	60.4	85.2	129.7

Tangent of Friction Angle = 0.68; Cohesion 9.0 (p.s.i)

TEST No. 17 – T.



Specimen No.	67	69	68	66
Placement Dry Density (p.c.f.)	121.26	120.15	118.46	106.81
Failure Dry Density (p.c.f.)	121.64	121.12	121.17	120.71
Placement Moisture Content (%)	11.42	11.41	11.40	11.38
Applied Lateral Pressure (p.s.i)	6.2	12.5	25.0	50.0
Effective Lateral Pressure (p.s.i.)	5.0	10.4	20.5	29.4
Deviator Stress at Max. σ_1/σ_3 (p.s.i)	69.2	81.9	97.9	109.6

Tangent of Friction Angle = 0.50; Cohesion 19.5 (p.s.i)

TEST No. 1 – C.

(*top*) placement density
(*bottom*) failure density

FIG. 13—Specimens Tested at Constant Placement and Failure Density.

shallow explorations hand-cut soil specimens are excellent. These are extremely important items when analyzing foundations for hydraulic structures in arid and semi-arid areas.

Load Removal—It is extremely difficult to control the effects of load removal because all specimens are expanded to some degree as they are removed from the *in situ* condition. Negative pore pressures can be induced even in natural, saturated, clayey soils when only limited men between the time it was taken and the time the laboratory tests were performed. These specimens were about 95 per cent saturated at the beginning of the laboratory test. It is felt that drainage and evaporation was limited through the use of expanding packers in the specimen tubes and preparation of the shear specimens in an 85 per cent humidity room. In recent studies of plastic clays from the foundation of San Luis Dam, the shear specimens tested showed

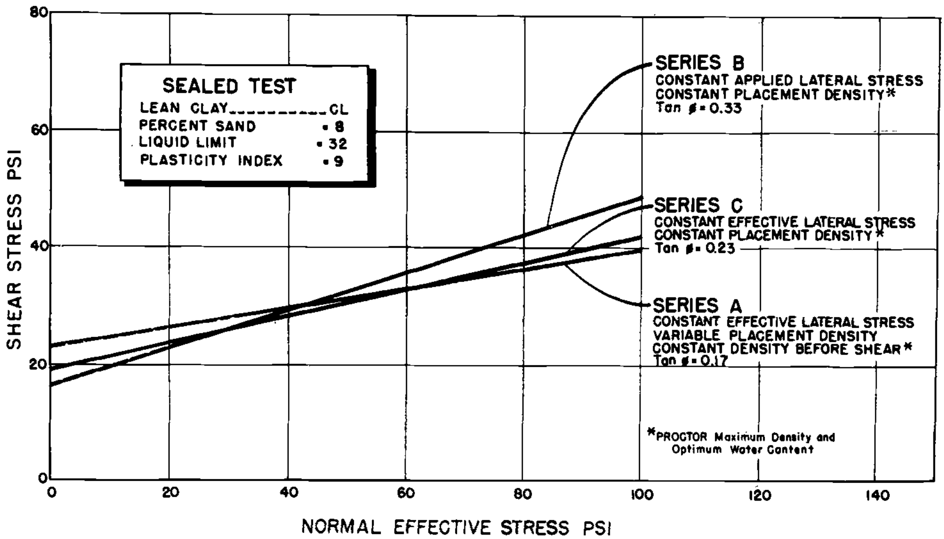


FIG. 14—Difference Between Triaxial Shear Tests With Specimens at Constant Placement Density and at Constant Placement Density at the Start of Shear Load Application.

expansion occurs. Figure 10 shows the difference between applied stresses and effective stresses in a drained consolidation test on a sandy clay soil when the negative pore pressures or capillary pressures are taken into account. The effect of these negative forces on shear strength interpretation is shown in Figure 11. Figure 12 shows the negative pore pressures that existed at the beginning of the test for a lean clay soil taken below lake water level in the Great Salt Lake area. The negative pressures measured at the beginning of the test can only be accounted for by an expansion of the specimen from 3 to 6 psi negative pressures and from 95 to 96 per cent saturation when tested in the laboratory. These specimens were taken in the perched groundwater table.

Resaturation of Laboratory Specimens—When specimens of clay, especially specimens of expansive clay, are resaturated in the laboratory, the strength of the soil may be changed to an extreme degree, depending upon the procedures used. For instance, specimens of expansive clay having cohesive strength values of 15 psi at natural water content (90 to 95 per cent saturation) may suffer a reduction in cohesion to as small an amount as 1 psi as they are allowed to saturate and expand in water without restraint.

The Effect of Placement Conditions on Effective Stress-Strain Parameter of Compacted Clays (Unsaturated Soils):

Placement density conditions of shear test specimens can greatly affect the results of the shear tests. Figure 13 shows the difference in shear strength values obtained with specimens of a lean clay prepared at constant placement density with the shear parameters obtained when the specimens were so prepared that failure took place at near constant density. Figure 14 demonstrates this further. Series *C* and *A* envelopes show the difference between triaxial shear tests with specimens at constant placement density as compared with those in which the specimens had a constant density at the start of shear load application (after all-around chamber pressure consolidation).

From Figure 13 it can be seen that the failure density of shear specimens is a controlling factor in the stress-strain parameters. The variations obtained in the failure density between specimens of a shear test suite may cause one to question the use of the Mohr-Coulomb criteria for soils. However, the slope of the envelope still has valid use for practical problem solutions, but we should not tie the slope of the envelope too closely to the actual friction component of the shear strength.

In all of the tests described above, the unconsolidated undrained procedure was used. Pore pressure measurements were taken and the results were expressed in terms of effective stresses.

L. G. CHAN[3]—*The Effect of Creep and Time of Loading on the Strength of Soils*—These comments are based on a review of creep tests performed on Seven Sisters clay, Floodway clay, and SSRD clays by PFRA.

The tests on Seven Sisters clay were run as consolidated undrained triaxial tests with 90 per cent of the maximum deviator stress of a companion consolidated undrained test left on until failure by creep. These tests showed a definite loss, though slight, in strength through creep.

The tests on Floodway clay and SSRD clays were run as companion unconsolidated undrained tests having the same minor principal stress, but the time of loading for each load increment was varied so that test duration varied from 15 to 90,000 min.

The Floodway clay specimens showed decreasing strengths as $(\sigma_1 - \sigma_3)$, with increasing test duration, or loss in strength through creep.

The SSRD (*A*) field-compacted glacial clay specimens decreased in strength as $(\sigma_1 - \sigma_3)$ with increasing test duration up to about 500 min, and strengths increased with increasing test duration from 500 to 60,000 min.

The SSRD (*B*) freshly laboratory compacted glacial clay specimens indicated a trend depending on the moisture content at which the specimen was compacted. Specimens compacted at a water content drier than the optimum moisture content for the material decreased in strength as $(\sigma_1 - \sigma_3)$ with increase in test duration from 400 to 40,000 min. Specimens compacted at optimum moisture content decreased in strength with increasing test duration time up to 500 min and then showed an increase in strength from 500 to 40,000 min. Specimens compacted at a moisture content wetter than optimum moisture content showed an increase in strength with increasing test duration to 40,000 min. It is suggested that this increase in strength may be caused by thixotropy.

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